December 2015

This version of the Diablo Canyon Power Plant Units 1 and 2 Final Safety Analysis Report Update (FSARU) is the licensee's version submitted to the NRC on May 6, 2015. This version has certain sensitive information identified by staff of the Nuclear Regulatory Commission (NRC) per 10 CFR 2.390(d)(1) that needs to be withheld from the public. The material included within is classified as non-publicly available information. As of December 2015, this is the latest FSARU revision submitted to the NRC.

The sensitive information was identified due to meeting the NRC's criteria on sensitive information, as specified in SECY-04-0191, "Withholding Sensitive Unclassified Information Concerning Nuclear Power Reactors from Public Disclosure," dated October 19, 2004, ADAMS ML042310663, as modified by the NRC Commissioners Staff Requirements Memorandum on SECY-04-0191, dated November 9, 2004, ADAMS ML043140175.

The following information was considered sensitive by NRC staff:

Figure	Drawing	Description
9.5-1	No drawing no.	Fire Protection System – Water System
9.5-2	No drawing no.	Fire Protection System - Seismically Qualified Portion of Water System
9.5-3	No drawing no.	Fire Protection System - Carbon Dioxide System
9.5F-20A	No drawing no.	Buttress Area, Unit 1, Elevation 85'
9.5F-20B	No drawing no.	Buttress Area, Unit 1, Elevation 85'
9.5F-20C	No drawing no.	Buttress Area, Unit 2, Elevation 85'
9.5F-20D	No drawing no.	Buttress Area, Unit 2, Elevation 104'
9.5F-21	No drawing no.	Fire Areas, Buttress Area
9.5F-22	No drawing no.	Fire Areas, Buttress Area, Section A1-A1
9.5F-23	No drawing no.	Fire Areas, Buttress Area, Section B1-B1

9.5F-24	No drawing no.
9.5F-25	No drawing no.
9.5F-26	No drawing no.
9.5F-27	No drawing no.
9.5F-28	No drawing no.
9.5F-29	No drawing no.
9.5F-30	No drawing no.
9.5F-31	No drawing no.
9.5F-32	No drawing no.
9.5F-33	No drawing no.
12.1-1	No drawing no.
12.1-2	No drawing no.
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12.1-4	No drawing no.
12.1-5	No drawing no.
12.1-6	No drawing no.
12.1-7	No drawing no.
12.1-8	No drawing no.
12.1-9	No drawing no.

12.1-10	No drawing no.
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Any other material that is listed as "deleted" was deleted by the licensee as part of their continuous update process for the FSARU.

Pacific Gas and Electric Company®

Diablo Canyon Power Plant Units 1 and 2

Final Safety Analysis Report Update

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Chapter 1

INTRODUCTION AND GENERAL DESCRIPTION OF PLANT

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FIGURES

NOTE:

[a] This figure corresponds to a controlled engineering drawing that is incorporated by reference into the FSAR Update. See Table 1.6-1 for the correlation between the FSAR Update figure number and the corresponding controlled engineering drawing number.

CHAPTER 1

INTRODUCTION AND GENERAL DESCRIPTION OF PLANT

1.1 INTRODUCTION

The Final Safety Analysis Report (FSAR) Update for the Diablo Canyon Power Plant (DCPP) is submitted in accordance with the requirements of 10 CFR 50.71(e) and contains all the changes necessary to reflect information and analyses submitted to the U.S. Nuclear Regulatory Commission (NRC) by Pacific Gas and Electric Company (PG&E) or prepared by PG&E pursuant to NRC requirements since the submittal of the original FSAR. The original FSAR was submitted in support of applications for permits to operate two substantially identical nuclear power units (Unit 1 and Unit 2) at the DCPP site. The DCPP site is located on the central California coast in San Luis Obispo County, approximately 12 miles west southwest of the city of San Luis Obispo.

The Construction Permit for Unit 1 (CPPR-39) was issued April 23, 1968, in response to PG&E's application dated January 16, 1967 (USAEC, Docket No. 50-275). The Construction Permit for Unit 2 (CPPR-69) was issued on December 9, 1970; the application was made on June 28, 1968 (USAEC, Docket No. 50-323).

Westinghouse Electric Corporation and PG&E jointly participated in the design and construction of each unit. The plant is operated by PG&E. Each unit employs a pressurized water reactor (PWR) nuclear steam supply system (NSSS) furnished by Westinghouse Electric Corporation and similar in design concept to several projects licensed by the NRC. Certain components of the auxiliary systems are shared by the two units, but in no case does such sharing compromise or impair the safe and continued operation of either unit. Those systems and components that are shared are identified and the effects of the sharing are discussed in the chapters in which they are described. The NSSS for each unit is contained within a steel-lined reinforced concrete structure that is capable of withstanding the pressure that might be developed as a result of the most severe postulated loss-of-coolant (LOCA) accident. The containment structure was designed by PG&E to meet the requirements specified by Westinghouse Electric Corporation.

While the reactors, structures, and all auxiliary equipment are substantially identical for the two units, there is a difference in the reactor internal flow path that results in a lower coolant flow rate for Unit 1. Consequently, the original license application reactor ratings were 3,338 MWt for Unit 1 and 3,411 MWt for Unit 2. The corresponding net electrical outputs were approximately 1,084 MWe and 1,106 MWe, respectively.

During the design phase, the expected ultimate output of the Unit 1 reactor was 3,488 MWt; the expected ultimate output of the Unit 2 reactor was 3,568 MWt. The corresponding NSSS outputs were 3,500 MWt and 3,580 MWt. (The difference of 12 MWt is due to the net contribution of heat to the reactor coolant system from nonreactor sources, primarily pump heat.) The corresponding estimated ultimate net electrical outputs were 1,131 MWe for Unit 1 and 1,156 MWe for Unit 2.

The NRC issued a low power-operating license for Unit 1 on September 22, 1981. PG&E voluntarily postponed fuel loading due to the discovery of design errors in the annulus region of the containment structure. Subsequently, the NRC suspended portions of the license on November 19, 1981, pending completion of an Independent Design Verification Program.

After completion of redesign and construction activities in November 1983, the NRC reinstated the fuel load portion of the Unit 1 low power-operating license. On April 19, 1984, the NRC fully reinstated the low power-operating license, which included low power testing. The Unit 1 full power-operating license was issued on November 2, 1984. Commercial operation for Unit 1 began on May 7, 1985, with a license expiration date of April 23, 2008.

The NRC issued a low power-operating license for Unit 2 on April 26, 1985. Unit 2 fuel loading was completed on May 15, 1985. A full power-operating license for Unit 2 was issued on August 26, 1985. Unit 2 commercial operation began on March 13, 1986, with a license expiration date of December 9, 2010.

In March 1996, the NRC approved license amendments extending the operating license for Unit 1 until September 22, 2021, and for Unit 2 until April 26, 2025.

In July 2006, the NRC approved license amendments extending the operating license for Unit 1 until November 2, 2024, and for Unit 2 until August 26, 2025.

In October 2000, the NRC approved a license amendment (LA) 143 to increase the Unit 1 rated reactor thermal power from the original value of 3,338 MWt to 3,411 MWt to increase production and be consistent with Unit 2. LA 143 also documented the evaluation performed to revise the net contribution of heat to the reactor coolant system from nonreactor sources (primarily pump heat) to a nominal value of 14 MWt and established a NSSS power outlet of 3,425 MWt for both Unit 1 and Unit 2.

1.2 GENERAL PLANT DESCRIPTION

1.2.1 PRINCIPAL SITE CHARACTERISTICS

1.2.1.1 Location

The DCPP site consists of approximately 750 acres located in San Luis Obispo County, California, adjacent to the Pacific Ocean and roughly equidistant from San Francisco and Los Angeles. The site location, the site boundary, and the location of principal structures are shown in Figure 1.2-1. The minimum distance from either reactor to the nearest site boundary on land is one-half mile, the minimum exclusion distance. The low population zone (LPZ), as defined in 10 CFR 100, is the area immediately surrounding the exclusion area. For DCPP, the LPZ is an area encompassed by a radius of 6.2 miles. This zone contains residents for whom there is reasonable probability that appropriate protective measures, as described in the DCPP Emergency Plan (Reference 1) can be taken in the event of a serious accident. The population center distance, as defined by 10 CFR 100, is approximately 10 miles, the distance to the nearest boundary of the city of San Luis Obispo.

1.2.1.2 Topography

The plant site occupies a coastal terrace that ranges in elevation from 60 to 150 feet above sea level and is approximately 1000 feet wide. Plant grade is at elevation 85 feet. The seaward edge of the terrace is a near-vertical cliff. Back from the terrace and extending for several miles inland are the rugged Irish Hills, an area of steep, brush-covered hillsides and deep canyons that are part of the San Luis Mountains and attain an elevation of 1500 feet within about a mile of the site. Access to the site is by a private road from Avila Beach, a distance of nearly 8 miles.

1.2.1.3 Meteorology

The climate of the site area is typical of that along the central California coast. In the dry season, mainly May through September, the Pacific Anticyclone stays off the California coast and prevents Pacific storms from moving eastward across the state. In the winter or wet season, November through March, the Pacific Anticyclone moves southward, weakening in intensity, and allows Pacific storms to enter the state. More than 80 percent of the average annual rainfall of 16 inches occurs during this 5-month period. April and October are considered transitional months. The average annual temperature of the site area is about 55°F, which reflects the strong maritime influence.

Most stations along the coast show a 5 to 10°F mean temperature difference between the coldest winter month and the warmest summer month. Extreme temperatures may range from 104°F in the summer to as low as 24°F in the winter. However, the recurrence interval of days having these extremes is in the order of 5 to 10 years. Maximum summer temperatures of 85°F and minimum winter temperatures of 35°F are exceeded only 1 percent of the time at both Morro Bay and Pismo Beach. Additional site temperature data are presented in Section 2.3. The onsite meteorological measurements program was initiated in July 1967. Data collected are presented in Section 2.3 and are used to establish atmospheric diffusion characteristics of the site. Severe weather conditions, such as tornadoes and hurricanes, have not been recorded in this area. Thunderstorms are also a rare phenomenon with the average occurrence of lightning being less than 3 days per year.

1.2.1.4 Hydrology

Hydrological considerations at the plant site are limited to possible effects of plant operations on domestic water supplies and to the possibility of flooding. A survey of domestic water supplies in the environs shows that operation of the plant will not jeopardize any existing or planned facility. The topography of the site and the limited rainfall preclude any possibility of flooding.

1.2.1.5 Geology

A comprehensive geological investigation has demonstrated that the site is geologically suitable for a nuclear power plant. Foundations are on firm bedrock fully capable of carrying the loads. Movement along the few small breaks in the vicinity of the plant has not occurred for at least 100,000 years and may well have taken place millions of years ago. The site was investigated in detail for faulting and other possibly detrimental geologic conditions. Results of faulting investigations are discussed in Section 2.5.3.7 and are based on site geology data presented in Section 2.5.1.2. Landslides do not threaten the plant.

1.2.1.6 Seismology

Seismological investigations were undertaken to determine the potential for earthquakes in the site area, to form a basis of the establishment of seismic design criteria, and to evaluate the adequacy of seismic design margins for the plant (Section 2.5). Records indicate that seismic activity within 20 miles of Diablo Canyon has been very low compared to other parts of California. Until PG&E’s seismological investigation of the Hosgri fault zone located approximately 3 miles offshore, the seismically significant fault system nearest the site was considered to be the Nacimiento Fault located about 20 miles away as discussed in Section 2.5.2.9. The largest earthquake known to have been associated with this fault system occurred at an epicentral distance to the site of about 44 miles. It is listed with a Richter magnitude 6. At its closest point, the San Andreas Fault passes some 48 miles from the site.

PG&E's reevaluation of the plant's capability to withstand a postulated Richter magnitude 7.5 "Hosgri" earthquake is discussed in Section 3.7.

1.2.1.7 Oceanography

Condenser cooling water for the plant is pumped from the Pacific Ocean and returned to the ocean at Diablo Cove through an outfall at the water's edge. Controlled releases of low-level liquid radioactive wastes are discussed in Section 11.2. The Pacific Ocean in the area of the site is turbulent and has a great capacity for dilution of wastes and diffusion of heated cooling water. Investigations of the occurrence and maximum size of tsunamis (seismic sea waves) coincident with high tide and with short period storm waves are discussed in Design Criteria Memorandum T-9, Appendix A. These studies showed that extreme water elevation within the intake basin without a breakwater would be 44.32 feet above mean lower low water. The intake structure houses the safety-related auxiliary cooling water systems, which are protected against tsunami and wave splash with watertight compartments. This is discussed in Section 2.4.5.7.

1.2.2 FACILITY DESCRIPTION

The plant incorporates two substantially identical PWR nuclear power units, each consisting of an NSSS, turbine-generator, auxiliary equipment, controls, and instrumentation. The general arrangement of the plant and the site is shown in Figure 1.2-1. Principal structures, shown in Figure 1.2-2, include the containment structures, turbine building, and auxiliary building (which includes the control room, the fuel handling areas, and the ventilation areas). Arrangement plans and sections are shown in Figures 1.2-3 through 1.2-32. The descriptions that follow apply to both units unless otherwise specified.

1.2.2.1 Design Criteria

The principal design criteria for the DCPP nuclear units are those fundamental architectural and engineering design objectives established for the plant. The bases for development and selection of the design criteria used in this plant are: (a) those that provide protection to public health and safety, (b) those that provide for reliable and economic plant performance, and (c) those that provide an attractive external appearance to the plant.

The essential systems and components of the plant are designed to enable the facility to withstand, without loss of capability to protect the public, the forces resulting from normal operation plus those that might be imposed by natural phenomena. The designs are based on the most severe of the natural phenomena recorded for the vicinity of the site, with margin to account for uncertainties in the historical data.

The DCPP units are designed to comply with the "General Design Criteria for Nuclear Power Plant Construction Permits," published in July 1967. A discussion of conformance to these criteria is contained in Section 3.1. In addition, a summary discussion of the designs and procedures that are intended to meet the NRC General Design Criteria published as Appendix A to 10 CFR 50 in 1971 is provided in Chapter 3, Section 3.1, Appendix 3.1A.

1.2.2.2 Nuclear Steam Supply System

The NSSS consists of a PWR and associated auxiliary fluid systems. The reactor coolant system (RCS) consists of four parallel reactor coolant loops, each containing a steam generator and a reactor coolant pump. A pressurizer is connected to the hot leg of one reactor coolant loop.

The reactor core is composed of an array of 193 fuel assemblies, each containing 264 fuel rods. These rods are composed of uranium dioxide pellets enclosed in zirconium alloy tubes with welded end plugs. All fuel rods are pressurized with helium during fabrication to reduce stress and increase fatigue life. Reactor control and shutdown functions are performed by the rod cluster control assemblies (RCCAs). The RCCAs are stainless steel tubes containing a silver-indium-cadmium absorber and are positioned by drive mechanisms of the magnetic latch type. A soluble poison (boron) is introduced into the reactor coolant to compensate for long-term reactivity changes. The moderator temperature coefficient can be slightly positive at the beginning of cycle when boron concentration is high. However, for most operating conditions, the moderator coefficient is non-positive, but the power coefficient is negative at all times.

The reactor vessel and reactor internals contain and support the fuel and RCCAs. The vessel is cylindrical with hemispherical heads and is clad with stainless steel.

The pressurizer is a vertical cylindrical pressure vessel with hemispherical heads and is equipped with electrical heaters and spray nozzles for system pressure control.

The steam generators are vertical U-tube type heat exchangers with Inconel tubes. Reactor coolant flows inside the tubes; steam is generated in the shell and flows through the main steam lines to the turbine. When operating at 100 percent power, integral moisture separating equipment reduces moisture content of the steam at the exit of the steam generators to ≤0.05 percent. Under transient conditions at ≤100 percent power, the moisture content at the exit of the steam generators is <0.25 percent. The reactor coolant pumps are vertical, single-stage, centrifugal units equipped with controlled leakage shaft seals.

Auxiliary systems are provided to charge the RCS and add makeup water, to purify reactor coolant water, to provide chemicals for corrosion inhibition and reactor control, to cool system components, to remove residual heat when the reactor is shut down, to cool the spent fuel storage pool, to sample reactor coolant water, to provide for emergency safety injection, and to vent and drain the RCS.

1.2.2.3 Engineered Safety Features

The engineered safety features (ESFs) provided for the DCPP have sufficient capacity and redundancy to protect the health and safety of the public by keeping exposure below the limits set forth in 10 CFR Part 100 for any postulated malfunction or accident, including the most severe LOCA.

The ESFs provided in the DCPP are:

(1) A containment system that consists primarily of a steel-lined, reinforced concrete containment structure designed to prevent significant release to the environs of radioactive materials that could result from accidents inside the containment (refer to Sections 6.2.1 and 6.2.4).

(2) An emergency core cooling system (ECCS) that provides water to cool the core in the event of an accidental loss of primary reactor coolant water. The ECCS also supplies dissolved boron into the cooling water to provide shutdown margin (refer to Section 6.3).

(3) A containment spray system (CSS) to help limit the peak temperature and pressure in the containment in the event of a LOCA or main steam line break (MSLB) (refer to Section 6.2.2). The CSS, in conjunction with the spray additive system (SAS), also helps to limit the offsite radiation levels following the postulated LOCA by removing airborne iodine from the containment atmosphere during the injection phase.

(4) A containment fan cooler system (CFCS) that functions in conjunction with the CSS to limit the temperature and pressure in the containment structure in the event of a LOCA or MSLB (refer to Section 6.2.2). The CFCS also provides mixing of the sprayed and unsprayed regions of the containment atmosphere to improve airborne fission product removal (refer to Section 6.2.3). The CFCS function of mixing the containment atmosphere for hydrogen control is discussed below.

(5) A SAS that functions by adding sodium hydroxide, an effective iodine scrubbing solution, to the CSS water, to reduce the content of iodine and other fission products in the containment atmosphere and prevent the re-evolution of the iodine in the recirculated core cooling solution following a LOCA (refer to Section 6.2.3).

(6) The long term buildup of gaseous hydrogen in the containment following a LOCA is primarily controlled by ensuring a mixed containment atmosphere and providing equipment for monitoring hydrogen concentrations. The CFCS is the primary means credited for containment atmosphere mixing (refer to Section 6.2.5).

(7) A fuel handling building ventilation system (FHBVS) provides a significant reduction in the amounts of volatile radioactive materials that could be released to the atmosphere in the event of a major fuel handling accident (refer to Section 9.4.4).

(8) An auxiliary building ventilation system (ABVS) that provides the capability for significant reduction in the amounts of volatile radioactive materials

that could be released to the atmosphere in the event of leakage from the residual heat removal (RHR) system recirculation loop following a LOCA (refer to Section 9.4.2).

(9) A control room ventilation system (CRVS) permits continuous occupancy of the control room and technical support center (TSC) under design basis accidents by providing the capability to control infiltration of volatile radioactive material (refer to Sections 6.4.1 and 9.4.1).

(10) An auxiliary feedwater (AFW) system supplies water to the secondary side of the steam generators for reactor decay heat removal, when the main feedwater system is unavailable (refer to Section 6.5).

1.2.2.4 Instrumentation and Control

The primary purpose of the instrumentation and control system is to provide automatic protection against unsafe and improper reactor operation during steady state and transient power operation (ANS Conditions I, II, and III) and to provide initiating signals to mitigate the consequences of faulted conditions (ANS Condition IV). These plant conditions are discussed in Chapter 15, Accident Analysis.

The operation of the plant is monitored and controlled by operators in the control room, which is located in the auxiliary building.

1.2.2.5 Electrical Systems

The electrical systems generate and transmit power to PG&E's high-voltage system, distribute power to the auxiliary loads, and provide control, protection, instrumentation, and annunciator power supplies for the units. Power is generated at 25 kV. Auxiliary loads are served at 12 kV, 4.16 kV, 480 V, 120 Vac, 125 Vdc, and 250 Vdc.

Offsite ac power for the units' auxiliaries is available from two 230-kV transmission circuits and three 500-kV transmission circuits.

Onsite ac auxiliary power is supplied by each unit's main generator and is also available for vital loads from six diesel engine-driven generators. Three diesel generators are dedicated to each unit.

Onsite dc power is provided by three vital and two nonvital 125-V batteries in each unit. The two nonvital batteries are connected in series to provide 250-Vdc power in each unit.

1.2.2.6 Power Conversion System

The turbines are each tandem-compound, four-element, 1800 rpm units, having one high-pressure and three identical double flow low-pressure elements. Combination moisture separator-reheaters are employed between the high- and low-pressure elements to dry and superheat the steam. The auxiliaries include deaerating surface condensers, steam jet air ejectors, motor-driven condensate pumps, motor-driven condensate booster pumps, turbine-driven main feedwater pumps, six stages of feedwater heating, and a full flow condensate demineralizer system.

The steam and power conversion system is designed to receive the heat generated by the RCS during normal power operation, as well as following an emergency shutdown of the turbine-generator from full load. Heat rejection under the latter condition is accomplished by steam bypass to the condenser and pressure relief to the atmosphere.

1.2.2.7 Fuel Handling and Storage

The reactor is refueled using equipment designed to handle spent fuel under water from the time it leaves the reactor vessel until it is placed in a cask either for transport to the Diablo Canyon Independent Spent Fuel Storage Installation or for shipment from the site. Underwater transfer of spent fuel provides an optically transparent radiation shield, as well as a reliable source of coolant for removal of decay heat. Spent fuel is stored onsite in the spent fuel pools, which are fitted with special spent fuel storage racks to ensure that criticality cannot be approached. The fuel handling system also provides capability for receiving, handling, and storing new fuel assemblies.

1.2.2.8 Auxiliary Systems

Auxiliary systems are supporting systems included in the facility, some of which are required to perform certain functions during emergency or accident conditions. Included are the cooling water systems, the heating and ventilating systems, the fire protection system, the process auxiliaries, the compressed air system, the diesel generator fuel oil system, the communication systems, and the lighting systems.

1.2.2.9 Radioactive Wastes

The radioactive waste treatment systems provide all equipment necessary to collect, process, monitor, and discharge radioactive liquid, gaseous, and solid wastes that are produced during reactor operation. A major portion of the waste treatment equipment is common for Units 1 and 2. This equipment is located in the shared auxiliary building.

1.2.2.10 Shared Facilities and Equipment

Separate systems and equipment are provided for each unit, with few exceptions. A brief summary of shared facilities and equipment between both units follows. Interconnections between systems for Unit 1 and Unit 2 are shown in the system diagrams. The system diagrams are contained in the FSAR Update chapters referenced in the following paragraphs.

1.2.2.10.1 Site Facilities

The two units share a common auxiliary building. The turbine building is common to both units. The machine shop, access control area, warehouse area, telecommunications systems, and administrative offices are common.

The two units also share a common raw water storage reservoir, fire pumps, fire water storage tank, diesel fuel oil storage tanks and transfer pumps, auxiliary boiler, makeup water system, plant air system, and lubricating oil storage system.

1.2.2.10.2 Electrical Systems

The 230-kV line from the 230-kV switchyard serves the standby/startup transformers for both Units 1 and 2. These are normally arranged on the low voltage sides to serve a single unit; however, the 12-kV buses for Units 1 and 2 are connected by an open circuit breaker.

The plant has six diesel generator sets for emergency power.

1.2.2.10.3 Control Room

The plant is provided with a central control room located in the auxiliary building which is common to Units 1 and 2. Physical separation of control panels prevents interaction of the Unit 1 and Unit 2 control systems.

1.2.2.10.4 Chemical and Volume Control System

Several components of the chemical and volume control system are shared, as detailed in Chapter 9.

1.2.2.10.5 Radioactive Waste Treatment Systems

The major portion of the waste treatment equipment is shared by Units 1 and 2. This equipment is located in the shared auxiliary building and is described in Chapter 11.

1.2.2.10.6 Emergency Facilities and Equipment

The emergency facilities and equipment, both onsite and offsite, are discussed in the Emergency Plan which applies to both Units 1 and 2.

1.2.3 REFERENCES

1. Emergency Plan, Diablo Canyon Power Plant - Units 1 and 2, Pacific Gas and Electric Company.

1.2.4 REFERENCE DRAWINGS

Figures representing controlled engineering drawings are incorporated by reference and are identified in Table 1.6-1. The contents of the drawings are controlled by DCPP procedures.

1.3 COMPARISON TABLES

1.3.1 COMPARISON WITH SIMILAR FACILITY DESIGNS

Table 1.3-1 presents a comparison of the principal similarities and differences of the design of the DCPP units with those of Unit 1 at Trojan Nuclear Power Plant and Units 1 and 2 at Zion Station. This comparison is historical in nature and is valid only through March 1984.

1.3.2 COMPARISON OF FINAL AND PRELIMINARY DESIGNS

Table 1.3-2 identifies the major design changes made since the submittal of the DCPP Unit 2 Preliminary Safety Analysis Report. The comparison was considered to be valid through July 1974.

1.4 IDENTIFICATION OF AGENTS AND CONTRACTORS

PG&E is the architect engineer, constructor, operator, and owner of DCPP and, as such, assumes full responsibility and authority for the design, construction, startup, and operation of Diablo Canyon Units 1 and 2. This section identifies the principal consultants, the nuclear steam system supplier, the suppliers of other equipment affecting nuclear safety, and the principal contractors engaged in the construction of the units. PG&E has performed all work for the planning, design, estimating, procurement, construction, installation, inspection, testing, and associated services necessary to provide complete plans and specifications and related services necessary to furnish a complete, operable, and acceptable plant, except for those items and functions that were furnished by those mentioned below.

1.4.1 CONSULTANTS

The consultants whose contracts exceed one hundred thousand dollars are listed in Table 1.4-1. The list is historical in nature and is valid only through March 1986. They performed investigations and submitted reports and recommendations to PG&E on the subjects indicated in the table. Application of the material submitted is the responsibility of PG&E.

1.4.2 NUCLEAR STEAM SUPPLY SYSTEM SUPPLIER

The NSSS was designed and furnished by the Westinghouse Electric Corporation. Westinghouse performed the detailed engineering design for all Westinghouse-supplied components and systems of the NSSS and procured, expedited, inspected, and delivered to PG&E all such equipment and components. Westinghouse provided design criteria, outline and/or assembly drawings, systems flow diagrams, and other data, as required, for PG&E to install, erect, operate, and maintain Westinghouse-supplied equipment and components. For all Westinghouse-supplied nuclear auxiliary systems, Westinghouse performed systems engineering, prepared reference designs and systems descriptions, and provided overall operating and engineering instructions.

Westinghouse further provided pertinent design criteria and data on the NSSS to enable PG&E to design the balance of plant. Westinghouse provided functional test procedures and technical assistance during construction, installation, inspections, and testing of its equipment and systems. A description of Westinghouse-provided technical assistance is given in Chapter 14, Initial Tests and Operations.

Westinghouse also performed the post-accident transient analysis of the plant containment system. This analysis consisted of determining the mass and energy releases (including metal-water reaction) as a function of time for the design basis LOCA and main steam line break. From the foregoing data, PG&E has determined the design pressure and temperature, containment volume cooling requirements, etc. In general, Westinghouse has performed such transient analyses on the plant as are required for the Westinghouse-furnished NSSS and turbine-generator unit. Transient analyses involving non-Westinghouse-supplied systems and components are PG&E's responsibility.

Westinghouse also supplied material that is informational in nature, some of which appears in response to questions asked of Westinghouse during review meetings. Other information and recommendations have been offered by Westinghouse from their background experience. This type of material is not contractually binding for either company, nor was it intended to be a commitment of final design or operation. This material includes:

(1) The conceptual design of the dry reactor containment system. The specific designs of the reactor containment structure and associated ESFs are developed by PG&E.

(2) The details of a recommended waste disposal system that consists of equipment to collect, process, and dispose of radioactive liquid, gaseous, and solid wastes produced as a result of reactor operation

(3) Five recommended ESFs that consist of: steel-lined concrete reactor containment vessel, the safety injection system, the containment fan coolers, the containment spray equipment, and the air recirculation filters. Emergency power systems to operate the ESF systems are also as recommended by Westinghouse.

(4) The details of the recommended fuel handling facilities including structures, equipment, transfer, and operation

(5) The details of the recommended sampling system and analytical facilities

(6) The details of the recommended radiation shielding

(7) The outline of a recommended health physics program and recommended supplies

(8) The general criteria and preliminary design data for certain balance of plant.

Westinghouse, as a supplier to PG&E, is required to conform to the PG&E Quality Assurance Program as described in Chapter 17, Quality Assurance.

1.4.3 OTHER EQUIPMENT SUPPLIERS

Suppliers of important equipment or materials furnished to PG&E are listed in Table 1.4-2. In each case, the equipment was fabricated, or the material supplied qualified, to written specifications and, if Design Class I, under the Quality Assurance

Program in effect at the time of purchase. This list of suppliers is historical in nature and is valid only through March 1986.

1.4.4 CONSTRUCTION AND INSTALLATION CONTRACTORS

The principal construction and installation contractors whose contracts exceed one hundred thousand dollars are listed in Table 1.4-3. The list of contractors is historical in nature and is valid only through March 1986. The contracts are agreements between PG&E as owner-constructor and the contractors as independent contractors, with specific provisions for inspection, testing, and quality assurance.

Each contract specification identifies any Design Class I equipment involved and requires the implementation of the supplier's Quality Assurance Program (see Chapter 17, Quality Assurance). In addition, PG&E maintains a staff of inspectors to assure the quality of non-Class I equipment installation.

1.5 REQUIREMENTS FOR FURTHER TECHNICAL INFORMATION

The design of DCPP is based on proven concepts, systems, and equipment in order to minimize the potential for cost and schedule overruns and to enhance the reliability of operation. As a consequence, there have been few requirements for research and development programs to confirm the adequacy of the design. Those programs identified for DCPP have been satisfactorily completed, as well as any other programs that have been identified as valuable to define margins of conservatism or possible design improvements. Table 1.5-1 is a list of those programs that have been addressed in earlier revisions of the original FSAR. This table provides a listing of the technical reports that include a discussion the programs and their results. The listing is historical in nature and is valid only through November 1975.

1.6 MATERIAL INCORPORATED BY REFERENCE

1.6.1 WESTINGHOUSE TECHNICAL REPORTS

Title	Section Reference	Date Submitted To AEC/NRC
1. C. J. Kubit, Safety-Related Research and Development for Westinghouse Pressurized Water Reactors, Program Summaries-Fall, 1971-Spring, 1972, WCAP-7856, April 1972.	1.5	5/9/72
2. F. T. Eggleston, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Winter 77 - Summer 78, WCAP-8768, Rev. 2, October 1978.	1.5	10/78
3. R. M. Hunt, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Fall 1970, WCAP-7614-L, November 1970.	1.5	11/70
4. R. M. Hunt, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Spring 1970, WCAP-7498-L, May 1970.	1.5	5/70
5. M. D. Davis, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Fall 1974, WCAP-8485, March 1975.	1.5	3/75
6. C. J. Kubit, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Spring 1974, WCAP-8385, July 1974.	1.5	7/74
7. C. J. Kubit, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Spring 1972, WCAP-7856, May 1972.	1.5	5/72
8. J. M. Hellman, Fuel Densification Experimental Results and Model for Reactor Operation, WCAP-8218-P-A, March 1975 (Proprietary) and WCAP-8219-A, March 1975 (Non-Proprietary).	1.5	3/75
9. L. Geninski, et al, Safety Analysis of the 17 x 17 Fuel Assembly for Combined Seismic and Loss-of-Coolant Accident, WCAP-8288, December 1973.	1.5	12/73

Title	Section Reference	Date Submitted To AEC/NRC
10. E. E. DeMario and S. Nakazato, Hydraulic Flow Test of the 17 x 17 Fuel Assembly, WCAP-8279, February 1974.	1.5	2/74
11. F. W. Cooper, Jr., 17 x 17 Drive Line Components Test - Phase IB, II, III - Drop and Deflection, WCAP-8446, December 1974.	1.5	12/74
12. K. W. Hill, et al, Effect of 17 x 17 Fuel Assembly Geometry on DNB, WCAP-8396-P-A, Feb. 1975 (Proprietary) and WCAP-8297-A, February 1975.	1.5	2/75
13. Motley, F. W., et al, The Effect of 17 x 17 Fuel Assembly Geometry on Interchannel Thermal Mixing, WCAP-8298-P-A (Proprietary) and WCAP-8299-A, January 1975.	1.5	1/75
14. A. J. Burnett and S. D. Kopelic, Westinghouse ECCS Evaluation Model October 1975 Version, WCAP-8622 (Proprietary) and WCAP-8623 (Non-Proprietary), November 1975.	1.5	11/75
15. Irradiation of 17 x 17 Demonstration Assemblies in Surry Units No.1 and 2, Cycle 2, WCAP-8262, July 1974.	1.5	7/74
16. G.J. Bohm, Indian Point Unit 2 Internals Mechanical Analysis for Blowdown Exitation, WCAP-7822, December 1971.	3.9	12/20/71
17. P. M. Wood, et al, Use of Burnable-Poison Rods in Westinghouse Pressurized Water Reactors, WCAP-7113, October 1967.	3.9	11/6/67
18. R. F. Barry, et al, Power Distribution Monitoring in the R. E. Ginna PWR, WCAP-7756, September 1971.	3.9	10/5/71
19. L. T. Gesinski, Fuel Assembly Safety Analysis for Combined Seismic and Loss-of-Coolant Accident, WCAP-7950, July 1972.	3.9	7/14/72
20. S. Fabic, Description of the BLODWN-2 Computer Code, WCAP-7918, Rev. 1, October 1970.	3.9	10/70

	Title	Section Reference	Date Submitted To AEC/NRC
21.	S. Fabic, Loss-of-Coolant Analysis: Comparison Between BLODWN-2 Code Results and Test Data, WCAP-7401, November 1969.	3.9	2/5/70
22.	S. Kraus, Neutron Shielding Pads, WCAP-7870 including Appendix B, February 1972.	4.2	2/17/72
23.	C. M. Friedrich and W. H. Guilinger, CYGLO-Z, A Fortran IV Computer Program for Stress Analysis of the Growth of Cylindrical Fuel Elements with Fission Gas Bubbles, WCAP-TM-574, November 1966.	4.2	11/66
24.	A. F. McFarlane, Power Peaking Factors: WCAP-7912L, June 1972 (Westinghouse Proprietary) and WCAP-7912, March 1972.	4.3	3/8/72
25.	J. A. Christensen, et al, Melting Point of Irradiated UO_2, WCAP-6065, February 1965.	4.4	2/65
26.	G. Hetsroni, Hydraulic Tests of the San Onofre Reactor Model, WCAP-3269-8, June 1964.	4.4	6/64
27.	G. Hetsroni, Studies of the Connecticut-Yankee Hydraulic Model, WCAP-2761, June 1965, (NYO-3250-2).	4.4	6/65
28.	J. S. Shefcheck, Application of the THINC Program to PWR Design, WCAP-7359-L, August 1969 (Westinghouse Proprietary) and WCAP-7838, January 1972.	4.4, 15.1, 15.2, 15.4	1/17/72
29.	F. D. Carter, Inlet Orificing of Open PWR Cores, WCAP-9004, (Westinghouse Proprietary) and WCAP-836, January 1972.	4.4	3/19/69 1/17/72
30.	J. A. Nay, Process Instrumentation for Westinghouse Nuclear Steam Supply System, WCAP-7671, April 1971.	5.2, 7.1, 7.2, 7.3	5/10/71
31.	W. O. Shabbits, Dynamic Fracture Toughness Properties of Heavy Section-A-533 Grade B Class I Steel Plate, WCAP-7623, September 1972.	5.2	9/15/72

Title	Section Reference	Date Submitted To AEC/NRC
32. W. S. Hazelton, et al, Basis for Heatup and Cooldown Limit Curves, WCAP-7924, July 1972.	5.2	8/14/72
33. F. Bordelon and A. Nahavandi, A Space-Dependent Loss of Coolant Accident and Transient Analysis for PWR Systems (SATAN Digital Computer Code), WCAP-7845, January 1972.	5.2	8/9/72
34. J. Locante and E. G. Igne, Environmental Testing of Engineered Safety Features Related Equipment (NSSS-Standard Scope), WCAP-7744, Volume 1, August 1971.	5.5, 6.3	9/14/71
35. J. B. Lipchak and R. A. Stokes, Nuclear Instrumentation System, WCAP-7669, April 1971.	7.1, 7.2, 7.7	5/6/71
36. D. N. Katz, Solid State Logic Protection System Description, WCAP-7672, June 1971.	7.1, 7.2, 7.3	5/27/71
37. J. T. Haller, Engineered Safeguard Final Device or Activator Testing, WCAP-7705, February 1973.	7.1	2/27/73
38. J. Bruno, Isolation Tests Process Instrumentation Isolation Amplifier, Westinghouse Computer and Instrumentation Division, Model 131-110, WCAP-7824, December 1971.	7.2	12/16/71
39. R. Bartholomew and J. Lipchak, Test Report, Nuclear Instrumentation System Isolation Amplifier, WCAP-7819, Revision 1, January 1972.	7.2	1/17/72
40. W. C. Gangloff, An Evaluation of Anticipated Operational Transients in Westinghouse Pressurized Water Reactors, WCAP-7486, May 1971.	7.2, 15.2	5/21/71
41. T. W. T. Burnett, Reactor Protection System Diversity in Westinghouse Pressurized Water Reactors, WCAP-7306, April 1969.	7.2, 15.4	4/9/69
42. A. E. Blanchard, Rod Position Monitoring, WCAP-7571, March 1971.	7.7	4/5/71

Title	Section Reference	Date Submitted To AEC/NRC
43. J. J. Loving, In-Core Instrumentation (Flux Mapping System and Thermocouples), WCAP-7607, July 1971.	7.7	7/27/71
44. H.G. Hargrove, FACTRAN, A Fortran IV Code for Thermal Transients in a UO_2 Fuel Rod, WCAP-7908-A, December 1989.	15.1,15.2, 15.3,15.4	11/29/83
45. Deleted in Revision 22		
46. Deleted in Revision 22		
47. T. W. T. Burnett, et al, LOFTRAN Code Description, WCAP-7907-P-A (Proprietary), April 1984 and WCAP-7907-A (Non-Proprietary), April 1984.	6.2D,15.1, 15.2, 15.3,15.4	10/11/72
48. R. F. Barry, LEOPARD - A Spectrum Dependent Non-Spatial Depletion Code for the IBM-7094, WCAP-3269-26, September 1963.	4.3	9/63
49. R. F. Barry, et al., The TURTLE Diffusion Depletion Code, WCAP-7213, June 1968 (Westinghouse Proprietary) and WCAP-7758, September 1971.	4.3	6/68, 9/71
50. D. H. Risher, Jr. and R. F. Barry, TWINKLE - A Multi-Dimensional Neutron Kinetics Computer Code, WCAP-7979-P-A (Proprietary) and WCAP-8028-A (Non-Proprietary), January 1975.	15.1,15.2, 15.3	1/4/73
51. Deleted in Revision 22		
52. F. M. Bordelon, Calculation of Flow Coastdown after Loss of Reactor Coolant Pump (Phoenix Code), WCAP-7973, August 1973.	15.1	1/17/73
53. M. Ko, Setpoint Study for PG&E Diablo Canyon Units 1 and 2, WCAP-8320, June 1974.	15.1	6/74
54. F. W. Cooper, Jr., 17x17 Drive Line Components Tests Phase IB-II-III D-Loop Drop and Deflection, WCAP-8446 (Proprietary) and WCAP-8449 (Non-proprietary), (December 1974).	15.1	12/74

	Title	Section Reference	Date Submitted To AEC/NRC
55.	J. V. Miller (Ed.), Improved Analytical Methods Used in Westinghouse Fuel Rod Design Calculations, WCAP-8720, October 1976.	15.1	10/76
56.	F. E. Motley and F. F. Cadek, DNB Test Results for New Mixing Vane Grids (R), WCAP-7695-P-A (Proprietary) and WCAP-7958-A (Non-Proprietary), January 1975.	4.4	1/75
57.	Cooper, et al., Overpressure Protection for Westinghouse Pressurized Water Reactors, WCAP-7769, Revision 1, June 1972.	15.2	10/8/71
58.	D.S. Huegel, et al, RETRAN-02 Modeling and Qualification for Westinghouse Pressurized Water Reactor Non-LOCA Safety Analyses, WCAP-14882-P-A (Proprietary), April 1999 and WCAP-15234-A (Non-Proprietary), May 1999	15.2	6/6/97
59.	M.E. Nissley, et al, Realistic Large-Break LOCA Evaluation Methodology Using the Automated Statistical Treatment of Uncertainty Method (ASTRUM), WCAP-16009-P-A, January 2005	15.4	6/2/2003
60.	T. Q. Nguyen, et al, Qualification of the PHOENIX-P/ANC Nuclear Design System for Pressurized Water Reactor Cores, WCAP-11596-P-A (Proprietary),June 1988	4.3, 15.1	11/87
61.	Y. S. Liu, et al, ANC: A Westinghouse Advanced Nodal Computer Code, WCAP-10965-P-A (Proprietary), September 1986 and WCAP-10966-A (Non-Proprietary), September 1986	15.1	5/86

	Title	Section Reference	Date Submitted To AEC/NRC
62.	S. M. Bajorek, et al, Code Qualification Document for Best Estimate LOCA Analysis, WCAP-12945-P-A, Volume I: Models and Correlations, Revision 2, March 1998 Volume II: Heat Transfer Model Validation, Revision 1, March 1998 Volume III: Hydrodynamics, Components, and Integral Validation, Revision 1, March 1998 Volume IV: Assessment of Uncertainty, Revision 1, March 1998 Volume V: Quantification of Uncertainty, Revision 1, March 1998	15.1, 15.4	5/97
63.	N. Lee, et al, Westinghouse Small Break ECCS Evaluation Model Using the NOTRUMP Code, WCAP-10054-P-A (Proprietary) and WCAP-10081-A (Non-Proprietary), August 1985 and , Addendum to the Westinghouse Small Break ECCS Evaluation Model Using the NOTRUMP Code: Safety Injection Into the Broken Loop and COSI Condensation Model, WCAP-10054-P-A, Addendum 2, Revision 1, July 1997 (Westinghouse Proprietary)	15.1, 15.3	11/88
64.	P. E. Meyer, NOTRUMP, A Nodal Transient Small Break And General Network Code, WCAP-10079-P-A (Proprietary) and WCAP-10080-A (Non-Proprietary), August 1985	15.1, 15.3	11/88

1.6.2 MISCELLANEOUS TECHNICAL REPORTS

	Title	Section Reference
1.	H. E. Cramer, et al, "Survey of the Elevated Line Source," Technical Report, Contract No. DA-42-007-AMC-120(R), GCA Technical Report No. 66-13-G, GCA Corporation, Bedford, Mass., 61 pp., 1967.	2.3
2.	C. R. Dickson, et al, "Aerodynamic Effects of the EBR-II Containment Vessel Complex on Effluent Concentration," Proc. of the AEC Meteor. Inf. Meeting at Chalk River, Ontario, September 11-14, 1967, pp. 87-104.	2.3

Title	Section Reference
3. J. G. Edinger, "The Influence of Terrain and Thermal Stratification of Flow Across the California Coastline," AFCRL-TR-60-438, Final Report, Contract No. AF(604)-5212, University of California, Los Angeles, 1960	2.3
4. P. A. Leighton, "Geographic Aspects of Air Pollution," Geographic Review, LVI, No. 2, 1966, pp. 153-174.	2.3
5. J. C. Ulberg, "Meteorological Data for Predicting Inhalation Hazards from Space Unit Launch Operations at Point Arguello," USNRDL-TR-1112, U.S. Naval Radiological Defense Laboratory, San Francisco, Ca., 1966, pp. 83.	2.3
6. H. E. Cramer, et al, "Meteorological Prediction Techniques and Data System," Final Report Contract No. DA-42-007-CML-552, U.S. Army, Dugway Proving Ground, Dugway, Utah, 1964.	2.3
7. F. Pasquill, Atmospheric Diffusion, D. Van Nostrand Company, Ltd., London, 1962, pp. 190.	2.3
8. R. A. Dean, "Thermal Contact Conductance Between $U0_2$ and Zircaloy-2," CVNA-127, May, 1962.	4.4
9. A. M. Ross and R. L. Stoute, "Heat Transfer Coefficient Between $U0_2$ and Zircaloy-2," NRCL-1552, June, 1962.	4.4
10. L. S. Tong, "Prediction of Departure from Nucleate Boiling for an Auxiliary Non-Uniform Heat Flux Distribution," Journal of Nuclear Energy, Vol. 21, 1967, pp. 241-248.	4.4
11. J. Weisman, et al, "Experimental Determination of the Departure from Nucleate Boiling in Large Rod Bundles at High Pressure," Chem, Eng., Prog. Sump. Ser., 64, No. 82, 1968, pp. 114-125.	4.4
12. L. S. Tong, et al, "Critical Heat Flux (DNB) in Square and Triangular Array Rod Bundles," JSME, Semi-International Symposium, Paper No. 256, Tokyo, Japan 1967	4.4

Title	Section Reference
13. L. A. Stephan, The Effects of Cladding Material and Heat Treatment on the Response of Waterlogged $U0_2$ Fuel Rods to Power Bursts, IN-ITR-111, January, 1970.	4.4
14. Takasaki Tagami, Interim Report on Safety Assessments and Facilities Establishment Project in Japan for Period Ending June 1965, (No. 1).	6.2
15. N. Ranz and N. Marshall, Chem. Engr., Prog. 48, 3. pp. 141-146 and 48, 4, 1952, pp. 173, 180.	6.2
16. M. A. Styrikovich, et al, Atomnoya Energya, Vol. 17, No. 1, July 1964, pp. 45-49, (Translation in UD 621.039.562.5)	6.2
17. W. F. Pasedag and J. L. Gallagher, "Drop Size Distribution and Spray Effectiveness," Nuclear Technology, 10, 1971, p. 412.	6.2
18. L. F. Parsly, Design Considerations of Reactor Containment Spray Systems, ORNL-TM-2412, Part VII, 1970.	6.2
19. W. D. Fletcher, et al, "Post-LOCA Hydrogen Generation in PWR Containments," Journal of American Nuclear Society, June 1970.	6.2
20. W. D. Fletcher, et al, "Post-LOCA Hydrogen Generation in PWR Containments," Nuclear Technology 10, 1971, pp. 420-427.	6.2
21. H. E. Zittel, Radiation and Thermal Stability of Spray Solutions, ORNL-NSRD Program Bi-Monthly Report for May-June, 1969. ORNL-TM-2663, September 1969.	6.2
22. W. B. Cottrell, ORNL Nuclear Safety Research and Development Program Bi-Monthly Report for July-August 1968, ORNL-TM-2368, November 1968.	6.2
23. W. B. Cottrell, ORNL Nuclear Safety Research and Development Program Bi-Monthly Report for September-October 1968, ORNL-TM-2425, p. 53, January 1969.	6.2
24. H. E. Zittel and T. H. Row, "Radiation and Thermal Stability of Spray Solutions," Nuclear Technology, 10, 1971, pp. 436-443.	6.2

Title	Section Reference
25. A. O. Allen, The Radiation Chemistry of Water and Aqueous Solutions, Princeton, N.J., Van Nostrand, 1961.	6.2
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27. M. G. Zabetakis, Research on the Combustion and Explosion Hazards of Hydrogen-Water Vapor-Air Mixtures, AECU-3327, September 1956.	6.2
28. E. A. J. Eggleton, A Theoretical Examination of Iodine-Water Partition Coefficients, AERE(R)-4887, 1967.	6.2
29. M. E. Meek and B. F. Rider, Summary of Fission Product Yields for U-235, U-238, Pu-239, and Pu-241 at Thermal, Fission Spectrum and 14 Mey Neutron Energies, APED-5398, March 1, 1968.	11.1
30. J. J. DiNunno, et al, Calculation of Distance Factors for Power and Test Reactor Sites, TID-14844, March 23, 1962.	11.1
31. J. O. Blomeke and M. F. Todd, Uranium-235 Fission - Product Production as a Function of Thermal Neutron Flux, Irradiation Time, and Decay Time, ORNL-2127, August 19, 1957.	11.1
32. F. J. Perkins and R. W. King, "Energy Release from the Decay of Fission Products," Nuclear Science and Engineering,1958.	11.1
33. D. F. Toner and J. S. Scott, Fission Product Release from UO_2 Nuclear Safety, Volume 3, No. 2, December 1961.	11.1
34. J. Belle, "Uranium Dioxide: Properties and Nuclear Applications," Naval Reactor, DRD of NRC, 1961.	11.1
35. S. G. Gillespie and W. K. Brunot, "EMERALD NORMAL - A Program for the Calculation of Activity Releases and Doses from Normal Operation of a Pressurized Water Plant," Program Description and User's Manual, Pacific Gas and Electric Company, January 1973. (Provided to AEC in 1973.)	11.1, 11.2, 11.3, 15.1

Title	Section Reference
36. Deleted in Revision 1.	
37. A. B. Sisson, et al, "Evaluation for Removal of Radio-nuclides from PWR Steam Generator Blowdowns," International Water Conference, November 2, 1971.	11.2
38. E. K. Duursma and M. G. Gross, "Marine Sediments and Radioactivity," Radioactivity in the Marine Environment, National Academy of Sciences, 1971.	11.6
39. Statistical Summary of the United States, U.S. Bureau of Census.	11.6
40. Radioactivity in the Marine Environment, National Academy of Sciences, 1971.	11.6
41. W. K. Brunot, EMERALD - A Program for the Calculation of Activity Releases and Potential Doses From a Pressurized Water Reactor Plant, Pacific Gas and Electric Company, October 1971. (Provided to AEC in 1971.)	15.1
42. L. S. Tong, "Prediction of Departure from Nucleate Boiling for an Axially Non-Uniform Heat Flux Distribution," J. Nuclear Energy, 21, 1967, pp. 241-248.	15.2
43. F. W. Dittus and L. M. K. Boelter, University of California (Berkeley), Public Eng., 1930, 2,433.	15.4
44. W. H. Jens and P. A. Lottes, Analysis of Heat Transfer, Burnout, Pressure Drop, and Density Data for High Pressure Water, USAEC Report ANL-4627, 1951.	15.4
45. T. G. Taxelius, ed. Annual Report - Spert Project, October 1968 September 1969. Idaho Nuclear Corporation, IN-1370, June 1970.	15.4
46. R. C. Liimatainen and F. J. Testa, Studies in TREAT of Zircaloy-2-Clad, UO_2 Core Simulated Fuel Elements, ANL-7225, January-June 1966, November, 1966, p. 177.	15.4

Title	Section Reference
47. Regulatory Guide 1.109, Calculation of Annual Doses to Man from Routine Releases of Reactor Effluents for the Purpose of Evaluating Compliance with 10 CFR Part 50, Appendix I, Revision 1, October 1977.	15.5

1.6.3 DRAWINGS INCORPORATED BY REFERENCE

Controlled engineering drawings were removed from the FSAR Update at Revision 15. The drawings are considered incorporated by reference. Table 1.6-1 identifies the controlled engineering drawings that are incorporated by reference and also provides a cross-reference of the controlled engineering drawings to the respective FSAR Update figure number. The contents of the drawings are controlled by DCPP procedures.

DESIGN COMPARISON OF DIABLO CANYON POWER PLANT UNITS 1 & 2,
ZION STATION, AND TROJAN NUCLEAR PLANT
(Historical-valid through March 1984)

Chapter Number	System/Component	Section Reference	Discussion
1	Introduction	1.1	All are 4-loop plants Reactor power ratings (Core thermal output): Diablo Canyon Unit 1: 3338 MWt Diablo Canyon Unit 2 & Trojan: 3411 MWt Zion: 3250 MWt
3	Containment	3.8.2	All containments are steel-lined concrete structures. Diablo Canyon uses conventional reinforcing; Trojan and Zion are posttensioned. Comparative dimensions and designed pressures are: (I.D., ft / Net Vol, cu ft / Design Press, psig) Diablo Canyon: 140 / 2,630,000 / 47 Trojan: 124 / 2,000,000 / 60 Zion: 140 / 2,860,000 / 47
4	Reactor	4.2.1	Generally similar to Zion and Trojan, but differences exist in design based on nuclear and thermal-hydraulic design parameters.
	Reactor Vessel	4.2.2	Diablo Canyon Unit 1: Design of thermal shields and upper and lower Internals support structures, etc., is similar to Zion. Diablo Canyon Unit 2: Design of neutron pads and upper and lower support structures, etc., is similar to Trojan.
	Reactivity Control	4.2.3	Similar to Zion and Trojan.

TABLE 1.3-1

Chapter Number	System/Component	Section Reference	Discussion
4 (Cont'd)	Nuclear Design	4.3	Similar to Zion and Trojan except differences exist in fuel burnup rates, fuel enrichments, k_{eff}, and core kinetics characteristics.
	Thermal-hydraulic Design	4.4	Similar to Zion and Trojan except for rating differences.
5	Reactor Coolant System	5.1, 5.2, 5.3, 5.4, 5.5	All three plants are similar in design except Diablo Canyon will use codes that specifically apply. Zion has loop stop valves.
	Reactor Vessel	5.4	Diablo Canyon Units 1 and 2 each have 50-year design life while Trojan and Zion are designed for 40 years. Diablo Canyon Unit 2, Trojan, and Zion have four of the inner CRDM housings moved into the outer rows as compared to Diablo Canyon Unit 1. All have the same number of CRDM housings. Diablo Canyon Unit 1 uses ASME Code, Section III, 1965 Edition and addenda through winter 1966. Zion uses ASME Code, Section III, 1965 Edition and addenda through summer 1966. Diablo Canyon Unit 2 and Trojan use ASME Code, Section III, 1968 Edition. Diablo Canyon Unit 1 and Zion have blowout collars on bottom tubes only. Diablo Canyon Unit 2 does not have blowout collars. Trojan has blowout collars on control rod drive mechanisms and bottom tubes.
	Reactor Coolant Pumps	5.5.1	Diablo Canyon Units 1 and 2 and Trojan have spacer couplings. Zion does not.
	Steam Generators	5.5.2	Diablo Canyon and Zion have 1,100 psia secondary side design pressure. Trojan uses 1,200 psia.

TABLE 1.3-1

Chapter Number	System/Component	Section Reference	Discussion
5 (Cont'd)	Reactor Coolant Piping	5.5.3	Diablo Canyon Units 1 and 2 and Zion use seamless forged pipe sections and 90° elbows which are cast sections joined by electroslag welds. Trojan uses centrifugally cast pipe sections. Zion's design is modified to accommodate the loop stop valves. Diablo Canyon and Zion are designed to B31.1. Trojan is designed to B31.7.
	Residual Heat Removal System	5.5.6	All are similar in design.
	Pressurizer	5.5.10	Head material is cast for Diablo Canyon Unit 1 and Zion while it is fabricated plate for Diablo Canyon Unit 2 and Trojan. The shell material is fabricated plate for all four units.
6	Containment Spray System	6.2.3	All are similar in design.
	Emergency Core Cooling System	6.3	All are similar in design.
7	Reactor Trip System	7.2	Diablo Canyon Units 1 and 2 and Trojan have solid-state logic protection systems while Zion has relay protection. The logic on trips associated with the reactor coolant pump power supplies is similar at Diablo Canyon Units 1 and 2 and Trojan. Zion is different because it has four separate buses, one for each pump, while Diablo Canyon and Trojan have two reactor coolant pumps per bus.
	Engineered Safety Features Actuation System	7.3	All four units have extended engineered safety features testability. Diablo Canyon Units 1 and 2 and Trojan have 2/3 high containment pressure logic for safety injection initiation, while Zion uses 2/4 high containment pressure.
	Systems Required for Safe Shutdown	7.4	System functions are similar on all four units.

TABLE 1.3-1

Chapter Number	System/Component	Section Reference	Discussion
7 (Cont'd)	Safety-related Display Instrumentation	7.5	Parametric display is similar for all four units. The physical configuration may differ.
	Other Safety Systems	7.6	All four units have residual heat removal isolation valve interlocking and automatic closure devices.
	Control Systems	7.7	Diablo Canyon Units 1 and 2 and Trojan have digital rod position indication systems, while Zion has analog rod position indication.
8	Electric Power	Fig. 8.1-1	Diablo Canyon and Trojan auxiliary systems supply loads at 12 and 4.16 kV. Zion does not have 12-kV loads.
	Standby Power	8.3.2	Trojan has two 4.16-kV ESF buses with one standby diesel generator unit (two engines in line per unit) on each bus. Diablo Canyon and Zion have five standby diesel generators, two for each unit and one that can be transferred to either unit.
9	Chemical and Volume	9.3.4	Similar, except Diablo Units 1 and 2 and Zion have 12 percent boric acid concentration systems, while Trojan has a 4 percent system.
10	Steam and Power Conversion System	10.2	Turbines are similar with three double-flow low pressure elements and six stages of feedwater heating.
11	Radioactive Waste Management	Entire chapter	Systems and treatment provided are similar.

TABLE 1.3-2

MAJOR DESIGN CHANGES SINCE THE PSAR
(Historical-valid through July 1974)

Item	Changes in Design	Original FSAR Section References
1.	Reactor vessel internals changes (Unit 2) - Thermal shield replaced by neutron pads, and lower core support plate and upper internals support system redesigned.	4.2.2
2.	Tetra boron carbide control rod poison material changed to silver-indium-cadmium.	4.2.3
3.	Pellet density, fuel rod pressure, and burnable poison loading pattern have changed to reflect more detailed design calculations and latest operating experience.	4.3
4.	Reactor vessel top and bottom head penetration and control rod drive mechanisms have been redesigned and removable insulation has been provided on the closure head to enable inservice inspection.	4.3, 5.4, 5.4.2, 5.4.4
5.	Safety injection now provides cold leg injection with cold leg or hot leg recirculation.	6.3
6.	Rod withdrawal step from rod drop signal and automatic turbine load cutback initiated by rod drop have been replaced by the power range neutron flux rate trips.	7.2
7.	Relay logic for reactor protection and engineered safety features actuation system has been changed to solid-state logic.	7.2, 7.3
8.	On-line testing has been provided for engineered safety features actuation system.	7.3
9.	Analog rod position indication has been replaced by digital rod position indication.	7.7.1
10.	Deleted in Revision 15	
11.	Design criteria changes in the event of inleakage of contaminated water into component cooling water (CCW) system. This tends to minimize possible release of reactor coolant outside containment via the CCW system.	9.2.2

TABLE 1.3-2

Item	Changes in Design	Original FSAR Section References
12.	The compressed air system is revised to eliminate the emergency air system. This satisfies safety criteria while reducing potential leakage paths from containment.	9.3.1
13.	The stainless steel liner in the spent fuel pool and transfer canal is changed from Design Class I to Design Class II. The liners prevent minor leakage, and the pool and canal structures remain Design Class I and are relied upon to prevent major failure.	9.1.2
14.	The primary water storage tank is reclassified from Design Class I to Design Class II. This is no longer the primary source of makeup water to the CCW System. A backup source to a makeup supply to a Design Class I system itself need not be classified Design Class I.	9.2.4
15.	The detectors for the fire detection alarm system are relocated to give more specific identification of the location of the source. Instrument ac power is provided.	9.5.1
16.	In the containment structure, vertical joints are provided with a shear key, as required by ACI 301-66.	3.8.2
17.	The Chief Mechanical Engineer is no longer the designated Project Engineer. This change was made April 26, 1971.	17.0
18.	Those parts of the fire protection system that protect Design Class II and III equipment and structures are not required to be Design Class I.	9.5.1
19.	Inspection procedures for cable were modified to require tests on sample reels from each production run, rather than on each reel.	8.5.2
20.	The fuel assembly array is revised from 15 x 15 to 17 x 17. The change was initiated to maintain sufficient flexibility to fulfill the requirements of, or any changes to, 10 CFR 50.46: "Acceptance Criteria for Emergency Core Cooling Systems for Light-Water Nuclear Power Reactors."	4 and 15
21.	The steam generator blowdown treatment system is added to provide for treatment in the event that there is primary-to-secondary leakage.	11.2.2

Item	Changes in Design	Original FSAR Section References
22.	An alarm system is added to indicate the rupture of a auxiliary seawater cooling header. The function of the alarm is to alert the operator in the event that the header breaks, so that the operator can route the cooling water to the redundant supply header and restore the cooling function to the CCW system.	9.2
23.	The turbine building is protected from floods due to pipe breaks by the addition of an 18-inch overflow drain from the sump system to the circulating water discharge canal.	9.2
24	Protection against flooding of the turbine building is provided by design changes to decrease the probability of rupturing the expansion joints at the water box. Also, the consequences of a rupture are minimized by the addition of expansion joint sleeves.	10.4
25.	A system is added and designed to detect and alarm in the event that there are loose parts in the reactor coolant system.	3.9
26.	A Design Class I supply of demineralized water is provided for makeup to the spent fuel pool.	9.1
27.	The ventilation system for the fuel handling area is modified and reclassified to Design Class I.	9.4
28.	A Design Class I containment hydrogen purge system is provided for reducing the containment atmosphere hydrogen concentration in the event of a LOCA. The system has redundant sets of supply and exhaust fans and filters. Each fan is on a separate vital bus.	6.2
29.	The heating, ventilating, and air conditioning system for the control room is modified and is reclassified as Design Class I. The system has four modes of operation designed to make the control room habitable: (a) during normal operation,(b) during long-term occupancy, (c) in the event that there is excessive airborne activity external to the control room, and (d) in the event that there is a fire in the control room.	9.4

TABLE 1.3-2

Item	Changes in Design	Original FSAR Section References
30.	Design changes add the capability of maintaining reactor coolant system temperature during hot shutdown operations; when the reactor is subcritical, the steam dumps to the main condenser. This is accomplished by a controller in the steam line, operating in the pressure control mode, which is set to maintain the steam generator steam pressure.	5.1
31.	Meteorological monitoring equipment will provide data to be recorded in the control room during plant operation.	16.4

TABLE 1.4-1

PRINCIPAL CONSULTANTS AND CONTRACT DESCRIPTION
(Historical-valid through March 1986)

Principal Consultant	Contract (Over $100,000)
Anco Engineers, Inc.	Raceway system qualification; seismic testing of mechanical equipment
Arremany & Associates	Nondestructive examination services
Associated Technical Training Services	Operator training
Babcock & Wilcox, Inc.	Engineering support
Bechtel Power Corporation	Project management, engineering, construction procurement, startup, project cost and scheduling, quality assurance
J. R. Benjamin and Associates, Inc.	Seismic verification of auxiliary and turbine buildings; frequency of vessel impact on intake structure
William K. Brunot	Reliability and risk analysis
Burns & Roe, Inc.	Engineering quality control services
California Department of Fish and Game	Marine biology studies
California Polytechnic State University	Marine biology studies
Chemrad Corporation	Radiological and health physics support
Cygna Energy Services, Inc.	Piping support and HVAC equipment qualification
Earth Sciences Associates	Geological investigations
Ecological Analysts, Inc.	Marine biology studies

Principal Consultant	Contract (Over $100,000)
EDS Nuclear, Inc.	Control room (HVAC) design, technical support center pressurization system design, piping anchors design review, radiation shielding design review, emergency core cooling system nozzle fatigue analysis
Energy Training Corporation	Operator training
Geri Engineering, Inc.	Reactor vessel inservice inspection tool modification
Harding - Lawson Associates	Soil investigations, geophysical surveys
Hydro - Research Science	Discharge structure model study
Innova Corporation	Pipe support design review and redesign engineering
James Engineering Company	Engineering support
Kaiser Engineers, Inc.	Program management and engineering services, independent assessment of alternative cooling water systems
Lambert & Company	Nondestructive examination services
Nuclear Services Corporation	Pipe break analysis
Nucon, Inc.	Engineering support for fuel load and startup
NUS Corporation	Engineering studies of spent fuel pool storage expansion
Nutech Engineers, Inc.	Seismic and environmental qualification engineering support
Offshore Technology Corporation	Intake structure hydraulic model studies
Omar J. Lillevang	Breakwater design, breakwater damage study

Principal Consultant	Contract (Over $100,000)
Pickard, Lowe, & Garrick, Inc.	Probabilistic risk assessment
Project Assistance Corporation	Quality assurance program support
Radiation Research Associates, Inc.	Shielding design review
Regents of the University of California	Thermal physical modeling studies
R. F. Reedy and Associates	Quality assurance verification
Robert L. Cloud Associates, Inc.	Hosgri seismic reverification program
Stone & Webster Engineering Corporation	Independent design verification program - Phase II
TERA Corporation	Source modeling studies, general engineering, thermal discharge assessment, assessment of alternative cooling water systems
Terra Technology Corporation	Seismic recording system maintenance and records processing
Teledyne Engineering Services	Independent design verification program
URS/John A. Blume and Associates	Seismic structural criteria, electrical seismic testing criteria, seismic research program, seismic review and reverification, independent internal review, Hosgri seismic evaluation
Waltek Services	Technical support services
Westinghouse Electric Corporation	Long-term seismic research program, environmental qualification, piping redesign and qualification, seismic reverification technical support
Woodward - Clyde and Associates	Seismic surveys
Wyle Laboratories	Seismic test and engineering support

TABLE 1.4-2

SUPPLIERS OF IMPORTANT EQUIPMENT AND MATERIALS
OTHER THAN NUCLEAR STEAM SUPPLY SYSTEM
(Historical-valid through March 1986)

Supplier	Equipment/Materials
Alco Engines Division, White Industrial Power, Inc.	Diesel generator units
American Bridge Co., Division of U.S. Steel Corp.	Furnish structural steel
AMF Cuno Division	Radioactive waste filters
Armco Steel Corp.	Containment wall penetration flued heads
Babcock & Wilcox	Safety parameter display system
Berkeley Steel Construction Co.	Radioactive waste tanks
Bingham-Willamette Pump Co.	Component cooling water pumps, auxiliary saltwater pumps
Byron Jackson Pump, Division of Borg-Warner Corp.	Auxiliary feedwater pumps
Capital Westward Inc.	Radioactive waste tanks
Chem-Nuclear Services Inc.	Radioactive resin removal and transfer system
Chemetron Corp.	Carbon dioxide
Combustion Engineering	Subcooled margin monitor
Contromatics Corp.	Radioactive waste valves
De Laval Turbine Inc.	Diesel fuel transfer pumps
Dresser Industries Inc.	Steam generator safety valves
Fairbanks-Morse Co.	Fire pumps

TABLE 1.4-2

Supplier	Equipment/Materials
Fenwall Inc.	Halon 1301 system
Fisher Controls Co.	Class I pressure/level controllers, and control valves
Fulton Shipyard	Fuel handling area crane
General Electric Co.	Electrical penetrations of containment structure, 12 and 4.16-kV switchgear
Grinnell Co.	Radioactive waste valves
Harnischfeger P&H	Turbine building bridge cranes
Ingersoll-Rand Co.	Radioactive waste pumps, reactor coolant drain tank pump, makeup water system pumps
J. E. Lonergan Co.	Class I safety relief valves
Mine Safety Appliances Co.	Radwaste gas analyzers
Murphy Pacific Corp.	Furnish structural steel
M. W. Kellogg Co. (Pullman Power Products)	Main systems piping
National Controls, Inc.	Condensed waste drumming system
Paul Monroe Hydraulics	Dome service crane snubbers
Pyrotronics, Inc.	Fire detection and alarm system
Quick Manufacturing Co.	Dome service crane
Schutte and Koerting Co.	Main steam isolation and check valves
Scott Company of California	Ventilation and air filters, fire water and stand pipe system

TABLE 1.4-2

Supplier	Equipment/Materials
Fenwall Inc.	Halon 1301 system
Velan Valve Corp.	Radioactive waste valves
Viking Automatic Sprinkler Co., Chemtron Corp. (Subcontractor)	Water spray and CO_2 fire protection systems
Westinghouse Electric Corp.	Radioactive waste evaporators
Yuba Manufacturing Co.	Component cooling water heat exchanger, polar crane

TABLE 1.4-3

CONSTRUCTION AND INSTALLATION CONTRACTORS
(Historical-valid through March 1986)

Contractor	Units 1 and 2 In-progress Contracts Over $100,000
Bechtel Construction Inc.	Plant maintenance
Plant Asbestos - Thorpe Insulation	Furnish and install insulation
Promatec	Furnish and install insulation
Pullman Power Products	Erect plant and steam piping
Westinghouse Electric Corporation	Erect turbine-generator
Contractor	Units 1 and 2 Completed Contracts Over $100,000
A. J. Diani Construction	Paving of access roads
American Bridge	Complete structural steel
Ames Associates	Repair breakwater
Arrowhead Industrial Water	Furnish demineralizer
Bigge Crane & Rigging Co.	Heavy equipment handling
Bigge Drayage Co.	Material receiving/storage (Pismo Beach laydown and storage yard)
Bostrom-Bergen	Structural steel
Bovee & Crail Construction	Erect intake equipment & pipe
Chemtrol Corporation	Furnish and install penetration seals
Continental Heller Corp.	Construct security building
E.H. Haskell Company	Parking lot

Contractor	Units 1 and 2 In-progress Contracts Over $100,000
Endurance Metal Product Co.	Erect miscellaneous steel
Granite Constr. Co. and Gordon H. Ball, Inc.	Construct breakwaters
Guy F. Atkinson Company	Construction of seismic modifications, construct buildings
Healy - Tibbits	Repair breakwater
H&H Construction, Inc.	Grading and paving (Pismo Beach laydown and storage yard)
H. H. Robertson Co.	Install siding and roofing
H.P. Foley Company	Install wiring, electrical equipment, and instrumentation. Construct seismic modifications of buildings.
Morgan Equipment Co.	Batch plant
Murphy Pacific Corp.	Furnish and install structural steel erect turbine building cranes
Pinkerton's Inc.	Security guard service
Pittsburg-Des Moines Steel Co.	Fabricate auxiliary building tanks, construct storage tanks
Pullman Power Products	Construct pipe rupture restraints
Relocate Structures, Inc.	Construct administration building
Robert McMullan & Son, Inc.	Finish painting
Sanchez & Son, Inc.	Grading and paving (Pismo Beach laydown and storage yard)
San Luis Garbage	Disposal of waste water

TABLE 1.4-3

Contractor	Units 1 and 2 In-progress Contracts Over $100,000
Scott Company of California	Furnish and install air conditioning and ventilation
System Steel Builders	Construction offices, warehouse buildings (Pismo Beach laydown and storage yard)
Tech-Sil, Inc.	Furnish and install penetration seals
Valley Trucking of Santa Maria, Inc.	Disposal of waste water
Viking Automatic Sprinkler Co.	Furnish and install fire protection
Walter Bros. Construction Co. and Milburn & Sansome Co.	Plant roads and site preparation, construct Harford Drive-County Road construct access road
Wismer & Becker Contracting Engineers	Install electrical equipment switchyard, install mechanical equipment/erect NSSS

TABLE 1.5-1

RESEARCH AND DEVELOPMENT PROGRAMS
(Historical-valid through November 1975)

PROGRAM	TECHNICAL REPORTS
Fuel development for operations at high power densities Full-length emergency core cooling heat transfer tests (FLECHT) Reactor vessel thermal shock Verification test (17 x 17) Delayed departure from nucleate boiling (DDNB)	F. T. Eggleston, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Winter 77 - Summer 78, WCAP-8768, Rev. 2, October 1978.
Core stability evaluation Blowdown forces program	C. J. Kubit, Safety-Related Research and Blowdown Development for Westinghouse PWRs, Program Summaries. Fall 1972, WCAP-8004, January 1973.
Containment spray Fuel rod burst Loss-of-coolant analysis ESADA DNB	R. M. Hunt, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Fall 1970, WCAP-7614-L, November 1970.
Burnable poison Flashing heat transfer	R. M. Hunt, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Spring 1970, WCAP-7943-L, May 1970.
Fuel development for operation at high power densities	M. D. Davis, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Fall 1974, WCAP-8483, March 1975.

TABLE 1.5-1

PROGRAM	TECHNICAL REPORTS
Incore detector Gross failed fuel detector	C. J. Kubit, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Spring 1974, WCAP-8385, July 1974.
Environmental testing of engineered safety features related equipment	C. J. Kubit, Safety-Related Research and Development for Westinghouse PWRs, Program Summaries. Spring 1972, WCAP-7856, May 1972.
In-pile fuel densification	J. M. Hellman, Fuel Densification Experimental Results and Model for Reactor Operation, WCAP-8218-P-A March 1975 (Proprietary) and WCAP-8219-A, March, 1975 (Non-proprietary).
Verification test (17 x 17)	L. Geninski, et al, Safety Analysis of the 17 x 17 Fuel Assembly for Combined Seismic and Loss-of-Coolant Accident, WCAP-8288, December 1973.
Verification test (17 x 17)	E. E. DeMario and S. Makazato, Hydraulic Flow Test of the 17 x 17 Fuel Assembly, WCAP-8279, February 1974.
Verification test (17 x 17)	F. W. Cooper, Jr., 17 x 17 Drive Line Components Test - Phase IB, II, II, - Drop and Deflections, WCAP-8446, December 1974.

TABLE 1.5-1

PROGRAM	TECHNICAL REPORTS
Departure from nucleate boiling (DNB)	K. W. Hill, et al, Effect of 17 x 17 Fuel Assembler Geometry on DNB, WCAP-8396-P-A, February 1975 (Proprietary) and WCAP-8297-A, February 1975.
Incore flow mixing	F. W. Motley, et al, The Effect of 17 x 17 Fuel Assembly Geometry on Interchannel Thermal Mixing, (WCAP-8298-P-A) (Proprietary) and WCAP-8299-, January 1975.
LOCA heat transfer tests	A. J. Burnett and S. D. Kopelic, Westinghouse ECCS Evaluation Model October 1975 Version, WCAP-8622 (Proprietary) and WCAP-8623 (Non-Proprietary), November 1975.
17 x 17 fuel surveillance program	Irradiation of 17 x 17 Demonstration Assemblies in Surry Units No. 1 and 2, Cycle 2, WCAP-8262, July 1974.

TABLE 1.6-1

CONTROLLED ENGINEERING DRAWINGS/FSAR UPDATE FIGURES
CROSS REFERENCE

Figure	Sheet	Drawing	Description
1.2-3		500001-1	Area Location Plan
1.2-4		57727-1	Auxiliary, Containment, and Fuel Handling Buildings (Units 1 & 2), Plan at Elevation 140 ft
1.2-5		57726-1	Auxiliary, Containment, and Fuel Handling Buildings (Units 1 & 2), Plan at Elevation 115 ft
1.2-6		57725-1	Auxiliary, Containment, and Fuel Handling Buildings (Units 1 & 2), Plan at Elevations 91 and 100 ft
1.2-7		57724-1	Auxiliary and Containment Buildings (Units 1 & 2), Plan at Elevation 85 ft
1.2-8		57723-1	Auxiliary and Containment Buildings (Units 1 & 2), Plan at Elevation 73 ft
1.2-9		57722-1	Auxiliary and Containment Buildings (Unit 1 & 2), Plan at Elevations 60 and 64 ft
1.2-10		500977-1	Containment Building (Unit 2), Plan at Elevations 115 and 140 ft
1.2-11		500971-1	Containment & Fuel Handling Buildings (Unit 2), Plan at Elevations 85, 91, and 100 ft
1.2-12		500968-1	Containment Building (Unit 2), Plan at Elevations 60, 64, and 73 ft
1.2-13		57721-1	Turbine Building (Unit 1), Plan at Elevation 140 ft
1.2-14		57720-1	Turbine Building (Unit 1), Plan at Elevation 119 ft
1.2-15		57719-1	Turbine Building (Unit 1), Plan at Elevation 104 ft
1.2-16		57718-1	Turbine Building (Unit 1), Plan at Elevation 85 ft
1.2-17		500967-1	Turbine Building (Unit 2), Plan at Elevation 140 ft
1.2-18		500966-1	Turbine Building (Unit 2), Plan at Elevation 119 ft
1.2-19		500965-1	Turbine Building (Unit 2), Plan at Elevation 104 ft
1.2-20		500964-1	Turbine Building (Unit 2), Plan at Elevation 85 ft
1.2-21		57728-1	Auxiliary Building (Units 1 & 2), Section A-A
1.2-22		57729-1	Auxiliary and Containment Buildings (Unit 1 & 2), Section B-B
1.2-23		57730-1	Auxiliary and Fuel Handling Buildings (Unit 1 & 2), Section C-C

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Figure	Sheet	Drawing	Description
1.2-24		57731-1	Containment, Turbine, and Fuel Handling Buildings (Unit 1) Section D-D
1.2-25		57732-1	Auxiliary, Turbine, and Fuel Handling Buildings (Unit 1), Section E-E
1.2-26		57733-1	Auxiliary, Fuel Handling, and Turbine Buildings (Units 1 & 2), Section F-F
1.2-27		57734-1	Turbine Building (Unit 1), Section G-G
1.2-28		500969-1	Containment Building (Unit 2), Section A-A
1.2-29		500972-1	Vent & Fuel Handling Buildings (Unit 2), Section B-B
1.2-30		500973-1	Turbine, Containment, & Fuel Handling Buildings (Unit 2), Section C-C
1.2-31		500974-1	Turbine Building (Unit 2), Sections D-D and E-E
1.2-32		500976-1	Turbine Building (Unit 2), Section F-F
2.4-7		468999-1	Typical Sections for Tribar Armor Construction
2.4-8		469001-1	Restored Cross-sections and Embedment Plan
3.2-1	1 of 4	102001-3	Piping Schematic Legend
3.2-1	1A of 4	102023-2	Piping Schematic Legend
3.2-1	2 of 4	108001-3	Piping Schematic Legend
3.2-1	2A of 4	108023-2	Piping Schematic Legend
3.2-2	1 of 23	102002-4	Piping Schematic - Condensate System (23 Sheets)
3.2-2	2 of 23	108002-4	
3.2-2	3 of 23	102002-5	
3.2-2	3A of 23	102002-5A	
3.2-2	4 of 23	108002-5	
3.2-2	5 of 23	102002-6	
3.2-2	5A of 23	102002-6A	
3.2-2	6 of 23	108002-6	
3.2-2	7 of 23	102002-7	
3.2-2	8 of 23	108002-7	
3.2-2	9 of 23	102002-8	
3.2-2	9A of 23	102002-8A	
3.2-2	10 of 23	108002-8	
3.2-2	10A of 23	108002-8A	
3.2-2	11 of 23	102002-9	
3.2-2	11A of 23	102002-9A	

Figure	Sheet	Drawing	Description
3.2-2	11B of 23	102002-9B	
3.2-2	12 of 23	108002-9	
3.2-2	12A of 23	108002-9A	
3.2-2	13 of 23	102002-10	
3.2-2	14 of 23	108002-10	
3.2-2	15 of 23	102002-11	
3.2-2	16 of 23	108002-11	
3.2-3	1 of 11	102003-3	Piping Schematic - Feedwater System (11 Sheets)
3.2-3	1A of 11	102003-3A	
3.2-3	2 of 11	108003-3	
3.2-3	3 of 11	102003-4	
3.2-3	4 of 11	108003-4	
3.2-3	5 of 11	102003-4A	
3.2-3	6 of 11	108003-4A	
3.2-3	7 of 11	102003-5	
3.2-3	8 of 11	108003-5	
3.2-3	9 of 11	102003-6	
3.2-3	10 of 11	108003-6	
3.2-4	1 of 16	102004-3	Piping Schematic - Turbine Steam Supply System (16 Sheets)
3.2-4	2 of 16	108004-3	
3.2-4	3 of 16	102004-4	
3.2-4	4 of 16	108004-4	
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3.2-4	12 of 16	108004-8	
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Figure	Sheet	Drawing	Description
3.2-5	1 of 12	102005-2	Piping Schematic - Extraction Steam and Heater Drip System (12 Sheets)
3.2-5	2 of 12	108005-2	
3.2-5	3 of 12	102005-3	
3.2-5	4 of 12	108005-3	
3.2-5	5 of 12	102005-4	
3.2-5	6 of 12	108005-4	
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3.2-5	8 of 12	108005-5	
3.2-5	9 of 12	102005-6	
3.2-5	10 of 12	108005-6	
3.2-5	11 of 12	102005-7	
3.2-5	12 of 12	108005-7	
3.2-6	1 of 10	102006-3	Piping Schematic - Auxiliary Steam System (10 Sheets)
3.2-6	1A of 10	102006-3A	
3.2-6	2A of 10	102006-3B	
3.2-6	2 of 10	108006-3	
3.2-6	3 of 10	102006-4	
3.2-6	4 of 10	108006-4	
3.2-6	5 of 10	102006-4A	
3.2-6	6 of 10	108006-4A	
3.2-6	7 of 10	102006-5	
3.2-6	8 of 10	108006-5	
3.2-7	1 of 9	102007-3	Piping Schematic - Reactor Coolant System (9 Sheets)
3.2-7	2 of 9	108007-3	
3.2-7	3 of 9	102007-4	
3.2-7	4 of 9	108007-4	
3.2-7	5 of 9	102007-5	
3.2-7	6 of 9	108007-5	
3.2-7	7 of 9	102007-5A	
3.2-7	7A of 9	102007-5B	
3.2-7	8 of 9	108007-5A	
3.2-8	1 of 34	102008-3	Piping Schematic - Chemical and Volume Control System (34 Sheets)

Figure	Sheet	Drawing	Description
3.2-8	1A of 34	102008-3A	
3.2-8	1B of 34	102008-3B	
3.2-8	1C of 34	102008-3C	
3.2-8	2 of 34	108008-3	
3.2-8	3 of 34	102008-4A	
3.2-8	4 of 34	108008-4A	
3.2-8	5 of 34	102008-4	
3.2-8	6 of 34	108008-4	
3.2-8	7 of 34	102008-4B	
3.2-8	7A of 34	102008-4C	
3.2-8	7B of 34	102008-4D	
3.2-8	7C of 34	102008-4E	
3.2-8	8 of 34	108008-4B	
3.2-8	8A of 34	108008-4C	
3.2-8	9 of 34	102008-5	
3.2-8	10 of 34	108008-5	
3.2-8	11 of 34	102008-5A	
3.2-8	12 of 34	108008-5A	
3.2-8	13 of 34	102008-5B	
3.2-8	14 of 34	108008-5B	
3.2-8	15 of 34	102008-5C	
3.2-8	16 of 34	108008-5C	
3.2-8	16A of 34	102008-5D	
3.2-8	17 of 34	102008-6	
3.2-8	17A of 34	102008-6A	
3.2-8	18 of 34	108008-6	
3.2-8	19 of 34	102008-7	
3.2-8	20 of 34	108008-7	
3.2-8	21 of 34	102008-8	
3.2-8	21A of 34	102008-8A	
3.2-8	22 of 34	108008-8	
3.2-8	23 of 34	102008-9	
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3.2-9	1 of 12	102009-3	Piping Schematic - Safety Injection System (12 Sheets)
3.2-9	2 of 12	108009-3	

Figure	Sheet	Drawing	Description
3.2-9	3 of 12	102009-4	
3.2-9	4 of 12	108009-4	
3.2-9	5 of 12	102009-5	
3.2-9	5A of 12	102009-5A	
3.2-9	6 of 12	108009-5	
3.2-9	6A of 12	108009-5A	
3.2-9	7 of 12	102009-6	
3.2-9	8 of 12	108009-6	
3.2-9	9 of 12	102009-7	
3.2-9	10 of 12	108009-7	
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3.6-2	1 of 2	438288-1	Containment Structure Pipe Rupture Restraints (2 sheets)
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3.6-5		515939-1	HELB Compartment Pressurization Study El. 85 ft Turbine Building
3.6-6		515940-1	HELB Compartment Pressurization Study El. 104 ft Turbine Building
3.6-7		515941-1	HELB Compartment Pressurization Study El. 119 ft Turbine Building
3.6-8		515942-1	HELB Compartment Pressurization Study El. 140 ft Turbine Building
3.6-9		515943-1	HELB Compartment Pressurization Study El. 60 ft Auxiliary and Containment Building
3.6-10		515944-1	HELB Compartment Pressurization Study El. 73 ft Auxiliary and Containment Building
3.6-11		515945-1	HELB Compartment Pressurization Study El. 85 ft Auxiliary and Containment Building
3.6-12		515946-1	HELB Compartment Pressurization Study El. 91 and 100 ft Auxiliary, Containment and Fuel Handling Bldg
3.6-13		515947-1	HELB Compartment Pressurization Study El. 115 ft Auxiliary, Containment and Fuel Handling Bldg
3.6-14		515948-1	HELB Compartment Pressurization Study El. 140 ft Auxiliary, Containment and Fuel Handling Bldg
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3.8-23		438267-1	Containment Structure, Polar Crane
3.8-45		438431-1	Auxiliary Building, Concrete Outline - Plan at El. 100 ft Areas H and K
3.8-46	1 of 2	438432-1	Auxiliary Building, Concrete Outline - Plan at El. 115 ft Areas J, GE, and GW (2 sheets)
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3.8-47	3 of 3	443205-1	
3.8-48		438445-1	Auxiliary Building, Concrete Outline - Section Areas J and K
3.8-49		438441-1	Auxiliary Building, Concrete Outline - Section Areas H and K
3.8-50	1 of 2	438457-1	Auxiliary Building, Concrete Reinforcing - Plan at El. 115 ft - Areas J, GE, and GW (2 sheets)
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3.8-51		438458-1	Auxiliary Building, Concrete Reinforcing - Plan at El. 115 ft - Areas H and K
3.8-52		443216-1	Auxiliary Building, Concrete Reinforcing - Plans at El. 115 and 140 ft - Area L
3.8-53		443460-1	Auxiliary Building, Concrete Reinforcing - Miscellaneous Sections - Area K
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3.8-59		439509-1	Auxiliary Building, Refueling Areas Overhead Crane
3.8-65	1 of 2	438034-1	Design Class I Tanks Concrete Foundations (2 Sheets)
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3.8-72	1 of 2	59459-1	Concrete Outline Plan – Top Deck Area 1 Intake Structure
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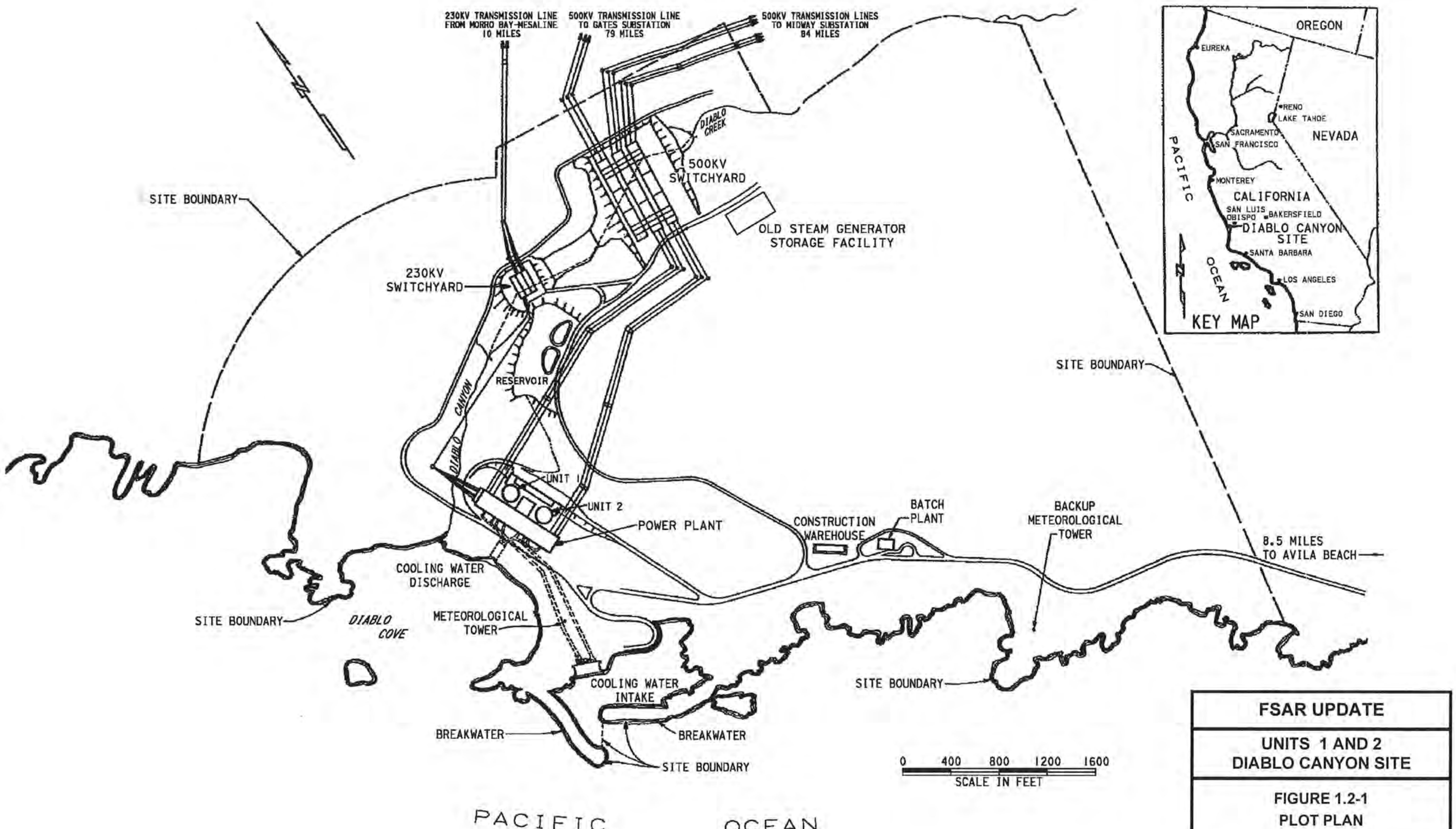
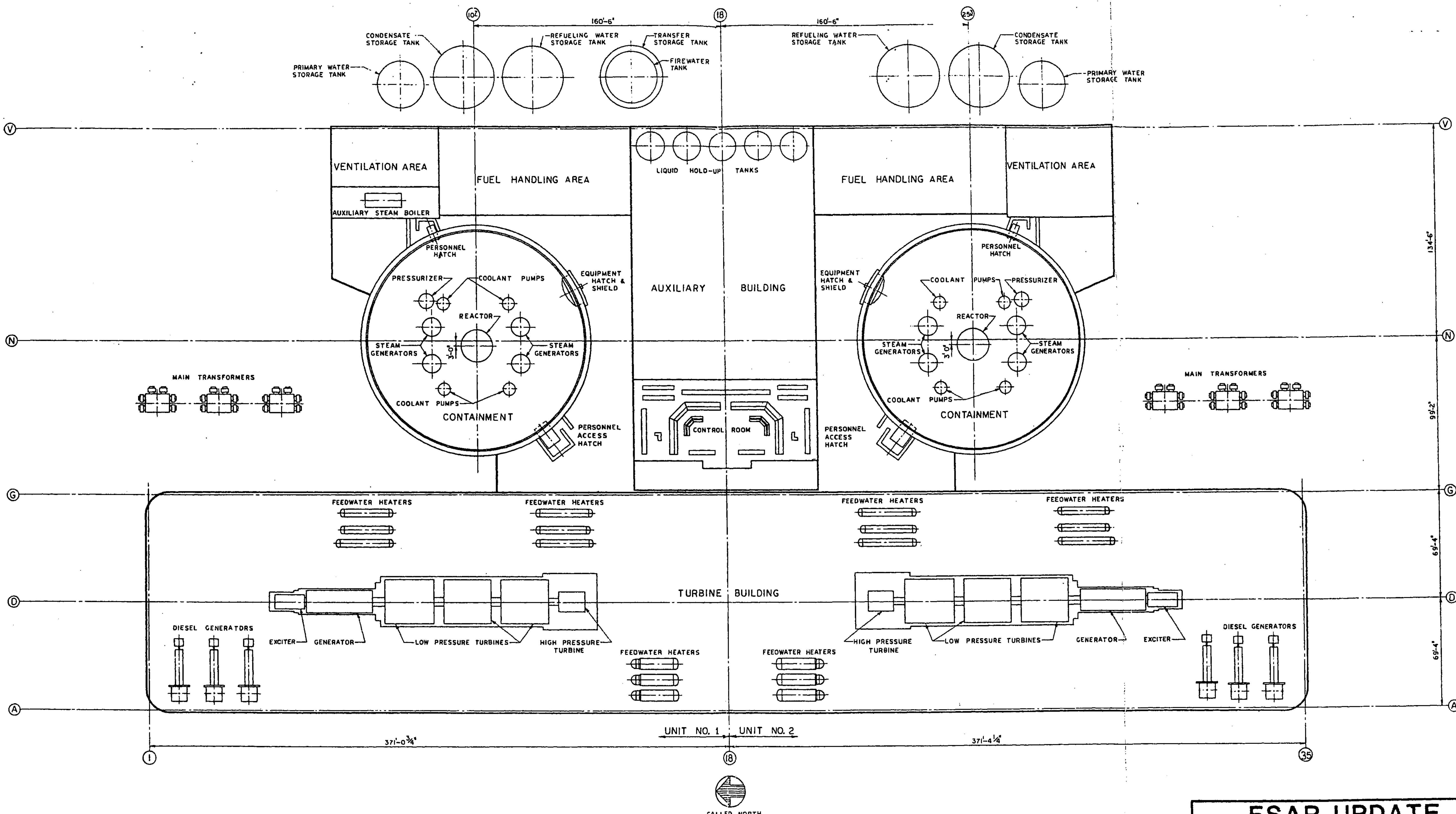


FSAR UPDATE

UNITS 1 AND 2 DIABLO CANYON SITE

FIGURE 1.2-1 PLOT PLAN

Revision 18 October 2008



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 1.2-2
PLANT LAYOUT

Revision 11A April 1997

Chapter 2

SITE CHARACTERISTICS

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NOTE:

[a] This figure corresponds to a controlled engineering drawing that is incorporated by reference into the FSAR Update. See Table 1.6-1 for the correlation between the FSAR Update figure number and the corresponding controlled engineering drawing number.

Chapter 2

SITE CHARACTERISTICS

This chapter describes the DCPP site and vicinity as they existed when the facility was licensed. In the past some changes to site characteristics have been incorporated into this chapter and parts of this chapter reflect this more recent information. Details of the current site area may not be completely consistent with the historic descriptions. Accurate and current site characteristics germane to the licensing bases are contained in the Emergency Plan, Annual Radiological Environmental Operating Report, and the Annual Radioactive Effluent Release Report.

HISTORICAL INFORMATION BELOW IS SHOWN IN ITALICS

This chapter provides information on the geological, seismological, hydrological, and meteorological characteristics of the Diablo Canyon Power Plant (DCPP) site and vicinity. Population distribution, land use, and site activities and controls are also discussed. This information, used in conjunction with the detailed technical discussions provided in other chapters, shows the adequacy of the site for the safe operation of nuclear power units.

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 DESIGN BASES

2.1.1.1 10 CFR Part 100 – Reactor Site Criteria

DCPP is committed to following the guidance set by the standard definition of exclusion area, low population zone (LPZ) and population center distance.

2.1.2 SAFETY EVALUATION

2.1.2.1 10 CFR Part 100 – Reactor Site Criteria

The DCPP commitment to exclusion area, low population zone and population center distance is described in the following sections.

HISTORICAL INFORMATION BELOW IS SHOWN IN ITALICS

2.1.2.1.1 Site Location

The DCPP site is adjacent to the Pacific Ocean in San Luis Obispo County, California, and is approximately 12 miles west-southwest of the city of San Luis Obispo, the county seat. The reactor for Unit 1 is located at latitude 35°12'44" N and longitude 120°51'14" W. The Universal Transverse Mercator (UTM) coordinates for zone 10 are 695,350 meters E and 3,898,450 meters N. The reactor for Unit 2 is located at latitude

35°12'41" N and longitude 120°51'13" W. The UTM coordinates are 695,380 meters E and 3,898,400 meters N. Figure 2.1-1 locates the site on a map of western San Luis Obispo County.

2.1.2.1.2 Site Description

The site boundary and the location of principal structures are shown in Figure 2.1-2. A portion of the site is bounded by the Pacific Ocean.

The DCPP site consists of approximately 750 acres of land located near the mouth of Diablo Creek. 165 acres of the DCPP site are located north of Diablo Creek; this acreage is owned by PG&E. The remaining 585 acres are located adjacent to and south of Diablo Creek. It was purchased in 1995 by Eureka Energy Company (Eureka), a wholly owned subsidiary of PG&E.

All coastal properties located north of Diablo Creek, extending north to the southerly boundary of Montana de Oro State Park and reaching inland approximately 1.5 mile has been owned by PG&E since 1988. Coastal properties located south of Diablo Creek and also reaching inland approximately 1.5 mile has been owned by Eureka since 1995. Prior to 1995, PG&E leased the property from the owner, Luigi Marre Land and Cattle Company. In 1988, PG&E purchased approximately 4500 acres located north of the DCPP site. This section of land consists of approximately 5 miles of coastline and reaches inland approximately 1.5 mile. Except for the DCPP site, the approximately 4500 acres are encumbered by a grazing lease that expires in the year 2000.

There are no plans for development of the property, most of which is within the area subject to the California Coastal Act of 1976. Any development plans would be subject to approval by a discretionary land use permitting process. In 1988 the San Luis Obispo County Planning Department was given authority by the California Coastal Commission to interpret the Act and incorporate it into the County of San Luis Obispo's General Plan, which included the right to issue coastal land use permits. Because it is a discretionary permitting process, the County of San Luis Obispo has the authority to require development projects to be approved by the California Coastal Commission rather than obtaining final approval by the County of San Luis Obispo, Board of Supervisors.

In addition, portions of the coastal property have been listed in the National Register of Historic Places pursuant to the "National Historic Preservation Act of 1966" as a place of historic significance due to the presence of numerous Native American remains and scientific data potential.

2.1.2.1.3 Exclusion Area Control

PG&E has complete authority to determine all activities within the site boundary and this authority extends to the mean high water line along the ocean. On land, the site boundary, the boundary of the exclusion area (as defined in 10 CFR 100), and the *boundary of the unrestricted area (as defined in 10 CFR 20) are shown in Figure 2.1-2. Minimum distances from potential release points for radioactive materials to the unrestricted area boundary and to the mean high water line are also shown in Figure 2.1-2.*

The definition of unrestricted area has been expanded over that in 10 CFR 20.1003. The unrestricted area boundary may coincide with the exclusion (fenced) area boundary, as defined in 10 CFR 100.3, but the unrestricted area does not include areas over water bodies. The concept of unrestricted areas, established at or beyond the site boundary, is utilized in the Technical Specifications limiting conditions for operation to keep levels of radioactive materials in liquid and gaseous effluents as low as is reasonably achievable, pursuant to 10 CFR 50.36a.

On land, there are no activities unrelated to plant operation within the exclusion area; it is not traversed by public highway or railroad. Normal access to the site is from the south by private road (PG&E road easement) that is fenced and posted by PG&E.

PG&E has the right, within the DCPP site, to use excavated materials during the construction of the plant (considering that PG&E obtains all permitting required by regulatory agencies prior to excavation). It is unclear legally if the owner retains all mineral rights. Whatever mineral rights an owner may retain, the owner cannot exercise any such rights in a manner that would interfere with PG&E's rights. Any proposed mining operation (including but not limited to excavation, drilling, and blasting) that would be conducted close enough to the plant to threaten the structural integrity of its foundations will be carefully reviewed and PG&E will take whatever steps it deems necessary to ensure that: (a) the health and safety of the public is not jeopardized, and (b) the operation of the plant is not disrupted. Any entry by the lessee onto the land is subject to PG&E's safety rules and regulations, as is the right to restrict the use of buildings and other structures, and to exclude persons therefrom to the extent necessary to comply with nuclear reactor site criteria.

The mineral rights within the 165 acre PG&E portion of the DCPP site are owned by PG&E, but there is no information suggesting that the land contains any commercially valuable minerals other than for use as borrow materials.

The offshore area (below the mean high water line) is not under PG&E's control. Due to the natural rough and precipitous conditions of the offshore area at Diablo Cove and near its southerly boundary, as shown in the aerial photograph, Figure 2.1-3, the area could only be occupied with great difficulty. (Some of these rocks have since been incorporated into the breakwater.) There is no history of public access to these rocks.

The Captain of the Port of Los Angeles-Long Beach, under the authority of 33 U.S.C. Section 1226 and Section 1231, has established a Security Zone in the Pacific Ocean, from surface to bottom, within a 2,000-yard radius of DCPP centered at position 35 12' 23"N, 120 51' 23" W (Datum 83). No person or vessel may enter or remain in this Security Zone without the permission of the Captain of the Port Los Angeles-Long

Beach. This Security Zone will be enforced by representatives of the Captain of the Port of Los Angeles-Long Beach, San Luis Obispo County Sheriff, and DCPP Security.

2.1.2.1.4 Population and Population Distribution

PG&E has reviewed the original population totals and projections within the 50-mile radius of the plant. The following population data are based on the 2000 census and on projections based on estimates prepared by the State of California Department of Finance. The portion of California that lies within 50 miles of the site is relatively sparsely populated, having approximately 424, 013 residents in 2000. A circle with a 50-mile radius includes most of San Luis Obispo County, about one-third of Santa Barbara County, and a minor, sparsely-populated portion of Monterey County. About 55 percent of the area within the 50-mile circle is on land, the balance being on the Pacific Ocean.

The 2000 census population of this region is very close to that projected in the original Final Safety Analysis Report (FSAR), and subsequent projections by the Department of Finance are similarly close to earlier projections. Table 2.1-1 shows population trends of the State of California and of San Luis Obispo and Santa Barbara Counties. Table 2.1-2 shows the growth since 1960 of the principal cities within 50 miles of the site. Table 2.1-3 lists all communities within 50 miles having a population of 1000 or more, gives distance and direction from the site, and gives the 2000 population.

2.1.2.1.4.1 Population Within 10 Miles

In 1980, approximately 16,760 persons resided within 10 miles of the site. The 1990 census counted approximately 22,200 residents within the same 10 miles. The 2000 census counted approximately 23,661 residents within the same 10 miles. As in 1980, the nearest residence is about 1-1/2 miles north-northwest of the site and two persons occupy this dwelling. There are 9 permanently inhabited dwellings, for about 17 residents, within 5 miles of the plant. The population within the 6-mile radius, used in the emergency plan, is estimated to be 100.

Figure 2.1-4 shows the 2000 population distribution within a 10-mile radius wherein the area is divided into 22-1/2° sectors, with part circles of radii of 1, 2, 3, 4, 5, and 10 miles. Figures 2.1-5 and 2.1-6 show projected population distributions for 2010 and 2025, respectively, and are based primarily on population projections published by the California Department of Finance. The distributions are based on the assumption that the land usage will not change in character during the next 25 years, and that population growth within 10 miles will be proportional to growth in San Luis Obispo County as a whole.

2.1.2.1.4.2 Population Between 10 and 50 Miles

Figure 2.1-7 shows the 2000 population distribution between 10 and 50 miles, within the sectors of 22-1/2°, as before, but with part circles of radii of 10, 20, 30, 40, and 50 miles. Figures 2.1-8 and 2.1-9 show projected distributions for 2010 and 2025, respectively, and are based primarily on population projections published by the California Department of Finance and interviews with area government officials. In 2000, some 82 percent of those persons within 50 miles of the site resided in the population centers listed in Table 2.1-3.

2.1.2.1.4.3 Low Population Zone

As previously mentioned, the population within the 6-mile radius used in the emergency plan is estimated to be 100. This number is derived from a survey of residences in this area, and approximates the low population zone (LPZ) as defined in 10 CFR 100. Coincidentally, 6 miles is the distance to the nearest residential community development at Los Osos, north of the site. It is assumed that the population within this mountainous and largely inaccessible zone will stay constant for the foreseeable future. Figure 2.1-15 shows the low population zone.

2.1.2.1.4.4 Transient Population

In addition to the resident population presented in the tables and population distribution charts, there is a seasonal influx of vacation and weekend visitors, especially during the summer months. This influx is heaviest along the coast from Avila Beach to south of Oceano.

During August, the month of heaviest influx, the maximum overnight transient population in motels and state parks in this area is approximately 100,000 persons. However, there are no significant seasonal or diurnal shifts in population or population distribution within the LPZ. Table 2.1-4 lists transient population for recreation areas within 50 miles of the site for the periods of record listed.

Within the LPZ, the maximum recorded number of persons at any single time is estimated to be 5000. This figure is provided by the State Department of Parks and Recreation and corresponds to the maximum daytime use of Montana de Oro State Park. Overnight use is considerably less, an estimated maximum of 400. Evacuation of these numbers of persons from the park in the event of a radiation release could be accomplished as provided for in the emergency plan, with a reasonable probability that no injury would result. For all accident analyses considered in Chapter 15, there is a wide margin of safety between exposures at the outer boundary of the LPZ for a 30-day period following a postulated accident and the allowable doses considered acceptable in 10 CFR 100 for the same location.

2.1.2.1.4.5 Population Center Distance

The population center distance as defined in 10 CFR 100 is approximately 10 miles, the distance to the nearest boundary of San Luis Obispo, situated beyond the San Luis Range, east-northeast of the site, with a 2000 population of 44,174.

2.1.2.1.4.6 Public Facilities and Institutions

Several elementary schools are located within 10 miles of the site, near Los Osos and Avila Beach. These serve the local community and do not draw from outlying areas. California Polytechnic State University is 12 miles north-northeast of the DCPP site and has an enrollment of approximately 16,000. Cuesta College is located 10 miles northeast of the DCPP site and has an enrollment of approximately 7,000.

Montana de Oro State Park is located north of the site. Its area of principal use is along the beach, between 4 and 5 miles north-northwest of the site. The total number of visitor days during a 12-month period over the last five years averages approximately 680,000.

2.1.2.1.5 Boundaries for Establishing Effluent Release Limits

On land, the boundary line of the unrestricted area (as defined in 10 CFR 20) coincides with the site boundary as shown in Figure 2.1-2. The relationship of the exclusion area to the unrestricted area and the site area is also shown in Figure 2.1-2. Control of access to the land area within this boundary is as described for the exclusion area control. As therein described, no special provisions have been made for control of access, during normal operation, to the offshore area below the mean high water line. Occupancy of this area by any member of the public is expected to result in exposures, during normal operation, within the limits established by 10 CFR 20 and will be maintained as low as reasonably achievable (ALARA).

2.1.2.1.6 Uses of Adjacent Lands and Waters

The San Luis Range, attaining a height of 1800 feet, dominates the region between the site and US Route 101. This upland country is used to a limited extent for grazing beef cattle and, to a very minor extent, dairy cattle. The terrain east of US Route 101, lying in the mostly inaccessible Santa Lucia Mountains, is sparsely populated with little development. A large portion of this area is included within the Los Padres National Forest.

2.1.2.1.6.1 Agriculture

San Luis Obispo County has relatively little level land, except for a few small coastal valleys such as the Santa Maria and San Luis Valleys, and some land along the county's northern border in the Salinas Valley and Carrizo Plain areas. Farming is a significant land use in the county. Principal crops include wine grapes, vegetables, *cattle, nurseries, fruits, nuts, and grain. There are several vineyards and wineries located in the county. The county's leading agricultural product is wine grapes, valued at $123,500,000 in 2003. The total farm acreage in the county is approximately 1,300,000. The county contains a total of 2,128,640 acres.*

2.1.2.1.6.2 Dairying

The nearest dairying activity is 12 miles northeast of the site at California State Polytechnic College and produces 1000 gallons of milk per day. Some replacement heifers and dry cows are sometimes pastured on property adjacent to site.

2.1.2.1.6.3 Fisheries

The DCPP site is located between two fishing harbors that support commercial and sport fishing activities. Port San Luis Harbor is located in Avila Beach, approximately 7 miles downcoast of the DCPP site. Morro Bay Harbor is located in Morro Bay, approximately 14 miles upcoast of the site. In 2003 the combined landings for the sport catch (known as commercial passenger fishing vessel fleet) totaled approximately 110,510 rockfish and 10,683 fish of other species, for a total of 8 fishing vessels. Sport catch are calculated by the number of fish caught.

Commercial landings are calculated by poundage of landings by port. In 2003 at Port San Luis and at Morro Bay Harbor, the landings were estimated to be as follows: 450,423 pounds of rockfish, 1,433,650 pounds of squid; 534,000 pounds of crab; 282,696 pounds of shrimp; and 1,592 pounds of urchins were landed.

There has been a dramatic decrease since 1970 in the abalone fishery, with approximately 621,000 pounds taken in 1966 and 200,000 pounds taken in 1970. Some data suggest that the southern movement of the Southern California sea otter may have had an impact on the red abalone population.

2.1.2.1.6.4 Surface and Groundwater

As discussed in Section 2.4, there are two public water supply groundwater basins within 10 miles of the site. Avila Beach County Water and Sewer District and San Miguelito Mutual Water and Sewer Company provide water to the Avila Beach and Avila Valley area.

2.1.2.1.6.5 Land Usage Within 5 Miles

An annual land use census is required by Regulatory Guide 4.8[(1)]. A census is required to be conducted at least once per year during the growing season (between February 15 and December 1 for the Diablo Canyon environs). The census is to identify the nearest milk animal and nearest garden greater than 50 square meters (500 square feet) producing broadleaf vegetation in each of 16 22-1/2° sectors within a distance of 8 kilometers (5 miles) of the plant. In addition, Regulatory Guide 4.8 requires the *identification of the location of the nearest residence in each of the 16 sectors within a distance of 5 miles.*

Land owners were identified from San Luis Obispo County records, and direct contact was made with them or their tenants. The only agricultural activities indicated by County personnel were cattle grazing in much of the area surrounding the site, and a farm in the east-southeast sector (along the site access road) producing legumes and cereal grass (grains).

Personal and telephone contacts with the land owners or tenants also identified a household garden greater than 500 square feet in the east sector in addition to the above mentioned farming. No milk animals were identified on these properties or within the first 5 miles in any sector.

The 1985 land use census results indicate the land use in the vicinity of the plant site has not changed significantly from that identified in Amendment 44 (July 1976) of the FSAR. A summary of the land use census is presented in Table 2.1-5 and Figure 2.1-14. Table 2.1-5 lists the distances measured in miles from the Unit 1 reactor centerline to the nearest animal, residence, and vegetable garden. The locations of gardens or farms greater than 500 square feet are shown in Figure 2.1-14. There is a farm in the southeast sector along the site access road on the coastal plateau; it starts approximately 2 miles from the plant and extends to 4.5 miles from the plant. Figure 2.1-14 also shows the nearest residence is 1.55 miles north-northwest of the plant. Nine permanent residences were identified within 5 miles of the plant.

2.1.3 REFERENCES

1. Regulatory Guide 4.8, Environmental Technical Specifications for Nuclear Power Plants, USNRC, December 1975.

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

This section establishes that DCPP is designed to safely withstand the effects of potential accidents at, or as a result of the presence of, other industrial, transportation, mining, and military installations or operations near the site which may have a potentially significant effect on the safe operation of the plant.

2.2.1 DESIGN BASES

2.2.1.1 Nearby Industrial, Transportation, and Military Facilities Safety Function Requirement

(1) Protection of the Intake Structure

The DCPP intake structure is appropriately protected from marine vessel collisions that may pose a significant hazard to the PG&E Design Class I auxiliary saltwater system.

2.2.1.2 10 CFR Part 100 Reactor Site Criteria

PG&E considered the characteristics peculiar to the site, the site location and the use characteristics of the site environs when evaluating the DCPP site.

2.2.1.3 Regulatory Guide 1.78, June 1974 - Assumptions For Evaluating The Habitability Of A Nuclear Power Plant Control Room During A Postulated Hazardous Chemical Release

The DCPP control room is appropriately protected from hazardous chemicals that may be discharged as a result of events and conditions outside the control of the plant.

2.2.2 LOCATIONS AND ROUTES

There are no industrial, transportation, mining, or military facilities within 5 miles of the DCPP site. The DCPP site is adjacent to the Pacific Ocean; however, no people or vessels are permitted to come within 2000 yards of the plant. Refer to Section 2.1.

Coastal shipping lanes are approximately 20 miles offshore. Prior to 1998, there were local tankers coming into and out of Estero Bay, which is north of the DCPP site. There is no further tanker traffic in either Port San Luis or Estero Bay. The local tanker terminal at Estero Bay closed in 1994, and Avila Pier ceased operation in 1998. Petroleum products and crude oil are no longer stored at Avila Beach, since the storage tanks there were removed in 1999. However, some petroleum products and crude oil continue to be stored at Estero Bay approximately 10 miles from the DCPP site.

Port San Luis Harbor and the Point San Luis Lighthouse are located approximately 6.5 miles south-southeast of the DCPP site. The Coast Guard operates and maintains a modern light station and navigating equipment adjacent to the lighthouse. Located approximately 6.5 miles east-southeast of the DCPP site is the Cal Poly pier that is owned by California Polytechnic State University and is used for research.

US Highway 101 is the main arterial road serving the coastal region in this portion of California. It passes about 9 miles east of the site, separated from it by the Irish Hills. US Highway 1 passes 10 miles to the north and carries moderate traffic between San Luis Obispo and the coast. The nearest public access is by county roads in Clark Valley (5 miles north) and See Canyon (5 miles east). Access to the site is by Avila Beach Drive (county road) to the entrance of Pacific Gas and Electric Company's (PG&E's) private access road (easement).

The Union Pacific Transportation Company provides rail service to the county by a route that roughly parallels US Highway 101. There is no spur track into the site.

The San Luis Obispo County Airport is 12 miles east of the site. There is a smaller airport near Oceano, 15 miles east-southeast of the DCPP site, which accommodates private planes only. The Camp San Luis Obispo airfield, 8 miles northeast of the DCPP site, is not operational.

Aircraft operating out of the San Luis Obispo County Airport are limited to general aviation, freight, and commuter flights weighing generally less than 100,000 pounds.

The approach route for visual landings passes 8 miles from the site, on the far side of the San Luis Range. The approach route for a portion of the traffic passes within approximately 4 miles of the DCPP site at an elevation of 3,000 feet, but is used infrequently.

The largest military and industrial complex is Vandenberg Air Force Base, located about 35 miles south-southeast of the site in Santa Barbara County. Vandenberg Air Force Base employs several thousand military and civilian personnel in the area of Lompoc-Santa Maria.

The closest US Army installation is the Hunter-Liggett Military Reservation located in Monterey County approximately 45 miles north of the site. The California National Guard maintains Camp Roberts, located on the border of Monterey County and San Luis Obispo County, southeast of the Hunter-Liggett Military Reservation and approximately 30 miles north of the DCPP site, and Camp San Luis Obispo, in San Luis Obispo County, located about 14 miles northeast of the DCPP site. In addition, as previously described, a US Coast Guard light station is located in Avila Beach on property commonly known as the Point San Luis Lighthouse property.

2.2.2.1 DESCRIPTIONS

No products are manufactured, stored or transported within 5 miles of DCPP site. Industry in the vicinity of DCPP site is mainly light and of a local nature serving the needs of agriculture in the area. Food processing and refining of crude oil are the area's major industries, although the numbers employed are not large.

2.2.3 SAFETY EVALUATION

2.2.3.1 Nearby Industrial, Transportation, and Military Facilities Safety Function Requirement

(1) Protection of the Intake Structure

Collisions of marine vessels with the intake structure are not a significant hazard to the safe operation of DCPP. The intake structure is protected by massive breakwaters as described in chapters 2.4 and 3.4. Jack R. Benjamin & Associates, Inc., (JBA) (Reference 1), consultants to PG&E, assessed the likelihood of marine vessel collisions with the intake structure thereby endangering operation of the PG&E Design Class I auxiliary saltwater (ASW) system pumps.

JBA investigated maritime traffic in the vicinity of Diablo Canyon looking for events that could lead to a marine vessel collision with the intake structure. The study considered 13 categories of large vessels, those greater than 100 feet in length and of more than 250 long tons displacement, and a single category including all smaller vessels. Quantitative data were developed for the larger vessel collisions and probability analyses made for both storm dependent and storm independent cases. Development of quantitative data for the smaller vessel collision proved to be not feasible due to the lack of sufficient records of small vessel traffic and accidental groundings. As an alternative approach for smaller vessels, a deterministic structural analysis was made to assess the potential damage to the intake structure for an extreme case collision scenario involving the largest of the smaller vessel category.

The investigations were based on the following conservative assumptions that resulted in computed frequencies of collisions substantially greater than likely to occur:

(1) The entire length of the breakwater is degraded to the mean lower low water (MLLW) level

(2) Any vessel crossing the breakwater boundary always impacts the intake structure

(3) All barges (either large or small vessels) are empty and have only a 3 to 4-foot draft

The storm-independent case probabilistic analysis for large vessels yielded a best-estimate frequency of 6.7 x 10^{-6} collisions per year. The storm-dependent probabilistic analysis, the best-estimate annual frequency of collision increased only moderately to 1.9×10^{-5}. The storm independent case, which realistically assumes vessels arriving randomly and encountering storm conditions only a fraction of the time, was used as the basis for evaluating the frequency of impact.

The results of the deterministic analysis indicated that collisions with the intake structure by small vessels of 250 tons or less would be inconsequential to the PG&E Design Class I function of the ASW pumps.

The study demonstrated that larger marine vessels are not likely to collide with the intake structure and that collisions by smaller vessels would not cause sufficient damage to the intake structure to impair the operation of the ASW system. It is, therefore, concluded that collisions of marine vessels with the intake structure are not a significant hazard to the safe operation of the power plant even if the entire breakwater were to be degraded to the MLLW level. The breakwater in the fully repaired normal condition provides a substantial physical barrier to vessels approaching the intake structure, further reducing the potential hazard from collisions.

2.2.3.2 10 CFR Part 100 Reactor Site Criteria

PG&E has identified and evaluated the characteristics peculiar to the site, including the site location and the use characteristics of the site environment.

DCPP is located in a remote, sparsely populated, undeveloped site that is an essentially agricultural area. None of the activities described in Sections 2.2.2.1 and 2.2.2.2 could constitute a hazard to the plant.

Due to very limited industry within San Luis Obispo County, any products or materials manufactured, stored, or transported beyond 5 miles are not likely to be a significant hazard to the plant.

No explosive or combustible materials are stored within 5 miles of the site and no natural gas or other pipelines pass within 5 miles of the DCPP site. The risk of fire is minimal, since adjacent hills are sparsely covered with low lying brush and grasses.

Missiles fired from Vandenberg Air Force base to the Western Pacific Missile Range are not directed north or west. Missile launch sites are some 36 miles due south of DCPP. Polar orbit launches are in a southerly direction.

Local shipping tankers come within 5 to 10 miles of the DCPP site. Coastal shipping lanes are approximately 20 miles offshore. Because shipping does not approach closer than 5 miles of the DCPP site and a limited number of tankers pass through, shipping does not pose a hazard to the DCPP site.

Aircraft operating in the area are small in size and few in number. Take-off and landing patterns do not come near the DCPP site and the probability of aircraft impacting or damaging the plant is very low.

On the DCPP site, as well as surrounding properties, there are no natural-draft cooling towers or other tall structures with a potential for damage to PG&E Design Class I equipment or structures in the event of collapse of such tall structures.

2.2.3.3 Regulatory Guide 1.78, June 1974 - Assumptions for Evaluating the Habitability of a Nuclear Power Plant Control Room During a Postulated Hazardous Chemical Release

DCPP has evaluated control room habitability in accordance with the Regulatory Guide 1.78, June 1974 screening criteria for stationary sources. Details of the evaluations are discussed in Sections 6.4 and 9.4.1.

The nearby industrial, transportation, and military facilities are all located at distances greater than 5 miles from the site. Chemicals stored or situated or frequently shipped by rail, water, or road routes at distances greater than 5 miles from the plant need not be considered because, if a release occurs at such a distance, atmospheric dispersion will dilute and disperse the incoming plume to such a degree that either toxic limits will never be reached or there would be sufficient time for the control room operators to take appropriate action. In addition, the probability of a plume remaining within a given sector for a long period of time is quite small.

2.2.4 REFERENCES

1. Charles A. Kircher, et al, Frequency of Vessel Impact With the Diablo Canyon Intake Structure, Jack R. Benjamin & Associates, Inc., Mountain View, CA, 1982.

2.3 METEOROLOGY

Historical summaries of normal and extreme values of meteorological parameters such as wind speed, wind direction, ambient air temperature, and precipitation are presented in this section. The historical data contained in this section were used for initial plant licensing and are not required to be updated. Wind speed and wind direction for tornado and dose analysis are discussed in Sections 3.3.2 and 15.5, respectively. The ambient air temperature for heating, ventilating, and air conditioning (HVAC) analysis is discussed in Section 9.4. Precipitation data for probable maximum flood are discussed in Section 2.4.3.

The onsite meteorological monitoring program is discussed in this section. The program provides meteorological information for use in (1) estimating potential radiation doses to the public resulting from actual, routine or accidental releases of radioactive materials to the atmosphere and (2) coping with radiological emergencies. Note that the dispersion factors calculated by the onsite meteorological monitoring program are produced and used for purposes of immediate radionuclide transport and dispersion assessment, and are therefore separate from those used for design bases radiological analyses as described in Section 15.5.5.

2.3.1 DESIGN BASES

2.3.1.1 General Design Criterion 11, 1967 – Control Room

Meteorological monitoring is provided to support actions to maintain and control the safe operational status of the plant from the control room.

2.3.1.2 General Design Criterion 12, 1967 – Instrumentation and Control Systems

Instrumentation and controls are provided as required to monitor meteorological conditions.

2.3.1.3 Meteorology Safety Function Requirements

(1) Calculation of Atmospheric Dispersion

The calculated relative concentration values are provided for use in (1) estimating potential radiation doses to the public resulting from actual, routine or accidental releases of radioactive materials to the atmosphere and (2) coping with radiological emergencies.

2.3.1.4 Safety Guide 23, February 1972 – Onsite Meteorological Programs

An onsite meteorological monitoring program that is capable of providing meteorological data needed to estimate potential radiation doses to the public as a result of routine or accidental release of radioactive material to the atmosphere and to asses other environmental effects is provided.

2.3.1.5 Regulatory Guide 1.97, Revision 3 – Instrumentation for Light-Water-Cooled Nuclear Power Plants to Assess Plant and Environs Conditions During and Following an Accident

Control room display instrumentation for use in determining the magnitude of the release of radioactive materials and in continuously assessing such releases during and following an accident is provided.

2.3.1.6 Regulatory Guide 1.111, March 1976 – Methods for Estimating Atmospheric Transport and Dispersion of Gaseous Effluents in Routine Releases from Light-Water-Cooled Reactors

Annual average relative concentration values are used during the postulated accident to estimate the long-term atmospheric transport and dispersion of gaseous effluents in routine releases.

2.3.1.7 NUREG-0737 (Item III.A.2), November 1980 – Clarification of TMI Action Plan Requirements

Item III.A.2 - Improving Licensee Emergency Preparedness–Long-Term:

Reasonable assurance is provided that adequate protective measures can and will be taken in the event of a radiological emergency. The requirements of NUREG-0654, Revision 1, November 1980, which provides meteorological criteria to ensure that the methods, systems and equipment for monitoring and assessing the consequences of radiological emergencies are in use, is implemented.

Item III.A.2.2 - Meteorological Data: NUREG-0737, Supplement 1, January 1983 provides the requirements for III.A.2.2 as follows:

Reliable indication of the meteorological variables specified in Regulatory Guide 1.97, Revision 3, for site meteorology is provided.

2.3.1.8 IE Information Notice 84-91, December 1984 – Quality Control Problems of Meteorological Measurements Programs

Meteorological data that are climatically representative, of high quality, and reliable in providing credible dose calculations and recommendations for protective actions in an emergency situation, and for doses calculated to assess the impact of routine releases of radioactive material to the atmosphere are available.

2.3.2 REGIONAL CLIMATOLOGY

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

2.3.2.1 Data Sources

The information used in determining the regional meteorological characteristics of Diablo Canyon Power Plant (DCPP) site consists of climatological summaries, technical studies, and reports by Dye (Reference 2), Edinger (Reference 3), Elford (Reference 4), Holzworth (Reference 6), Martin (Reference 8), Thom References 13 and 14), and a Weather Bureau Technical Paper (Reference 16), all pertinent to the region.

2.3.2.2 General Climate

The climate of the area is typical of the central California coastal region and is characterized by small diurnal and seasonal temperature variations and scanty summer precipitation. The prevailing wind direction is from the northwest, and the annual average wind speed is about 10 mph. In the dry season, which extends from May through September, the Pacific high-pressure area is located off the California coast, and the Pacific storm track is located far to the north. Moderate to strong sea breezes are common during the afternoon hours of this season while, at night, weak offshore drainage winds (land breezes) are prevalent. There is a high frequency of fog and low stratus clouds during the dry season, associated with a strong low-level temperature inversion.

The mean height of the inversion base is approximately 1100 feet. During the wet season, extending from November through March, the Pacific high-pressure area moves southward and weakens in intensity, allowing storms to move into and across the state. More than 80 percent of the annual rainfall occurs during this 5-month period. Middle and high clouds occur mainly with winter storm activity, and strong winds may be associated with the arrival and passage of storm systems. April and October are considered transitional months separating the two seasons.

The coastal mountains that extend in a general northwest-to-southeast direction along the coastline affect the general circulation patterns. The wind direction in many areas is more likely a result of the local terrain than it is of the prevailing circulation. This range of mountains is indented by numerous canyons and valleys, each of which has its own land-sea breeze regime. As the air flows along this barrier, it is dispersed inland by the valleys and canyons that indent the coastal range. Once the air enters these valleys and canyons, it is controlled by the local terrain features.

In areas where there are no breaks in the coastal range, the magnitude of the wind speed is increased and the variation in the wind direction decreases as the air is forced along the barrier. However, because of the irregular terrain profile and increased mechanical turbulence due to the rough terrain, vertical mixing and lateral meandering *under the inversion are enhanced. Therefore, emissions injected into the coastal regime are transported and dispersed by a complex array of land-sea breeze regimes that lead to rapid dispersion in both the vertical and horizontal planes.*

2.3.2.3 *Severe Weather*

The annual mean number of days with severe weather conditions, such as tornadoes and ice storms at west coast sites, is zero. Thunderstorms and hail are also rare phenomena, the average occurrence being less than three days per year, as reported by Dye (Reference 2) and Thom (Reference 13). The maximum recorded precipitation in the San Luis Obispo region is 2.35 inches in 1 hour at the DCPP site, and 5.98 inches in 24 hours at San Luis Obispo. The 24 hour maximum and the 1 hour maximum occurred on March 4, 1978. The 24 hour maximum recorded precipitation resulted from a semistationary low-pressure system located southwest of the central California coast that produced a series of frontal waves. These surges of warm, moist air moved into and across the central portion of the state and produced heavy precipitation. The 1 hour maximum was associated with the passage of a strong cold front.

The maximum recorded annual precipitation at San Luis Obispo was 54.53 inches during 1969. The average annual precipitation at San Luis Obispo is 21.53 inches. There are no fastest mile wind speed records in the general area of Diablo Canyon; surface peak gusts at 46 mph have been reported at Santa Maria, California, and peak gusts of 56 mph have been recorded at the 250 foot level on the tower at DCPP site. The frequency of occurrence of peak gusts of this magnitude is approximately once every 10 years. The 100 year recurrence interval wind speed for the site area is 80 mph, Thom (Reference 14). The number of days having a high air pollution potential averages ten per year, Holzworth (Reference 6).

One of the most severe tropical storms on record along the Southern California coast occurred September 24-25, 1939. It moved northward off the Southern California coast and came inland on the 25th in the Los Angeles area, but dissipated rapidly. This storm was attended by extremely heavy rains and winds of gale force in the Los Angeles area and southward. Precipitation amounts recorded during the storm are shown below; these data show that this storm had little or no effect on the DCPP site:

	Precipitation in Inches			
Location	September 24	September 25	September 26	Total
Los Angeles	1.62	3.96	0.04	5.62
Oxnard	0.00	1.67	0.02	1.69
Ventura	0.00	0.80	0.00	0.80
Santa Barbara	0.09	0.16	0.01	0.26
Santa Maria	1.13	0.29	0.00	1.42
San Luis Obispo	0.04	0.48	0.07	0.59

By definition, gale force winds range from 30 to 60 mph, so the intensity of this storm was about equal to the expected wind speed having a recurrence interval of 10 years at the site. The maximum daily precipitation of 4 inches recorded in this storm was well under the expected maximum probable precipitation estimated for DCPP site.

2.3.3 LOCAL METEOROLOGY

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

2.3.3.1 Data from Offsite Sources

Meteorological data from National Weather Service Stations are indicated below and data from other sources near the DCPP site had been gathered and reported previously in prior FSAR Updates as Appendix 2.3J. Since this appendix, as well as other appendices to this chapter (including Appendices 2.3A-K, 2.4A-C, and 2.5A-F) is merely of historical value at this time, they have been removed from this revision of the FSAR Update and are included only by reference collectively as Reference 27. However, all of these appendices are maintained available for review at PG&E offices. In addition, these appendices have also been docketed at the NRC as a part of Revision 0 through Revision 10 of the FSAR Update. Further, since the nearest National Weather Service Station is located approximately 30 airline miles southeast of the DCPP site, and since other offsite sources are separated from the site by rugged terrain, data from other sources are not considered indicative of site conditions. The only representative local data source is the onsite meteorological measurement program, data from which are summarized in Section 2.3.3.2, below, and presented in detail in Appendix 2.3J of Reference 27.

Precipitation and ambient air temperature data at National Weather Service stations surrounding DCPP are shown in Tables 2.3-6 and 2.3-7. Annual and monthly wind data summaries for Santa Maria, California, are shown in Tables 2.3-8 through 2.3-20.

The results of the analysis of the meteorological observations made at the DCPP site are summarized in the following sections and presented in further detail in References 1, 9, 10, and 11, and in Appendix 2.3J of Reference 27.

2.3.3.2 Onsite Normal and Extreme Values of Meteorological Parameters

Summaries of normal and extreme values of meteorological parameters are presented in this section for six stations located on DCPP property. Detailed data are included in the locations described in this section. Additional data from continued long-term operation of one site station (Station E) are presented in Appendix 2.3J of Reference 27.

2.3.3.2.1 Wind Speed and Wind Direction

The wind speed units in References 1, 9, and 10, and in Appendix 2.3J of Reference 27 are in miles per hour and were estimated to the nearest mile per hour. The wind speed values in the tables contained in Reference 9 and Appendix 2.3J of Reference 27 refer to the values included in each category. For example, the category of 4-7 includes all wind speed values for 4, 5, 6, and 7 mph. The wind speed values in the tables contained in References 1 and 10 are the midpoint values of the class intervals.

The seasonal and annual frequency distributions of wind speed and wind direction are shown graphically in Figures 1 through 4, Reference 9. The percentage occurrence (expressed as the percent of the total number of observations in the period) for each of the 16 wind direction sectors is represented by the length of the bars on the wind rose, and the average wind speed for each wind direction sector is plotted at the end of each bar.

The annual frequency distribution of wind speed and wind direction at the six DCPP stations is shown in Figure 1, Reference 9. The patterns at Stations E, A, and B are grossly similar with about 50 percent of the observations comprising northwesterly winds with average speeds of 10 to 15 mph. The percentage of indicated hourly mean wind speeds that are 2 mph or less varies from 21 percent at Station E to 14 percent at Station A. This variation may be attributed, in part, to the higher starting threshold of the sensors at Station E.

As shown in Tables S.2-1 and S.2-2 of Reference 11, there is a 4 percent difference in the percentage of indicated hourly mean wind speeds that are 2 mph or less for the two concurrent sets of measurements at the 25 foot level of Station E for the period April 1970 through March 1972. The measurements presented in Table S.2-1 were obtained from a lightweight cup and vane wind system, while the observations shown in Table S.2-2 are concurrent measurements obtained from a Bendix-Friez aerovane wind system. The wind flows at Stations C and D, both located in Diablo Canyon, reflect the channeling of the wind by the canyon walls; the predominant directions are up-canyon and down-canyon. The wind distribution at Station F tends to be somewhat circular, because of topographical factors, with the highest mean wind speeds identified with easterly flow.

The highest recorded peak gust at Station E is 84 mph, and the maximum recorded hourly mean wind speed is 54 mph, both recorded at the 76-m level of the primary tower.

Figure 2 of Reference 9 shows that during the dry season northwesterly flow is predominant; Figure 3 of Reference 9 shows there is an increase in southeasterly flow during the wet season compared to the annual distribution. Wind frequency distributions for the transitional months, April and October, show all six stations similar to the annual patterns. Because of the small variability from month to month within a particular season, monthly wind distributions have not been prepared.

The strong diurnal variability of the wind patterns at DCPP site is revealed in Figure 5 and in Figures I-1 through I-7 of Reference 9. The following time periods are shown in the figures for the six stations: Day, 1200-1700 PDT; Night, 2300-0500 PDT; Morning 0600-1100 PDT; and Evening, 1800-2200 PDT. During the day, the winds are northwesterly at Stations E, A, and B. The daytime flow at Stations C and D in Diablo Canyon is directed up-canyon. The most frequent daytime wind direction at Station F is from the northwest. During the night and morning periods, northerly and easterly drainage winds are typically present at all stations. The average nighttime wind speeds at Stations E, A, and B are approximately one-half as great as the average daytime speeds. At the other three stations, no large differences in mean wind speed between the daytime and nighttime regimes are apparent.

2.3.3.2.2 Ambient Air Temperature

Average ambient air temperatures for each month of the year, calculated from the hourly temperature measurements at Stations E, B, and F up to the year 1980, are plotted in Figures I-15 through I-17 of Reference 9. The average annual temperature at the plant site is about 55°F. Generally, the warmest mean monthly temperature occurs in October, and the coldest mean monthly temperature occurs in December. The highest and lowest hourly temperatures recorded at the Diablo Canyon site through the year 2000 were 97°F in October 1987 and 33°F in December 1990, respectively.

2.3.3.2.3 Atmospheric Water Vapor and Fog

Measurements of atmospheric water vapor and fog observations are not present throughout the entire meteorological data collection program. However, measurements of these parameters are not essential at DCPP site since regional data are adequate for design purposes and cooling towers are not being used.

2.3.3.2.4 Precipitation

Rainfall measurements made at the DCPP shown herein for two report periods. The first period was from July 1, 1967 through October 31, 1969 and is discussed in Section 7.7 and summarized in Table 7 of Appendix 2.3A in Reference 27. The second period was from May 1973 through April 1981 and is discussed in Section 2.3J.4.2 and summarized in Table 2.3J-3 of Appendix 2.3J of Reference 27. Precipitation occurs typically during the period of late October through the first part of May and most frequently in the presence of southeasterly wind flow in advance of a frontal system. The average annual precipitation in the area is about 16 inches. The highest monthly total during the period of record (1967-1981) was 11.26 inches as shown in Section 7.7 of Appendix 2.3A of Reference 27. The greatest amount of precipitation received in a 24 hour period was 3.28 inches as shown in Section 2.3J.4.2 and Table 2.3J-3 of Appendix 2.3J of Reference 27. These maximums were recorded in January 1969 and March 1978, respectively. The maximum hourly amount recorded at DCPP site during the periods of record is 2.35 inches as shown in Section 2.3J.4.2 of Appendix 2.3J of

Reference 27. The 1978-1979 winter season with 35.22 inches of rainfall was one of the heaviest precipitation seasons of record.

2.3.3.2.5 Wind Direction Persistence

The steadiness of the wind flow at DCPP site has been studied by tabulating the number of consecutive hours the hourly mean wind direction remained within a given 22.5° angular sector. The results, expressed in terms of percentage of all hourly observations, are plotted in Figures I-8 through I-14 of Reference 9, and presented also in Table 2.3J-17 of Appendix 2.3J of Reference 27, for periods ranging from 1 through 24 hours. The mean wind direction at all stations in the analysis of Reference 9 remained within the same 22.5° sector for two consecutive hours or longer in 31 to 42 percent of the observations. The persistence of the wind direction decreases rapidly for a longer time period with only 3 to 4 percent of the observations showing a persistence of 8 hours or longer.

The longest run of persistent wind direction in the total set of measurements occurred at Station B where a northwest wind direction lasted for 51 consecutive hours. The longest period of calm (hourly mean wind speed less than 1 mph) observed at Station E, near the plant location, was 10 hours. As shown in Table 2.3-1, the percentage of the total hourly mean wind speed observations that are less than 1 mph at Station E is 5.9 and 4.9 percent at the 25 foot and 250 foot levels, respectively. The percentage of time that the mean hourly wind speed would be less than 1 mph for 8 consecutive hours or longer is less than 0.5.

As indicated by the persistence analysis, despite the prevalence of the marine inversion and the northwesterly wind flow gradient along the California coast, the long-term accumulation of plant emissions in any particular geographical area downwind is virtually impossible. Pollutants injected into the marine inversion layer of the coastal wind regime are transported and dispersed by a complex array of land-sea breeze regimes that exist all along the coast wherever canyons or valleys indent the coastal range. These conclusions are strongly supported by Edinger's (Reference 3) comprehensive analysis of the influence of terrain and thermal stratification on wind circulations along the California coast, as well as the onsite diffusion studies by Cramer and Record (Reference 1).

2.3.3.2.6 Atmospheric Stability Conditions Defined by Turbulence Measurements

The Pasquill (Reference 17) stability categories (see Table 2.3-141) are frequently used as a convenient practical index for gauging the dispersal capacity of the atmosphere. For example, unstable and near-neutral stability conditions (Pasquill Categories A, B, C, D) are favorable for the dilution of pollutants; on the other hand, poor dilution occurs under stable conditions (Pasquill Categories E, F, G). Following a procedure outlined by Slade (Reference 12) the turbulence measurements obtained from the bidirectional vanes at Station E have been used to classify the wind observations at DCPP site according to the Pasquill stability categories. Table 4 of Reference 9, shows the *relationship between the range in azimuth and vertical wind angle and the Pasquill stability categories. Scaling factors used to convert the angle ranges to standard deviations were determined from the data presented in Table 2 of Reference 9. The annual wind distributions for the 250 foot level at Station E, given by the measurements made during the period from July 1967 through October 1969, are classified according to the range values of azimuth and vertical wind angles associated with the various Pasquill categories, Tables I-2 through I-6 and Tables I-14 through I-18 of Reference 9. The corresponding annual wind distributions for the 25 foot level are similarly classified, using the 250 foot turbulence measurements, in Tables I-8 through I-12, and I-20 through I-24 of Reference 9. As mentioned above, turbulence measurements were available only at the 250-foot level for this period.*

As shown in Table 5 of Reference 9, when the range in azimuth wind angle is used to determine the number of wind observations at Station E in the various Pasquill stability categories, 57 percent of the total observations are in the stable E, F, and G categories. The unstable categories A, B, and C contain 25 percent of the total observations. When the range in vertical wind angle is used to classify the Station E wind data, less than 20 percent of the total observations are in the E, F, and G stable categories. The unstable categories A, B, and C account for about 65 percent of the total observations. These apparent inconsistencies are explained in part by terrain restrictions on the azimuth wind variations at the site.

The results also indicate the routine presence of relatively large vertical turbulence intensities that are caused by the rough terrain at the site. Therefore, it is concluded that the range in vertical wind-angle is a better index of turbulent mixing at DCPP site than the range in azimuth angle. This conclusion is strongly supported by Luna and Church's (Reference 7) comprehensive analysis of the use of measured vertical turbulence values to define stability conditions at sites with rough terrain.

Toward the end of the 2 year meteorological measurement program, July 1967 through October 1969, a question arose as to the applicability of the azimuth and vertical wind fluctuations measured at the 250-foot level in determining the site dispersion characteristics for low-level releases resulting from an accident. Therefore, 1 year (October 1969 through September 1970) of concurrent azimuth and vertical wind-angle measurements were obtained at the 25- and 250-foot levels. A detailed analysis of these data is contained in Reference 10 where Tables S.1-1 through S.1-6, pages 7 through 12, and Tables S.1-13 through S.1-18, pages 19 through 24, contains the annual wind distributions classified according to the azimuth wind-angle for the 25- and 250-foot levels, respectively. The annual distributions classified according to vertical wind angle for the two levels are shown in Tables S.1-7 through S.1-12, pages 13 through 18, and Tables S.1-19 through S.1-24, pages 25 through 30.

When the range in azimuth wind-angle is used to classify these concurrent measurements, the 250 foot azimuth range yields the same percentages as the data collected during the period July 1967 through October 1969 (57 percent for the E, F, and G stable categories, and 25 percent for the unstable categories A, B, and C).

However, when the azimuth range measured at the 25 foot level is used to classify the total number of observations at the 25-foot level in the various Pasquill stability categories, 48 percent of the total observations are in the E, F, and G stable categories; the unstable categories A, B, and C contain 29 percent of the total observations.

When the range in vertical wind-angle is used to classify the 1 year of concurrent measurement, again at the 250 foot level, there is very little change from the data collected during the period of July 1967 through October 1969: 17 percent of the total observations are in the E, F, and G stable categories and 68 percent are in the unstable categories A, B, and C. At the 25-foot level, only 7 percent of the total observations are in the E, F, and G stable categories. The percentage of total observations in the unstable categories A, B, and C is 80 percent, compared to 66 percent calculated from the wind-angle measurements from the 250 foot level during the period of July 1967 through October 1969.

Because of the poor dilution normally associated with the Pasquill F and G stable categories, the annual percentage occurrences of the F and G categories, in combination with onshore winds of 2 mph or less were also determined and are shown in Tables S.1-1 and S.1-7 of Reference 10. Onshore wind directions include winds for southeast through west-northwest, measured clockwise. The results from the 25-foot level indicate that the Pasquill F and G and onshore wind combination defined above occurs slightly less than 4 percent of the time when the azimuth angle-range data are used as indices, and slightly more than 3 percent of the time when the vertical range-angle data are used as indices. These percentages, which were calculated from the wind-angle measurements from the 250-foot level, are approximately one percentage point less than those for the 25 foot level shown in Table 5 of Reference 9.

The seasonal distributions given in Figure 6 of Reference 9 show the highest percentage of stable conditions during the dry season for both the azimuth and vertical wind-angle classifications. Additional analyses and discussion are presented in Appendix 2.3K of Reference 27.

2.3.3.2.7 Atmospheric Stability Conditions Defined by Vertical Temperature Gradient Measurements

The gross relationship between the hourly wind observations at Station E and the thermal stratification can be shown by classifying the wind data into three stability categories defined by the vertical temperature difference measured between the 250- and 25-foot levels on the tower.

The following ranges of the vertical temperature difference between these two levels can be used to define the categories:

Stable	$(T_{250} - T_{25}) = +25.0$ to $+1.6$°F
Near Neutral	$(T_{250} - T_{25}) = +1.5$ to -1.5°F
Unstable	$(T_{250} - T_{25}) = -1.6$ to -25.0°F

A discussion of the effect of measurement interval on stability estimates of temperature gradients is provided in Appendix 2.3G of Reference 27.

Joint frequency distributions of hourly wind speed and wind direction measurements at the 250-foot level for the three stability categories are contained in Reference 9, Tables I-26 through I-28. Similar frequency distributions of the hourly wind observations at the 25-foot level are shown in Tables I-30 through I-32.

Over 70 percent of all the wind observations are grouped in the near-neutral category at both levels. This large percentage is probably explained by the small vertical temperature gradients in the surface layer of the maritime air that reaches the tower during onshore winds; the proximity of the tower to the shoreline, and the intense turbulent mixing induced by the rough terrain at DCPP site. Approximately 5 percent of the total hourly observations at each level are identified with stable thermal stratification and mean wind speeds of 2 mph or less. The percentage of total hourly observations and onshore winds (southeast through west-northwest measured clockwise), with mean wind speeds of 2 mph or less, is 3.2 for the 250-foot level and 1.4 for the 25-foot level. The corresponding percentages for the Pasquill F and G stability categories, as shown in Table 2 of Reference 10, page 4, are 6 at the 250-foot level and 3.2 at the 25-foot level when the range data for the vertical wind angle are used to define the Pasquill categories.

Wind data (speed and direction) classified into seven stability categories (Pasquill A through G) are shown in Tables 2.3-21 through 2.3-27. The wind data were measured at the 250-foot level and the vertical temperature difference measurements are 250-foot level minus 25-foot level. The wind speed values are in miles per hour and the values in the tables refer to the midpoint of each class interval. The rows are labeled with the wind direction at the midpoint of 22.5° intervals:

Midpoint, mph	*Class Interval, mph*
Calm	*Less than 1*
2.0	*1-3*
5.1	*4-7*
9.6	*8-12*
15.1	*13-18*
21.1	*19-24*
39.6	*> 24*

Wind data (speed and direction) classified into seven stability categories (Pasquill A through G) for the period May 1973 through April 1974 are shown in Tables 2.3-42 through 2.3-48. The wind data were measured at the 25-foot level and the vertical temperature difference measurements are 250-foot level minus 25-foot level. The wind speed values are in miles per hour and the values in the tables refer to the midpoint of each class interval. The rows are labeled with the wind direction at the midpoint of 22.5° intervals:

Midpoint, mph	*Class Interval, mph*
Calm	*Less than 1*
1.8	*0.6 to 3.1*
5.1	*3.1 to 7.1*
9.6	*7.1 to 12.1*
15.1	*12.1 to 18.1*
21.1	*18.1 to 24.1*
39.6	*> 24*

Wind data (speed and direction) classified into seven stability categories (Pasquill A through G) for the period May 1973 through April 1975 are shown in Tables 2.3-49 through 2.3-55 on an annual basis, and in Tables 2.3-56 through 2.3-139, on a monthly basis. The wind data were measured at the 10-meter level, and the vertical temperature gradient measurements were made at 76 meters minus 10 meters.

The wind speed values are in miles per hour and the values in the tables refer to the midpoint of each class interval. The rows are labeled with the wind direction at the midpoint of 22.5° intervals:

Midpoint, mph	*Class Interval, mph*
1.5	*1.0-3*
5.1	*3.1-7*
9.6	*7.1-12*
15.1	*12.1-18*
21.1	*18.1-24*
29.6	*24.1-35*
40.1	*35.1-45*
50.1	*>45*

These 2 years of data, May 1973 through April 1975, are considered representative of long-term conditions at DCPP site, and are in agreement with other data taken at the site, such as that in Reference 9, Table I-7, page 2.3A-87, July 1967 through December 1969 and the data in Appendix 2.3J of Reference 27. The prevailing wind direction is from the northwest and the mean annual wind speed is about 10 mph. Between 70 to 90 percent of the observations are contained in the stability classes D and E, Tables 2.3-42 through 2.3-48, and Tables 2.3-49 through 2.3-55.

During the August 1969 review by the Environmental Science Services Administration (ESSA) for Diablo Canyon Nuclear Unit 2, it was requested that the wind data be processed so that the distribution of wind speeds of 3 mph and less could be examined. Since the wind sensor had a nominal starting speed of 2.2 mph, the following procedures were followed in processing the wind data:

(1) Calm refers to hourly wind speed traces indicating zero wind speed and hourly direction traces that were either squarewave or straight line

(2) The values shown for the 1 and 2 mph categories were determined by equal area averaging

(3) For wind speed entries in the 1 and 2 mph categories that show a calm wind direction, refer to hourly records for which a mean wind direction could not be defined

Additional analyses and discussion are presented in Appendix 2.3J of Reference 27.

2.3.3.2.8 Atmospheric Stability Conditions Defined by Onsite Diffusion Studies

Twenty-seven onsite field tests involving releases of smoke and fluorescent particles were made during various meteorological regimes. The data from these tests were used for verifying the diffusion model computations by comparing predicted ground level concentrations to observed concentrations. The data also served as a guide in the selection of parameters used in the long-term diffusion model. The analysis of the field measurements was performed by the GCA Corporation and is described in Reference 1. Additional analyses and discussion are contained in Appendix 2.3K of Reference 27.

Analysis of the meteorological and diffusion data obtained during the onsite field tests at Diablo Canyon leads to the following conclusions:

(1) For daytime elevated (250 foot) releases into northwesterly flow, only four measured concentrations exceeded the values predicted by the Pasquill-Gifford curve for Category D; these four values exceed the predicted values for Category D by a factor of 2 or less.

(2) For releases into southeasterly flow (generally prefrontal conditions), the Pasquill-Gifford curve for Category B serves as the upper bound for the concentrations measured during the 250-foot releases.

(3) During light and variable winds, the fluorescent particle tracer was found along the coast both north and south of the release point; all measured concentrations for both 250 and 25-foot releases were below the Pasquill-Gifford curve for Category B.

2.3.3.3 Potential Influence of the Plant and Its Facilities on Local Meteorology

Modification of local meteorological parameters is not expected by the presence and operation of DCPP.

2.3.3.4 Topographical Description

The topographical features within a 10-mile radius of the plant site are shown in Figure 2.3-1. The vertical cross sections for the eight 22.5° onshore wind direction sectors (southeast through west-northwest) radiating from the plant are shown in Figure 2.3-2. Modification of the local topography by the plant is considered negligible.

Topographical influences on both short-term and long-term diffusion estimates are quite pronounced in that the ridge lines east of the plant location extend at least to the average height of the marine inversion base.

The implications of this barrier are:

(1) *Any material released that is diverted along the coastline will be diluted and dispersed by the natural valleys and canyons, which indent the coastline.*

(2) *Any material released that is transported over the ridgeline will be distributed through a deep layer because of the enhanced vertical mixing due to topographic features.*

2.3.4 ONSITE METEOROLOGICAL MEASUREMENT PROGRAM

The preoperational meteorological data collection program is described in detail in the references. This meteorological program was designed and has been updated continually to meet the requirements of Safety Guide 23, February 1972 (Reference 21).

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

Onsite Meteorological Measurement Program

Data were collected from a comprehensive station network, shown as points A through F in Figure 2.3-3, over a 28-month period from July 1967 through October 1969. Because of a considerable amount of missing data during the first few months of the operation of the meteorological data network, the data collection period was extended four additional months beyond July 1, 1969, to eliminate any bias in the annual distributions caused by incomplete data. The above meteorological measurements were also supplemented by a 12-month program of concurrent turbulence measurements at heights of 250 and 25 feet from October 1969 through September 1970, and by a 24-month program of concurrent wind measurements at the 25 foot level of Station E using a Bendix-Friez aerovane wind system and a lightweight cup and vane system from April 1970 through March 1972. A complete description of the onsite meteorological measurement program is given in Reference 9.

Figure 2.3-1 shows the plant location and site boundary. Locations of Stations A through F of the meteorological measurement network are as shown in Figure 2.3-3.

Stations A and B are approximately 3000 feet southeast of the plant location at elevations of 125 and 600 feet Mean Sea Level (MSL), respectively. Station C at elevation of 75 feet MSL and Station D at 350 feet MSL are in Diablo Canyon. Stations E and F are at elevations 85 and 920 feet MSL, respectively. The meteorological instruments at each of the six stations consisted of a Climet Model CI-26 cup and vane assembly mounted at a height of 35 feet above the surface. In addition, air temperature measurements were made at Station B at a height of 5 feet above the surface using a Foxboro Capillary System.

At Station E, currently the primary tower site, meteorological sensors were mounted at heights of 250 and 25 feet on a 260-foot tower. The sensors at the 250-foot level comprised a Bendix-Friez Model 120 Aerovane, a Meteorology Research Incorporated bidirectional vane, and a platinum resistance thermometer for measuring the vertical temperature gradient. The sensor installation at the 25-foot level comprised a Bendix-Friez Model 120 Aerovane and a platinum resistance thermometer for measuring ambient air temperature. A second Meteorology Research Incorporated bidirectional vane was installed at the 25-foot level at Station E in October 1969, and a Climet Model CI-26 cup and vane system was installed at the 25-foot level of Station E in April 1970 to obtain supplementary data. A tipping-bucket rain gauge was located near Station E at the surface.

At Station F, approximately 3000 feet directly east of the plant location at an elevation of 920 feet MSL, a Bendix-Friez Model 120 Aerovane and a Meteorology Research Incorporated bidirectional vane were mounted at the top of a 100-foot tower. Ambient air temperature measurements were made at the 5-foot level by means of a Foxboro Capillary Sensor. Accuracy specifications of the instrumentation used prior to the spring of 1973 are:

(1) The Bendix-Friez Model 120 Aerovane has a stated accuracy of ±2° over the complete direction range, an average wind speed error of ±0.5 mph for speeds under 10 mph, and ±1 mph for speeds between 10 and 200 mph

(2) A Climet Model CI-26 wind speed sensor has a stated accuracy of 2 percent or 0.25 mph (whichever is greater) and a wind direction accuracy of ±5°

(3) Meteorology Research Inc. bivanes have stated accuracies of ±3.6° for horizontal and ±2° for vertical direction

(4) The platinum resistance temperature gradient measurement system has an accuracy of ±0.2°F

Additional descriptions of the instruments are contained in Reference 9. The temperature gradient system and the Bendix-Friez wind systems were calibrated annually or more often when required. The lightweight cup and vane wind systems and *the bidirectional wind systems were calibrated every 90 days, or sooner when required. Inspection was performed on a daily basis, and maintenance as necessary.*

All of the meteorological sensor outputs from the network described above were recorded on continuous strip chart recorders at the site. Measurements of wind speed, azimuth wind direction, ambient air temperature, and vertical temperature gradient were reduced as hourly averages; rain gauge measurements were reduced to hourly totals; bidirectional vane measurements of the fluctuations in azimuth and vertical wind angles at Stations E and F were abstracted from the chart records in the form of 10 minute range values for the last 10 minutes of each hour. These range values were converted to 10 minute standard deviations of azimuth and vertical wind angle by the use of simple scaling factors and classified according to stability category following a procedure outlined by Slade (Reference 12).

Subsequent to November 1969, Station E became the primary meteorological measurement site at Diablo Canyon, and measurements were discontinued at Stations B, C, D, and F. Measurements at Station A were continued through August 1974.

During the spring of 1973 the instrumentation was changed. The Climet and Bendix-Friez systems were replaced with Teledyne Geotech Series 50 cup and vane sensors to improve reliability and response characteristics. The resistance thermometer system was changed to 4-wire Rosemont bridges and Teledyne Geotech aspirated shields and a sensor was added at the 150-foot level. The precipitation measurement system was changed to a weighing bucket gauge with a potentiometer. Signals from all of the above devices are processed by Teledyne Geotech Series 40 processors that provide output voltages and currents of 0-5 Vdc and 0-1 milliampere, respectively, to the digital and strip chart recorders. A Cambridge systems/EG&G chilled mirror dew point system was added at this time to provide dew point and backup ambient temperature at the 25 foot level. H. E. Cramer Corporation installed signal conditioning equipment of their own design that produced analog signals from the above equipment and the existing bivane equipment that were equivalent to 5 minute values of:

(1) Means of all parameters, except precipitation

(2) Variance of horizontal and vertical wind directions

(3) Peak wind speeds

The signal conditioning provided by H. E. Cramer also converted the Teledyne Geotech 0-360° wind direction output to a 0-540° wind direction signal to accomplish Items 1 and 2 above. H. E. Cramer also provided a digitizing and recording system that utilized Nonlinear Systems' equipment for digitizing and a Bright Industries 7-track magnetic tape recorder for storage of the 5 minute data.

In 1973, a minicomputer and printer were added to the digital system in the control room. Digital data were taken at the tape recorder input and transmitted to the control *room computer. The computer system was designed to calculate and display downwind concentrations based on real-time data.*

The weighing bucket precipitation gauge was replaced with a tipping bucket gauge in December 1976.

In December 1978, Station E was again upgraded. The equipment was moved to a new equipment shelter at the site and completely rewired. Although the sensors were retained, considerable changes were made to the processors and recording system. A new microprocessor temperature processor was installed to replace the Rosemont Bridge system and improve the accuracy of the temperature difference measurements. The entire H.E. Cramer signal conditioning, digitizing, and recording system was replaced by a Teledyne Geotech Automet V microprocessor-based digital data system. The Automet V also replaced the minicomputer and only the printer remained in the control room. The multipoint Servo recorder was modified to record 25 foot temperature and temperature differences: 150 foot by 25 foot and 250 foot by 25 foot. The Bright Industries 7-track magnetic tape recorder was replaced with a Kennedy Model 9000, 9-track, 1600 bits per inch, phase encoded, buffered tape system.

In June 1980, the system was again upgraded by incorporation of improved wind direction processors using a linear output voltage with no step changes and phase-locked loops to increase immunity to sensor signal distortion. The new processors output a signal that changes linearly from 0 to 5 volts at 180° and back to 0 volts at 360°. A digital signal is used to identify which 180° is being processed. This eliminates errors in the 360° transition as 0° and 360° are both 0 volts rather than 5 volts for 360° in the old system. Digital processing was also changed at this time to use unit vectors for standard deviation and mean direction calculations to eliminate potential ambiguities inherent in the older system. An additional communications link was installed at this time to transmit meteorological data to the technical support center (TSC) computer.

In May of 1981, the Automet system was revised to allow polling from the DCPP Emergency Assessment and Response System (EARS) computer, and a math processor was incorporated to speed up the processing of wind direction vectors.

In October of 1981, a new 60 meter tower was installed as a backup meteorological system. The backup tower has two levels of wind direction, wind speed, and temperature instrumentation. It is located approximately 1.2 km southeast of the primary tower. The instruments are at the 10 meter and 60 meter levels. Wind speed and wind direction processing is identical to the primary system. The temperature processing incorporates new analog processors from Teledyne Geotech with the same type of aspirated platinum resistance thermometers. The backup system is powered by batteries and is capable of 7 days of operation without external power.

The Automet microcomputer for the backup system is located in the TSC and receives data digitally from a remote terminal at the tower location over a 4-wire communications *link. The backup system printer and a 9-track magnetic tape recorder are also located in the TSC. A switching system has been incorporated into the primary meteorological printer in the control room and allows the backup system printout to be substituted for the primary system printout. This switching system reconfigures the backup system automatically when the switch is actuated so that 5-minute updates of the current 15-minute logs derived from backup data are printed on the control room printer. The primary system data are output on the printer in the TSC when the backup system is selected in the control room.*

In the spring of 1982, a visibility measurement system was installed at the base of the primary tower. The system relates local visual range to forward light scattering by the air along a 4 foot horizontal path. This system was removed in February 1985 after a sufficient record of information had been collected.

Onsite Meteorological Measurement Program (Current)

The current onsite meteorological monitoring system consists of two independent subsystems that measure meteorological conditions and process the information into useable data. The measurement subsystems consist of a primary meteorological tower and a backup meteorological tower.

The primary meteorological tower location is shown in Figure 2.3-3 as Station E. There are instruments located at the 10 m, 46 m, and 76 m elevations. The 10 m and 76 m elevations have wind speed, wind direction, and temperature sensors. The 46 m elevation has a temperature sensor. The 10 m level also has a dewpoint sensor. There is a precipitation measurement system at the base of the tower.

The backup meteorological tower is located approximately 1.2 km southeast of the primary tower and is listed as Station A in Figure 2.3-3. There are wind speed, wind direction, and temperature sensors at the 10 m and 60 m elevations.

The processors for the above instruments reside in the meteorological facilities located near the towers. The temperature in these facilities is maintained to support processor operation. These processors provide input to strip chart recorders and the meteorological dataloggers. The dataloggers provide input to their respective meteorological computers.

The primary meteorological computer is located in the primary meteorological facility. The backup meteorological computer is located in the TSC. These two computers communicate with each other and the EARS. The primary meteorological computer also communicates with the Unit 1 Transient Recording System (TRS) server. The backup meteorological computer also communicates with the Unit 2 TRS server. Primary and backup meteorological data are available on the Plant Process Computers (PPCs) via the TRS servers. Thus meteorological data are available in the control room and emergency response facilities in accordance with NUREG-0654, Revision 1, November 1980 (Reference 23).

A detailed discussion of each of the above instruments is provided in the following sections.

2.3.4.1 Wind Measurement System

The wind direction processor supplies voltage and current signals corresponding to -180 to 0 to 180 degrees. A digital signal is provided to identify which 180-degree sector the signal represents.

The wind speed signal is processed to develop a voltage signal for the data acquisition system and a current signal for the strip chart recorder.

2.3.4.2 Temperature Measurement System

The primary tower temperature measurement system employs a microprocessor system in conjunction with platinum resistance temperature detectors (RTDs) to measure temperature at three levels on the meteorological tower.

Analog outputs of the temperature processor are recorded on a 3-channel multipoint recorder and depict:

(1) 10-m temperature in degrees Fahrenheit from 0 to 120

(2) temperature difference 46 m to 10 m from -15 to 21°F

(3) temperature difference 76 m to 10 m from -15 to 21°F

Temperature probes are housed in aspirated radiation shields. Radiation errors are limited to less than 0.2°F at a radiation intensity of 1.56 gram-calories/cm/min. This radiation level represents approximately twice the highest summer radiation level for the DCPP site. Aspirators are individually monitored by motor current sensors and temperatures are invalidated if the motor current is out of a specified range.

The backup tower 10-m processor supplies an intermediate output that is used to sum with the intermediate output of the 60-m processor and provide a temperature difference output from the 60-m processor. Both processors supply a current signal to a multipoint strip chart recorder at the tower location and a voltage signal to the data acquisition system.

Measurement ranges are 0 to 120°F for the 10-m temperature and -15 to 21°F for the 60- to 10-m temperature difference.

2.3.4.3 Dew Point Measurement System

A chilled mirror dew point measuring system is used to monitor the dew point at the primary tower 10-m level. The output voltage signal represents a range of 0 to 100°F. The sensor head is equipped with an aspirator to present a representative atmospheric sample to the mirror.

The voltage signal is further processed to generate a buffered voltage output to the data acquisition system and a current signal to the strip chart recorder.

2.3.4.4 Precipitation Measurement System

Precipitation is measured by a tipping bucket rain gauge that delivers a pulse for each 0.01-inch increment of rainfall. This pulse is digitally accumulated by a processor module. The digital accumulator resets to zero after the 250th pulse and begins a new cycle. The digital accumulator output is processed by a digital-to-analog converter that provides a voltage signal to the data acquisition system and a current signal to the strip chart recorder.

2.3.4.5 Supplemental Measurement System

A supplemental meteorological measurement system is present in the vicinity of the DCPP site. This supplemental measurement system consists of three Doppler SODAR (Sonic Detection and Ranging) and seven tower sites located as indicated in Figure 2.3-4.

The Doppler sounders provide remote sensing of wind speed, wind direction, standard deviation of wind direction variability (sigma theta), vertical velocity, and standard deviation of vertical velocity (sigma w), as well as information on echo characteristics useful in deducing the presence of inversion layers. At each Doppler location, the above parameters are provided as 15-minute average values for each of twenty 30-m thick vertical layers above the instrument site. Layer midpoints extend from 40 m to 610 m above ground level, providing data to heights just exceeding the maximum height of the local terrain. A thorough evaluation of the Doppler technique has been made by the National Oceanic and Atmospheric Administration (NOAA) (Reference 25). The NOAA evaluation of the Doppler produced correlation coefficients on the order of 0.93 and higher for both wind speed and direction in comparison with measurements by sonic anemometers.

The offsite towers provide measurements of wind speed, wind direction, sigma theta, and temperatures as 15 minute averages. All of the supplemental tower measurements are taken at or near the 10-m level using instrumentation designed to meet or exceed ANSI/ANS 2.5-1984 (Reference 24) for meteorological measurements at nuclear plant sites. Tower data are telemetered to the TSC, Alternate Technical Support Center/Operational Support Center (Alternate TSC/OSC), Emergency Operations Facility (EOF), and General Office headquarters on a continuous basis. The data are archived as a permanent record. SODAR data are available on-demand via a dial-up modem interface in the EOF or remotely via computer.

Onsite meteorological data and supplemental wind speed and direction data are processed by the EARS software. The data are provided to the Meteorological Information and Dose Assessment System (MIDAS) software to make estimates and predictions of atmospheric effluent transport and diffusion during and immediately following an accidental airborne radioactivity release from the plant. The software can produce initial transport and diffusion estimates for the plume exposure emergency planning zone (EPZ) within 15 minutes following the classification of an incident. The MIDAS model is designed to use actual 15-minute average meteorological data from onsite and offsite meteorological measurement systems. The output from the model includes the dimensions, position, locations, and arrival time of the plume.

If one or more of the supplemental tower data are unavailable, EARS and MIDAS will fail over to the supplemental tower most representative of the region that is missing data. If transmission of all supplemental data fails, EARS and MIDAS will continue to be functional with onsite meteorological data as the only source.

2.3.4.6 Meteorological Datalogger

A datalogger is installed in both the primary and backup meteorological facilities. The dataloggers receive the outputs of the meteorological sensor signal processors and computer 15-minute averages and maximums. The dataloggers also assign quality values to each of the 15-minute values. On the quarter hour, the dataloggers output their 15-minute data sets to the meteorological computers.

The primary tower datalogger records the following:

(1) 10-m and 76-m wind speeds

(2) 10-m and 76-m wind direction

(3) 10-m temperature

(4) 76 –10-m temperature difference

(5) 46 –10-m temperature difference

(6) precipitation

(7) dewpoint

(8) 10-m, 46-m, and 76-m aspirator currents

The backup tower datalogger records the following:

(1) 10-m and 60-m wind speeds

(2) 10-m and 60-m wind direction

(3) 10-m temperature

(4) 60 –10-m temperature difference

(5) the sum of the aspirator currents

(6) battery monitor voltage

The dataloggers scan their inputs every 2 seconds (450 samples per 15 minutes). The following tests are performed to determine the validity of the meteorological sensor data:

(1) If the wind direction standard deviation (calculated using the Yamartino method) is less than 1, the wind data are considered invalid. (Appendix 2.3F of Reference 27 presents the historical Wind Direction Deviation Computation at Diablo Canyon and its reference has been retained to provide a continuity of understanding.

(2) If the 15-minute average wind speed is greater than 0.75 mph and the difference between the peak wind speed and the average wind speed is less than 0.3, then the wind speed data are considered invalid.

(3) If the wind speed is greater than 100 mph or less than 0 mph, that 2-second sample is invalid. If more than 150 samples are invalid (i.e., less than 10 minutes worth of good data), then the 15-minute wind speed data are invalid.

(4) If more than 150 delta temperature samples are greater than 21 or less than -15, then the 15-minute temperature difference data are invalid.

(5) If more than 150 dew point samples are greater than the 10-m temperature by 2 degrees, then the 15-minute dew point data are invalid.

(6) If more than 150 aspirator samples are out of a specified range, then both the 15-minute aspirator value and the associated temperature value are invalid.

2.3.4.7 Meteorological Computers

The primary meteorological computer resides in the primary meteorological facility and the backup meteorological computer is located in the TSC. The primary computer communicates with the primary datalogger, the Unit 1 TRS, the EARS server, and the backup meteorological tower computer. The backup meteorological computer communicates with the backup datalogger, the Unit 2 TRS, the EARS server, and the primary tower computer. Meteorological data are also available on the Unit 1 and Unit 2 PPCs via their respective TRS.

Each computer receives data from its respective datalogger on a 15-minute basis and sends its data set to the other computer. Each computer then calculates χ/Q, sigma Y, and sigma Z for 10 distances for both the primary and backup data sets. The primary computer sends both data sets to the Unit 1 TRS server and the EARS system. The backup computer sends both data sets to the Unit 2 TRS server and the EARS system.

Along with the 15-minute data set, each computer receives error flags, which are assigned to the appropriate data values, and these error flags are also sent to the PPCs and the EARS system. In this manner, the correct data quality is propagated through the entire system (datalogger, meteorological computer, PPC, and EARS).

The equation used to compute centerline χ/Q values is based on lateral fluctuations of wind direction (σ_A) for horizontal spread, and vertical temperature gradient (ΔT) for vertical spread of the plume for all daytime cases when the 10-meter speeds are not less than 1.5 m/sec. Nighttime cases in the same wind speed class are treated in accordance with the method of Mitchell and Timbre (Reference 19) as outlined in Table 2.3-144. For speeds less than 1.5 m/sec at the 10-meter level, both lateral and vertical spread of the plume are determined by the vertical temperature gradient. Estimates of both lateral and vertical plume dimensions are determined from the procedures described by Sagendorf (Reference 15).

Equations used to determine χ/Q are:

$$\frac{\chi}{Q} = \frac{1}{\bar{u}(\pi \sigma_y \sigma_z + CA)} \qquad (2.3\text{-}1)$$

$$\frac{\chi}{Q} = \frac{1}{\bar{u}(3\pi \sigma_y \sigma_z)} \qquad (2.3\text{-}2)$$

$$\frac{\chi}{Q} = \frac{1}{\pi\bar{u} + \sum_y \sigma_z} \qquad (2.3\text{-}3)$$

where:

$\frac{\chi}{Q}$ is the relative concentration (sec/m^3)

π is 3.14159

$\bar{u}$ is the wind speed at the 10-meter level (m/sec)

$\sigma_y \sigma_z$ are the lateral and vertical cloud dimensions, respectively, as a function of downwind distance. The vertical cloud dimension has an upper limiting value of 1000 m or the product (T_m) (H_m), whichever is less. T_m is a multiplier that is used as a simple substitute for the multiple reflection term and is approximately 0.8 (References 5 and 12)

H_m is the monthly average mixing layer depth for the four time periods of the day which were derived from Holzworth (Reference 6); data are given in Table 2.3-3.

A is the minimum cross-sectional area of the reactor building (1600 m^2)

C is constant (0.5)

$\sum_y$ = $M\sigma_y$ - at distances less than 800 m;

at distances greater than or equal to 800 m -

$$\sum{}_y = (M-1)(\sigma_y)_{800m} + \sigma_y$$

M is a correction factor for meandering and assumes the following values for speeds less than 2 m/sec:

Stability	$\bar{u} \leq 2$ m/sec M	2 m/sec$< \bar{u} \leq 6$ m/sec M
A,B,C	1	1
D	2	$(\bar{u}/6)^{-0.631}$
E	3	$(\bar{u}/6)^{-1.00}$
F	4	$(\bar{u}/6)^{-1.262}$
G	6	$(\bar{u}/6)^{-1.631}$

If both values at all levels are invalid, temperature differences (ΔT) are used to determine both lateral and vertical stability categories regardless of wind speed. When this occurs, the dispersion equation used contains the plume meandering correction term. The applicable correction term M for the specific stability and wind speed is that derived from Figure 3 of Regulatory Guide 1.145, Revision 1 (Reference 22), page 1.145-9.

During neutral (D) or stable (E, F, G) stability conditions when 10-m wind speed is less than 6 m/sec, horizontal plume meander is considered. This process consists of comparing the values from Equations 1 and 2, and selecting the higher value. This value is then compared with the value from Equation 3 and the lower value of these selected for χ/Q value. During all other meteorological conditions, plume meander is not considered. The appropriate χ/Q value in these cases is the higher value calculated from Equations 1 and 2.

The dispersion model described above is a generic model and was not developed specifically for the DCPP site. Certain factors specific to the DCPP site bear upon the use and interpretation of the modeling output. Analysis and treatment of such site-specific factors are presented in Appendix 2.3H of Reference 27.

2.3.4.8 Power Supply For Meteorological Equipment

Power for the main meteorological instrumentation building is supplied from Unit 1 480-V non-Class 1E bus. This source is supplied through a transfer switch and will automatically switch to Unit 2 480-V non-Class 1E bus if a failure occurs on the Unit 1 bus. The microprocessor and the meteorological sensors are backed up by an 8-hour battery source to prevent any problems during switching and maintain a continuous database.

The backup meteorological instrumentation is supplied with ac power from the underground Unit 2 12-kV startup bus. In case of an ac power failure, batteries supply emergency power for up to 1 week. During battery backup, the temperature system aspirators are not powered, thereby invalidating temperatures.

If the measurement systems are being operated on battery power, ΔT measurement is inactivated due to inability to aspirate the temperature shields. In this case, χ/Q values are based on lateral fluctuations of wind direction (σ_A) for both horizontal and vertical spread of the plume. Nighttime stability categories are adjusted, however, in accordance with the method of Mitchell and Timbre (Reference 19) as outlined in Table 2.3-144.

Should both automated tower systems become inoperative, a portable battery-powered meteorological system is available for deployment and use in providing χ/Q values for input to dose-calculation algorithms as described in the Emergency Plan and outlined in Appendix 2.3I of Reference 27. Translation of χ/Q values to centerline and plume-spread estimates may be accomplished in accordance with procedures in the same Appendix 2.3I of Reference 27. (Appendix 2.3I of Reference 27 is historical in nature; however, reference to it has been retained to provide a continuity of understanding. Current procedures meet the requirements of Regulatory Guide 1.145, Revision 1 (Reference 22)).

2.3.5 SHORT-TERM (ACCIDENT) DIFFUSION ESTIMATES

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

2.3.5.1 Objective

Estimates of dilution factors that apply at distances of 0.8 to 80 kilometers downwind from DCPP are shown in Table 2.3-41 for each wind direction sector. These dilution factors represent the distribution of χ/Q value within each wind direction sector at the various downwind distances.

2.3.5.2 Calculations

The cumulative probability distribution of the dilution factor at the distances noted above were computed using one of the diffusion models shown below for centerline dispersion estimates from a ground level release. These are defined as:

$$\frac{\chi}{Q} = \frac{1}{\bar{u}\left(\pi \sigma_y \sigma_z + CA\right)} \qquad (2.3\text{-}4)$$

$$\frac{\chi}{Q} = \frac{1}{3\pi \bar{u} \sigma_y \sigma_z} \qquad (2.3\text{-}5)$$

$$\frac{\chi}{Q} = \frac{1}{\pi \bar{u} \sum_y \sigma_y \sigma_z} \qquad (2.3\text{-}6)$$

where:

χ	=	*ground level centerline concentration, curies/cubic meter*
Q	=	*source emission rate, curies/second*
σ_y	=	*standard deviation of the lateral concentration distribution, meters*
σ_z	=	*standard deviation of the vertical concentration distribution, meters*
$\bar{u}$	=	*mean wind speed, meters/second*
C	=	*building wake shape factor, 0.5*
A	=	*minimum cross-sectional area of the reactor building, 1600 m^2*
Σ_y	=	*f(σy) = meander correction factor*

A complete description of the models and their selection for use is included in Reference 18.

The year-to-year variation in the frequency of occurrence of conditions producing high χ/Q values is small, so that data from one complete year are representative of the site. In fact, the addition of the second year's data from October 1970 through March 1971 *and April 1972 through September 1972, resulted in a change in percentage frequency for the combined F and G categories of only 0.1 percent. Frequency distributions for joint probabilities using the 2-year length of record are given in Tables 2.3-29 through 2.3-40. The wind speed values are in miles per hour and the values in the tables refer to the midpoint of each of the following class intervals: 0-3, 4-7, 8-12, 13-18, 19-24, and greater than 24. The rows are labeled with the wind direction at the midpoint of each 22.5° interval. The 1-year gap (April 1971 through March 1972) in the period of record, October 1970 through September 1972, resulted from an unauthorized bivane modification.*

Frequency distributions of wind speed and wind direction classified into seven stability classes as defined by the vertical temperature gradient are shown in Tables 2.3-21 through 2.3-28. The column headings are labeled in terms of mean hourly wind speed in miles per hour. The six wind speed categories are as follows: 1-3, 4-7, 8-12, 13-18, 19-24, and 25-55. The rows are labeled with the wind direction at the midpoints of 22.5° intervals. Table 2.3-28 shows the number of observations in each of the seven stability classes (Pasquill A through G) for the period of record July 1, 1967, through October 31, 1969, when the mean hourly wind speed is less than 1 mph. The wind data were measured at the 76 meter level, and the vertical temperature difference measurements are the 76 meter level minus the 10 meter level.

The radius of the low population zone (LPZ) at DCPP has been established to be 6 miles. Cumulative frequency distributions of atmospheric dilution factors at each 22.5° intersection with a 10,000-meter radius (slightly greater than 6 miles) for the period May 1973 through April 1975 are presented in Table 2.3-41, Sheets 7, 8, 9, and 10. Each data set used to compile the frequency distribution is comprised of averages taken over 1 hour, 8 hours, 16 hours, 3 days, or 26 days, using overlapping means updated at 1-hour increments as specified by the NRC.

Because of overlapping means, a 1 hour χ/Q is included in several observation periods: for example, an hourly χ/Q is included in 624 estimates of the 26-day averages. As a result, a single hourly measurement may influence the value of over 5 percent of the observations. Since overlapping means are used in the distributions, the data are not independent and no assumption of normality can be made. These data show χ/Q estimates from the 25th through the 100th percentile levels for each of the averaging periods.

2.3.6 LONG-TERM (ROUTINE) DIFFUSION ESTIMATES

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

2.3.6.1 Objective

Annual relative concentrations (χ/Q) were estimated for distances out to 80 kilometers from onsite meteorological data for the period May 1973 through April 1975. These relative concentrations are presented in Table 2.3-2; they were estimated using the *models described in Reference 18. The same program also produces cumulative frequency distributions for selected averaging periods using overlapping means having hourly updates. For critical offsite locations, measured lateral standard deviations of wind direction, σ_A, and bulk Richardson number, R_i, were used as the stability parameters in the computations. The meteorological input data were measured at the 10 meter level of the meteorological tower at DCPP site. Annual averaged relative concentrations calculated by the above methods are presented in Table 2.3-4.*

2.3.6.2 Calculations

The meteorological instrumentation that was used to obtain the input data for the previously discussed relative concentration calculations at DCPP site is described in Section 2.3.4. Procedures for obtaining annual averaged relative concentrations are described in detail in Reference 15.

2.3.6.3 Meteorological Parameters

The following assumptions were used in developing the meteorological input parameters required in the dispersion model:

(1) There is no wind direction change with height

(2) Wind speed changes with height can be estimated by a power law function where the exponent, P, varies with stability class and is assigned the following values:

Pasquill Stability Class	*Exponent (P)*
A & B	*0.10*
C	*0.15*
D	*0.20*
E	*0.25*
F & G	*0.30*

If more than five hourly observations are missing in any 24-hour period, the estimated 24-hour concentration value is not included in the analyses.

Meteorological data collected at DCPP site are representative of atmospheric conditions along a Pacific coastal area having a complex terrain near the shoreline. Use of these data in estimating downwind relative concentrations results in realistic estimates as shown in the report by Cramer and Record (Reference 1). This field program included ground level concentration measurements out to a distance of about 20 kilometers. All concentration measurements were approximated by near-neutral through unstable stability classifications, even though both vertical and lateral turbulence measurements, σ_E and σ_A in Table 3.1 of Reference 1, indicated several stable regimes.

Even during the nighttime periods when extreme stability may be expected, the relative concentrations in the area were characteristic of unstable lapse rates. Actual average temperature differences over the height of the tower for these trials, given in Table 2.3-142, show a high percentage of test periods with stable lapse rates. Five nighttime trials having light and variable winds were included; three were near ground level (8 meters) and two were elevated (76 meters) releases. Temperature gradient measurements indicated three of these trials having near-neutral and two with stable lapse rates, yet the measured ground level concentrations were at least two orders of magnitude less than the predicted peak concentrations for those stabilities. In fact, the diffusion rates, as shown in Figure 3-3 of Reference 1, based on measured ground level concentrations, were typical of those expected for extreme instability.

Results of this series of diffusion trials conducted at DCPP site have yielded considerable insight into the dispersal capabilities of a coastal site. They indicate that use of direct turbulence measurements and the split sigma approach to independently predict lateral and vertical cloud growth yield realistic estimates of site dilution factors without including any corrections or recirculation.

2.3.7 CONCLUSIONS

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

The principal conclusions reached as the result of the analysis of the data obtained during the onsite meteorological measurement program at DCPP site are listed below:

(1) Northwesterly wind directions with wind speeds averaging 10 to 15 mph can be expected to occur approximately 50 percent of the time.

(2) Wind directions within a 22.5° sector that persist for periods of 8 hours or longer will occur 3 to 4 percent of the time.

(3) Less than 4 percent of the total observations at the 25 foot level at Station E refer to the joint occurrence of mean wind speeds of 2 mph or less, onshore wind directions (southeast through west-northwest measured clockwise), and moderately stable and/or extremely stable thermal stratifications.

(4) Despite the prevalence of the marine inversion and the northwesterly wind flow gradient along the California coast in the dry season, the long-term accumulation of plant emissions, released routinely or accidentally, in any particular geographical area downwind from the plant is virtually impossible. Pollutants injected into the marine inversion layer of the coastal wind regime are transported and dispersed by a complex array of land-sea breeze regimes that exist all along the coast wherever canyons or valleys indent the coastal range. Because of the complexities of the wind circulation in these regimes and their fundamental diurnal nature, the

net result is a very effective and wide daily dispersal of any pollutants that are present in the marine coastal air.

2.3.8 SAFETY EVALUATION

2.3.8.1 General Design Criterion 11, 1967 – Control Room

Wind speed, wind direction, and differential air temperature measurements from the primary and backup meteorological towers are provided to control room personnel to respond to abnormal meteorological conditions in order to maintain safe operational status of the plant. The data are retrieved continually and provided to the PPC. High ambient air temperature is annunciated on the main control board.

2.3.8.2 General Design Criterion 12, 1967 – Instrumentation and Control Systems

Meteorological monitoring instrumentation is provided for DCPP Unit 1 and Unit 2 to provide meteorological conditions as discussed in Section 2.3.4.

2.3.8.3 Meteorology Safety Function Requirements

(1) Calculation of Atmospheric Dispersion

Calculation of atmospheric dispersion as discussed in Section 2.3.4.7 is based on methodology in Sagendorf (Reference 15) and Regulatory Guide 1.145, Revision 1.

2.3.8.4 Safety Guide 23, February 1972 – Onsite Meteorological Programs

As discussed in Section 2.3.4, the preoperational meteorological data collection program was designed and has been updated continually to meet the requirements of Safety Guide 23, February 1972.

2.3.8.5 Regulatory Guide 1.97, Revision 3 – Instrumentation for Light-Water-Cooled Nuclear Power Plants to Assess Plant and Environs Conditions During and Following an Accident

Wind speed, wind direction, and estimation of atmospheric stability indication in the control room provide information for use in determining the magnitude of the release of radioactive materials and in continuously assessing such releases during and following an accident (refer to Table 7.5-6 for a summary of compliance to Regulatory Guide 1.97, Revision 3).

2.3.8.6 Regulatory Guide 1.111, March 1976 – Methods for Estimating Atmospheric Transport and Dispersion of Gaseous Effluents in Routine Releases from Light-Water-Cooled Reactors

The pre-operational values of dilution factor and deposition factor used in the calculation of annual average offsite radiation dose are discussed in Section 11.3.7. The values of deposition rate were derived from Figure 7 of Regulatory Guide 1.111, March 1976, for a ground-level release.

2.3.8.7 NUREG-0737 (Item III.A.2), November 1980 – Clarification of TMI Action Plan Requirements

Item III.A.2 - Improving Licensee Emergency Preparedness–Long-Term:

As discussed in Section 2.3.4, the primary and backup meteorological data are available in the control room and emergency response facilities via the TRS servers and EARS, in accordance with NUREG-0654, Revision 1, November 1980.

As discussed in Section 2.3.4, the measurement subsystems consist of a primary meteorological tower and a backup meteorological tower. The primary meteorological computer and the backup meteorological computer communicate with each other, the EARS and also with the TRS server. Primary and backup meteorological data are available on the PPCs via the TRS servers and thus in the control room and emergency response facilities.

Item III.A.2.2 - Meteorological Data: NUREG-0737, Supplement 1, January 1983:

Table 7.5-6 and Section 2.3.8.5 summarize DCPP conformance with Regulatory Guide 1.97, Revision 3. Wind direction, wind speed, and estimation of atmospheric stability are categorized as Type E variables, based on Regulatory Guide 1.97, Revision 3. The PPC is used as the indicating device to display meteorological instrument signals. In addition, Type E, Category 3, recorders are located in the meteorological towers.

2.3.8.8 IE Information Notice 84-91, December 1984 – Quality Control Problems of Meteorological Measurements Programs

In addition to the primary meteorological towers, a supplemental meteorological measurement system is provided in the vicinity of the plant site in order to meet IE Information Notice 84-91. As discussed in Section 2.3.4.5, this supplemental measurement system consists of three Doppler SODAR and seven tower sites located as indicated in Figure 2.3-4. The primary and secondary meteorological towers in conjunction with the supplemental system adequately predict the meteorological conditions at the site boundary (800 meters) and beyond.

2.3.9 REFERENCES

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2.4 HYDROLOGIC ENGINEERING

2.4.1 DESIGN BASES

2.4.1.1 General Design Criterion 2, 1967 – Performance Standards

The PG&E Design Class I structures, systems and components essential to the prevention of accidents, or to mitigate of their consequences, are designed to withstand the additional forces that might be imposed by natural phenomena such as flooding.

2.4.1.2 Regulatory Guide 1.59, Revision 2, August 1977 – Design Basis Floods for Nuclear Power Plants

The PG&E Design Class I structures, systems, and components are designed to withstand and retain the capability to achieve and maintain cold shutdown during the worst probable site-related flood.

2.4.1.3 Regulatory Guide 1.102, Revision 1, September 1976 – Flood Protection for Nuclear Power Plants

The PG&E Design Class I structures, systems, and components are appropriately protected from damage caused by flooding through the use of exterior and incorporated barriers.

2.4.1.4 Regulatory Guide 1.125, Revision 1, October 1978 – Physical Models for Design and Operation of Hydraulic Structures and Systems for Nuclear Power Plants

Hydraulic modeling of the site intake breakwaters, systems, and structures is appropriately designed, tested, and documented to accurately describe the behavior of these plant facilities.

2.4.2 HYDROLOGIC DESCRIPTION

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

2.4.2.1 Site and Facilities

The general topography with outline of the drainage basin at Diablo Canyon Power Plant (DCPP) site is shown in Sheet 1 of 2 of Figure 2.4-1, reproduced from the United States Geological Survey (USGS) Port San Luis and Pismo Beach 7.5 minute topographic quadrangles (contour interval 40 feet, original scale 1:24,000). Figure 2.4-2 shows the Diablo Creek drainage basin to a larger scale. The area encompasses some 5 square miles and is bounded by ridges reaching a maximum elevation of 1819 feet at Saddle Peak. The figure also shows changes to the natural drainage features.

2.4.2.2 Hydrosphere

The hydrologic characteristics of the site are influenced by the Pacific Ocean on the west and by local storm runoff collected from the 5 square mile egg-shaped area drained by Diablo Creek. The maximum and minimum flows in Diablo Creek are highly variable. Average flows tend to be nearer the minimum flow value of 0.44 cfs. Maximum flows reflect short-term conditions associated with storm events. Usually within 1 or 2 days following a storm, flows return to normal. Flows during the wet season (October-April) vary daily and monthly. Dry season flows are sustained by groundwater seepage and are more consistent from day to day, tapering off over time. There is no other creek or river within the site area.

Water for the city of San Luis Obispo is obtained principally from Salinas Reservoir, about 23 miles east-northeast of the site. Whale Rock Reservoir on Old Creek, 17 miles north of the site, and Chorro Reservoir, about 13 miles northeast of the site, are also used. A few small uncovered reservoirs are used in connection with the San Luis Obispo water system and are located about 18 miles northeast of the site. A reservoir in Lopez Canyon is 20 miles east of the site. Smaller towns in the region of San Luis Obispo depend on wells for domestic water.

There are two public water supply groundwater basins within 10 miles of the DCPP site. Avila Beach County Water District serves Avila Beach (including Unocal) with water and sewer needs, and the San Miguelito Mutual Water District and Sewer District serves most of the Avila Valley area. An ocean water desalinization plant has been built and in operation at the site since 1985 (Reference 1).

The property owners to the north and south of the DCPP site capture surface water from small intermittent streams and springs for minimal domestic use. Property owned by PG&E captures water from Crowbar Canyon, 1 mile north of the DCPP site. PG&E's lessee captures water 2 to 4 miles south of the DCPP site from streams and springs between Pecho Canyon and Rattlesnake Canyon.

2.4.3 FLOODS

2.4.3.1 Flood History

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

Since 1968, Pacific Gas and Electric Company (PG&E) has kept a record of flows through a V-notched weir located on Diablo Creek, as shown in Figure 2.4-2.

Two major storms occurred in the area between the time the weir was established and June 1973. One occurred on January 18-25, 1969, and the other on January 16-19, 1973. On each occasion, streamflow washed out the weir so no definitive readings were obtained. Flood hydrograph reconstitution indicated that the

1969 flood could have peaked with a flow of approximately 430 cfs and the 1973 flood could have peaked with a flow of approximately 400 cfs.

A USGS gauging station (Los Berros Creek, No. 11-1416), located 21 miles southeast of the site near Nipomo, has a 15 square mile drainage basin, approximately three times the size of the Diablo Creek basin. The gauge at this station recorded a peak flow of 599 cfs on January 25, 1969. The flow at the same station on January 18, 1973, was about 324 cfs. Regional floods of January and February 1969 are reported by U.S. government publications in References 2, 3, and 4.

Ocean wave history is discussed in Reference 5.

2.4.3.2 Flood Design Considerations

2.4.3.2.1 Site Flooding

Topography and plant site arrangement limit flood design considerations to local floods from Diablo Creek and sea wave action from the Pacific Ocean. As discussed in Section 2.4.4, the canyon confining Diablo Creek remains intact and will pass any conceivable flood without hazard to PG&E Design Class I equipment. Channel blockage from landslides downstream of the plant, sufficient to flood the plant yard, is not possible because of the topographic arrangement of the site.

2.4.3.2.2 Flood Waves

Flooding conditions, for purposes of the following discussion, include the combined effects of a tsunami, wind-generated storm waves, storm surge ("piling up" of water near the shore due to a storm), and tides. The combination of these effects results in a rise and fall of the ocean surface level relative to a defined datum level. The reference datum is the mean lower low water level (MLLW). At DCPP, MLLW is 2.6 feet below the mean sea level (MSL), which is used as a reference datum for plant elevation. Values of water level rise and fall are expressed relative to MLLW. References to plant elevation are expressed relative to MSL.

When considering tsunami effects alone, the rise in water level is termed tsunami runup, and the fall of the water level is termed tsunami drawdown. Effects of both locally-generated (near-shore) tsunami and distantly-generated tsunami are considered. Tsunami runup and drawdown values given for locally-generated tsunami include the effects of subsidence at the plant site that is considered to occur as a result of near-shore earthquakes.

The wave terms are defined as follows:

Still Water Level (SWL)	The water level that includes the effects of tsunami, tide, and storm surge

Combined Wave Runup	The peak water level associated with storm wave action on top of SWL, but not including splash or spray effects associated with wave impacts
Splash Runup	The water level that includes wave runup effects plus splash effects, but not including spray effects
Combined Wave Drawdown	The lowest water level associated with tsunami coincident with low tide and short period storm waves

The rise in water level may result in submersion, associated hydraulic loading and ground erosion effects, and may result in flooding effects, on structures and system components located in the zone of influence.

The following effects are considered in determining the design water levels for DCPP:

Storm Waves: waves induced by the wind and pressure effects of a storm

Storm Surge: the "piling up" of water at the shore due to (a) a long duration storm wind acting on the water surface, (b) local reduction in atmospheric pressure, and (c) wave effects near the shoreline

Tide: the rise and fall of the surface of the ocean caused by the gravitational attraction of the sun and moon on the earth. Tidal range is typically based on the maximum annual higher high tide and the minimum annual lower low tide.

Tsunami: a long-period wave generated by a seismic event

In addition to water level changes resulting from the effects described above, the following effects are also considered:

Breakwater Damage: only partial credit is taken for protection provided by the breakwaters, considering that they could potentially be damaged by near-shore seismic activity or by storm waves

Resonance/Ponding Effects: local amplification of wave activity as a result of resonance effects in the intake basin, or increase in water level in the intake basin as a result of wave overtopping of the breakwaters, or wave ingress through the breakwater opening

Combined runup and drawdown effects on PG&E Design Class I structures and systems are as follows:

- Combined splash runup effects for applicable PG&E Design Class I facilities and their supporting structures are discussed in Section 2.4.7.6

- PG&E Design Class I systems include consideration of the effects of the combined drawdown and are discussed in Section 2.4.7.1.5

- Tsunami loads on the intake structure, including the effects of the combined wave runup are discussed in Section 2.4.7.6

2.4.3.2.3 Structural Evaluation

As discussed in Section 2.4.7.6, testing and analyses demonstrate that equipment and structures important to safety will remain operable in the event of a probable maximum tsunami, storm, and tide occurrence (Reference 21).

2.4.4 PROBABLE MAXIMUM FLOOD (PMF) ON STREAMS AND RIVERS

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The only stream on the site subject to a PMF study is Diablo Creek. The creek collects runoff from a drainage area of 5.19 square miles up from the ocean side.

The PMF was obtained by deriving an estimated probable maximum precipitation (PMP) with a duration of 24 hours over the subject drainage area. The most severe antecedent condition of ground wetness favorable to high flood runoff was assumed. In view of the low elevation of the site, snowmelt was not considered in the study.

It was assumed that during a PMF all culverts are plugged, and water is impounded to the crest of the lowest depression of the switchyard's fill. The artificial reservoir formed in this assumption is so small that the PMF could not affect the plant.

For a drainage area of 5.19 square miles, the PMF was found to have a peak discharge of 6878 cfs (1325 cfs/sq mi) or a total volume of about 4306 acre-feet for the 24-hour storm.

2.4.4.1 Probable Maximum Precipitation (PMP)

Due to the small drainage area of the site, a PMP with 24 hours duration of rain was selected. Determination of the PMP is based entirely on the methods and procedures outlined in Reference 6. The unrestricted cumulative convergence PMP determined by the above method is found to be 16.6 inches during the month of October. PMP values for other durations as interpolated by the method suggested in Reference 6 are shown in Table 2.4-1.

2.4.4.2 Precipitation Losses

Losses are a complex function of rain intensity and accumulated loss (as an index of ground wetness). Five loss rate variables in this study represent average loss, initial loss, rate of decrease of loss with wetness, relation of loss to rain intensity, and rate of recovery of loss rate between storm periods. The unit hydrograph and loss rate parameters are determined in a sequential successive approximation manner as described in Reference 7. Optimization of the basin parameters was performed with the aid of computer program No. 23-J-L211, "Unit Hydrograph and Loss Rate Optimization," developed by the U.S. Army Corps of Engineers, and modified by PG&E (Reference 8).

To obtain precipitation losses, the storm at DCPP site on January 24-25, 1969, was optimized with the runoff record at the USGS gauging station at Los Berros Creek for the same period. Actual rainfall-runoff optimization on Diablo Creek could have been done if the weir had not washed out during the major storms of 1969 and 1973. Nevertheless, geographic and geologic conditions of Los Berros Creek are similar to those of Diablo Creek; Los Berros is the nearest USGS gauging station in the vicinity of DCPP site. The records are good and unregulated. It is in the same hydrographic drainage area as the plant site and both drainage areas have relatively similar elevations. Geologic map comparison shows similarity of ground conditions. Isohyetal maps of major storms show similar magnitude of rainfall in both areas.

In the rainfall-runoff optimization fit using rainfall at DCPP site, the Los Berros recorded runoff responded well to the rainfall distribution at Diablo Canyon. Other rainfall stations around the gauging station were tried but no better fit could be derived than the above. On the foregoing consideration, the optimized loss rates are judged to be representative of the Diablo Canyon drainage basin.

The antecedent condition for the storm of January 24-25, 1969, was very favorable to heavy runoff. Heavy rains during the period of January 18-22, 1969, brought widespread but generally moderate flooding in the area. According to flood reports from USGS, this rain saturated the soil over much of the area. The time distribution of precipitation during the January 24-25 storm was conducive to rapid and intense runoff, because the heaviest rain occurred near the end of the storm when streams were already carrying large flood flows.

Choice of the January 24-25, 1969, storm gave, therefore, conservative results of loss rates. Precipitation data indicate that January 1969 was the wettest January in many years in the area.

As stated in Section 2.4.2.2, Hydrosphere, the average discharge at Diablo Creek is 0.5 cfs in its 16 years of record. However, base flow considerations were taken from the hydrograph of flood flow at Los Berros. The result of the optimization study is shown in Figure 2.4-4.

2.4.4.3 Runoff Model

Based on the discussion in the preceding section, the hydrologic response characteristics of Diablo Creek were considered as those that were optimized. The time of concentration of the Diablo Creek basin was calculated using the formula of the Bureau of Reclamation, Design of Small Dams, where:

$$T_c = \frac{\left(11.9L^3\right)^{0.385}}{H} \qquad (2.4\text{-}1)$$

where:

T_c = *time of concentration in hours*

L = *length of longest water course in miles*

H = *elevation difference in feet*

Due to the small size of the basins, Variables 2 and 3 in the rainfall-runoff study were taken as the optimized values. The definitions of the variables or parameters in the optimized model are shown in Sheet 3 of Figure 2.4-5. The first three variables represent unit hydrograph parameters.

The mechanics of the mathematical model used in this study are described in the program documentation of the "Unit Hydrograph and Loss Rate Optimization" computer program of the U.S. Army Corps of Engineers.

Based on the mechanics of this program, PG&E developed the computer program listed as Reference 8. The parameters obtained and defined in the optimization, or other values considered, are held constant and considered representative of the basin. No optimization is performed. This model is capable of modeling any basin rainfall amount and time distribution up to and including the PMP. Loss rates are also calculated in a nonlinear function represented by the equation:

$$L = K P^E \qquad (2.4\text{-}2)$$

where:

L = loss for each period
K = a function of four variables (average value and initial loss increment, which differ from flood to flood, and recovery rate and exponential recession rate, which are uniform for all floods)
P = rain for each period
E = loss rate variable equal to Variable 7 in the program

2.4.4.4 Probable Maximum Flood Flow

The PMP estimate obtained in Section 2.4.4.1 was distributed according to Reference 6. The loss rate parameters obtained in Section 2.4.4.2 were reduced by 50 percent to represent a much more severe antecedent condition and loss rate recession. The exponent of the loss rate equation (Variable 7) was not changed, but it was considered as an optimized regional value. Using the foregoing values as input, the synthetic PMF hydrograph for Diablo Creek up to the ocean side was derived with the aid of the PG&E computer program, Reference 8. The unit hydrograph constants were those that were derived in the runoff model. The hydrograph of inflow for the PMF is presented as a computer printout in Figure 2.4-5, Sheet 2. The peak flow for the PMF was found to be 6878 cfs (1325 cfs/sq mi) with a runoff factor of 0.92.

The switchyard embankment creates a dam upstream of the plant with a potential reservoir storage capacity of 1100 acre-feet. The possibility exists that this small reservoir is full prior to a PMF as a result of culvert plugging. Therefore, storage attenuation of inflow PMF was not considered.

Section 2.4.11 discusses the capability of roof and yard drainage to handle runoff from local PMP without risk of flooding PG&E Design Class I buildings.

2.4.4.5 Water Level Determinations

Figure 2.4-3 shows that the hydraulic capacity of the canyon is in excess of 10,000 cfs. There is more than 11 feet of freeboard if the road crossing is washed out and more than 7 feet of freeboard if the road crossing remains intact; thus, there is no risk of flood to PG&E Design Class I equipment.

2.4.4.6 Coincident Wind Wave Activity

Wave runup, discussed in Section 2.4.6, coincident with PMF will have little effect on computed water surfaces. The roadway acting as a weir at an elevation of 65 feet above MLLW (refer to Figure 2.4-3) provides higher backwaters than the combined waves discussed in Sections 2.4.6 and 2.4.7.

2.4.5 POTENTIAL DAM FAILURES (SEISMICALLY INDUCED)

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

There are no dams in the watershed and failure of dams outside the watershed could not generate sea waves higher than those discussed in Sections 2.4.6 and 2.4.7. The potential storage of water upstream of the switchyard fill described in Section 2.4.4.4 poses no flood threat since the switchyard fill is more than five times as wide as it is deep and the maximum storage of 1100 acre-feet has a face depth of 120 feet.

2.4.6 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

2.4.6.1 Probable Maximum Winds and Associated Meteorological Parameters

Hurricanes or line squalls of sufficient magnitude to generate surge flooding (storm-generated, long-period sea waves) have not been recorded on the Pacific coastline. This lack of observed events in 200 years of record lends reasonable assurance that such an event will not occur during the lifetime of the power plant. However, the effects of wind-generated storm waves, storm surge, and tides are conservatively considered in the evaluation of water level and its effects on PG&E Design Class I equipment and structures.

2.4.6.2 Surge and Seiche History

As discussed above, there is no record of surge flooding associated with hurricanes or line squalls. The history of short-period wave trains generated from remote storms in this region is limited. As described below, to compensate for the lack of historical knowledge, conservative flood levels have been developed on the basis of hindcasts and three-dimensional model testing.

2.4.6.3 Surge and Seiche Sources

Since there is no record of hurricanes, cyclonic type wind storms, squall lines, etc., on the Pacific Coast, these phenomena are not a design consideration. However, design for any credible flooding, including tsunami in combination with wave and tide action as discussed in Section 2.4.7, is conservatively considered.

2.4.6.4 Wave Action

Wave action behavior at DCPP was originally developed on the basis of hindcasts based on a statistical evaluation of historical data in combination with previous scale model testing. PG&E conducted an extensive review of the historical data that led to the estimation of the return periods of the critical storms; e.g., the 1905 storm and the 1981 storm. A major Pacific storm in January 1981 resulted in extensive damage to the west breakwater protecting the intake basin, and led to a review of all the design waves and water levels.

As a result of the damage, PG&E undertook a test program to determine critical wave behavior at the intake basin, including wave height, wave direction, wave runup, resulting forces, and the effects of wave splash on the intake structure and the auxiliary saltwater (ASW) system. A three-dimensional physical model of the basin and its surroundings was constructed, representing in a 1:45 scale the sea floor, the intake structure, and the breakwaters in storm-induced damage conditions.

The tests included the effects of: (a) wind-generated storm waves, including storm surge and tides, and (b) the effects of tsunami plus storm waves. The effects of the waves, including the wave heights, are discussed in detail in Section 2.4.7.

Because data related to wind-generated storm waves were very limited, PG&E developed and implemented a test program to generate the required data (Reference 16). The test program developed site-specific design basis flood events (References 16, 20).

Although the maximum still water level of 17 feet, for probable maximum tsunami, high tide, and storm surge, was conservatively used in the scale model tests (References 16 and 20), the still water level of 15.5 feet, as approved by the NRC, may be used (Reference 28).

Waves for the scale model tests were mechanically generated. Wave heights, outside the breakwater, of up to 45 feet, with periods of 12, 16, and 20 seconds were generated. The results for the model testing indicated that the response waves within the intake basin reached a maximum height that did not increase further in response to increases in the offshore wave height. This phenomenon is due to the effects of the natural terrain and the presence of the degraded breakwater. Therefore, the maximum credible wave event is based on the maximum response of the wave height within the basin, in combination with the still water level in the basin, and is used for assessing the maximum inundating effects and wave forces at the intake structure.

A wave data buoy was installed immediately off DCPP in May 1983 to directly obtain data on wave action. The data are recorded on site and telemetered to the Scripps Institute at La Jolla, California, where they are assimilated with data from other Pacific Coast buoys interconnected with the Scripps "Coastal Data Information Program."

2.4.6.5 Resonance/Ponding

As discussed in Section 2.4.6.4, PG&E developed and implemented a test program to simulate the effects of storm waves and tsunamis on the intake basin. The scale model included the detailed relief of the surrounding submerged terrain, the breakwaters, and the intake structure. The action of the waves on the scale model automatically incorporates the resonance and ponding effects of the intake basin.

2.4.6.6 Runup and Drawdown

Estimates of storm and tsunami wave runup and drawdown, and their effects on the plant, are presented in Section 2.4.7.

2.4.6.7 Protective Structures

The only PG&E Design Class I system that has components within the projected sea wave zone is the ASW system. The ASW pump motors are housed in watertight compartments within the intake structure. These compartments are designed for a combination tsunami-storm wave activity to elevation +48 feet MLLW (+45.4 feet MSL). The massive concrete intake structure ensures that the pumps remain in place and operate during extreme wave events. The intake structure is arranged to provide redundant paths for seawater to the pumps, ensuring a dependable supply of seawater.

In addition to the ASW pumps, the buried ASW piping outside of the intake structure, which is not attached to the circulating water tunnels, is vulnerable to the effects of tsunami and storm waves. An evaluation was conducted by Bechtel Corporation for PG&E to determine what protective measures were required to protect this buried ASW piping. This evaluation is described in Reference 40. Based on this evaluation, erosion protection, consisting of gabion mattresses, reinforced concrete pavement above this buried piping, and an armored embankment southeast of the intake structure, were designed and installed to resist the effects of tsunami and storm waves.

The model test program (References 16, 20) and resultant evaluations led to various structural modifications, including the extension of the ASW air vent structures with steel tubular snorkels having openings between elevations 48 and 52 feet MLLW. The snorkels were installed during 1982 and 1983 plant modifications. Analysis of the installed extensions by P. J. Ryan (Reference 18) further demonstrated that ingestion of sufficient water by the snorkels is extremely unlikely to jeopardize the operation of the ASW pumps. Section 2.4.7.6 provides additional details.

2.4.7 PROBABLE MAXIMUM TSUNAMI FLOODING

The tsunami evaluation and design have evolved as a result of a number of studies and analyses during the original plant design period, the operating license review period, and following the breakwater damage in 1981. The licensing basis for tsunami evaluation is presented in Sections 2.4.7.1 to 2.4.7.6. The background and evolution of the tsunami design and evaluation are provided in Section 2.4.7.7.

2.4.7.1 Probable Maximum Tsunami

Tsunamis are classified according to the distance from the shore to the location of the event (generator) that causes the wave. The design tsunami for DCPP represents the envelope of the following two classes of tsunamis:

> Distantly-generated tsunami: a tsunami whose generator is located more than several times the principal source dimension (e.g., length of postulated fault rupture) from the plant, Marine Advisors, Inc., 1966 (Reference 24)

Locally-generated (near-shore) tsunami: a tsunami whose generator is closer than the distance defined for distantly-generated tsunami

The tsunami runup and drawdown at the intake structure are dependent on the source of the tsunami, the distance to the tsunami generator, and the near-shore undersea terrain, including the topography of the intake basin and the configuration of the breakwater.

Wave heights for the two classes of tsunamis considered in the design of DCPP are described in the following sections.

2.4.7.1.1 Distantly-Generated Tsunamis

The predominant sources of distantly-generated tsunamis are limited to areas of earthquake and volcanic activity on the circum-Pacific belt. Distant sources relative to DCPP include the Aleutian area, the Kuril-Kamchatka region, and the South American coast.

The lack of historical data for the site during the construction permit review raised a question on the degree of confidence for a “virtually no risk of being exceeded” assurance. In 1967, the AEC staff and its consultants, the United States Coast and Geodetic Survey (USCGS), agreed that the probable maximum tsunami at the site, which had virtually no risk of being exceeded, would be less than the 17- to 20- foot waves experienced at Crescent City, California, as a result of the 1964 Anchorage, Alaska, earthquake (Reference 35). To expedite the permit schedule, PG&E decided to use 20 feet as the maximum distantly-generated tsunami wave height.

2.4.7.1.2 Near-Shore Tsunami

A number of investigations and analyses to determine the tsunami-generation potential of near-shore earthquake faults were performed during the period from 1966 to 1975. The design basis tsunami wave heights are based on the analysis performed in 1975 by Hwang, Yuen, and Brandsma (Reference 28). The following earthquake sources and characteristics were considered in the analysis:

- Santa Lucia Bank fault, located approximately 29 miles from the site, considering a resultant displacement of 9.8 feet and a vertical displacement (6.6 feet) equal to 2/3 of the resultant displacement

- Santa Maria Basin fault (later identified as the Hosgri fault), located approximately 3.5 miles from the site, considering a resultant displacement of 11 feet and a vertical displacement (7.3 feet) equal to 2/3 of the resultant displacement

The analysis considered the cases of the breakwaters (a) present as originally constructed, (b) completely absent, and (c) in damaged conditions, in which the sides of the breakwaters slump to a 1-on-4, 1-on-5, or 1-on-6 vertical-to-horizontal slope.

The Santa Maria Basin fault source controls, producing a maximum runup of 9.2 feet and a maximum drawdown of 0.0 feet (Reference 28).

The design basis maximum combined wave runup is the greater of that determined for near-shore or distantly-generated tsunamis, and results from near-shore tsunamis. The bases of these runup values are given in the following two subsections.

- For distantly-generated tsunamis, the combined runup is 30 feet
- For near-shore tsunamis, the combined wave runup is 34.6 feet, as determined by hydraulic model testing (References 21 and 37)

2.4.7.1.3 Combined Wave Runup for Distantly-Generated Tsunamis

The combined wave runup for distantly-generated tsunamis is the same as the value adopted during the construction permit review. The value adopted at that time was 30 feet, as imposed by the NRC (Reference 35).

2.4.7.1.4 Combined Wave Runup for Near-Shore Tsunamis

The combined wave runup for near-shore tsunamis, 34.6 feet, is based on observations during scale model testing (Reference 21), which was performed subsequent to the 1981 breakwater damage. This runup value represents the maximum runup observed at the location of the ventilation shafts in the test model, excluding wave spray. Wave splash and spray, which can extend to higher elevations, are discussed in Section 2.4.7.6.

A degraded breakwater model was used, representing the crest of both breakwaters reduced to MLLW, the seaward slopes below that level remaining as originally constructed, and the intake basin sides widened by as much as the material above MLLW could achieve while coming to rest at a slope of 1 vertical to 1.5 horizontal. The model represents the worst-case breakwater damage that could result from the cumulative effects of severe storms, a tsunami, and Hosgri effects (References 23 and 33).

Tsunami, storm surge, and tide effects have relatively long periods and were combined to represent a static change in the elevation of the still water surface. The dynamic effects of storm waves, which have shorter periods, were then superimposed.

2.4.7.1.5 Combined Wave Drawdown Minimum Water Level

The maximum combined wave drawdown is the greater of that determined for near-shore or distantly-generated tsunamis, and results from distantly-generated tsunamis. This value constitutes the design combined drawdown value, which is 9.0 feet.

- Combined wave drawdown for distantly-generated tsunamis: The combined wave drawdown value of 9 feet, derived by a study performed during the construction permit review, is based on the combination of tsunami, storm wave, storm surge, and tide (Reference 24).

- Combined wave drawdown for near-shore tsunamis: The maximum combined wave drawdown determined by analysis for the case with the breakwaters intact, as originally constructed, is 4.07 feet (Reference 28). The maximum combined drawdown for the case with the breakwater degraded to MLLW has not been evaluated. However, analysis for the case of no breakwater present shows that the drawdown effect is 4.40 feet (Reference 28). Therefore, the drawdown for near-shore tsunamis will be less than for distantly-generated tsunamis. There is a significant margin between the 4.07 feet of drawdown and the available pump submergence depth.

2.4.7.2 Historical Tsunami Record

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

There is no historical record of tsunamis for DCPP site due to the remote location with respect to populated areas. The historical review of the region shows tsunamis that have been recorded in the region are of the same order of magnitude as the normal tide range and that local configurations play a large part in the ultimate effects of the tsunami.

At the California coast, reactions to tsunamis from distant sources have been generally moderate, with the exception of certain sensitive areas that have historically shown an abnormally high response as compared to the coast in general. Avila Beach is the closest sensitive area to DCPP.

A review of historical tsunami records and studies of the underwater topography has determined that wave heights recorded at Avila Beach are the result of local conditions that do not affect DCPP (Reference 24). The review demonstrated that DCPP need consider only a distantly-generated tsunami height of 5.0 to 6.0 feet, corresponding to the normal tidal range. Thus, a 6-foot change in the water level above or below MLLW could result (Reference 24). Hence, the 20-foot tsunami runup from a distantly-generated tsunami suggested by the USCGS (Reference 32) is extremely conservative.

2.4.7.3 Source of Tsunami Wave Height

2.4.7.3.1 Distantly-Generated Tsunamis

As discussed in Section 2.4.7.1.1, the predominant sources of distantly-generated tsunamis are limited to areas of earthquake and volcanic activity on the circum-Pacific belt. Distant sources relative to DCPP include the Aleutian area, the Kuril-Kamchatka region, and the South American coast.

2.4.7.3.2 Near-Shore Tsunamis

A number of investigations and analyses to determine the tsunami-generation potential of near-shore earthquake faults was performed during the period from 1966 to 1975. The following earthquake sources and characteristics were considered in the analyses:

- Santa Lucia Bank fault, located approximately 29 miles from the site, considering a resultant displacement of 9.8 feet and a vertical displacement (6.6 feet) equal to 2/3 of the resultant displacement

- Santa Maria Basin fault (later identified as the Hosgri fault), located approximately 3.5 miles from the site, considering a resultant displacement of 11 feet and a vertical displacement (7.3 feet) equal to 2/3 of the resultant displacement

The design basis tsunami wave heights are based on the analysis performed in 1975 by Hwang, Yuen, and Brandsma of Tetra Tech, Inc. (Reference 28).

2.4.7.4 Tsunami Height Offshore

Estimates of tsunami heights from distant generators offshore are postulated to have dissipated to wave trains with heights on the order of astronomical tidal range of 6 feet. Locally-generated tsunami runup heights from seismic activity or from submarine landslides are estimated to be a maximum of 9.2 feet (Reference 28).

2.4.7.5 Hydrography and Harbor or Breakwater Influences on Tsunami

Since the approach to the intake structure is across very irregular submerged terrain, PG&E decided after the January 1981 storm, which significantly damaged the breakwater, that the wave behavior under both extreme tide and tsunami condition would most reliably be evaluated through the use of a three-dimensional physical scale model. The effects of the intake basin, natural sea floor, and the breakwaters (in the damaged state) were considered in the testing and evaluation. Resonance and ponding effects are automatically incorporated by the model testing.

The 80- by 120-foot, 1:45 scale model was designed and constructed on the basis of detailed surveys and soundings. Wave-making machines were positioned at various parts of the basin to drive waves of defined heights, periods, and directions toward the intake basin. Appropriate instrumentation was included to measure and record wave characteristics, and to measure and record critical forces and loads on the intake structure (References 16 and 20).

2.4.7.6 Effects on PG&E Design Class I Facilities

The only PG&E Design Class I system that has components within the projected sea wave zone is the ASW system. The intake structure, within which this equipment is housed, has a main deck elevation of +20 feet above MLLW; it will withstand a tsunami coincident with high tide and depth-limited maximum storm waves that can occur within the intake basin. The PG&E Design Class I equipment is installed in watertight compartments to protect it from adverse sea wave events to elevation +48 feet above MLLW.

In addition to the ASW pumps, the buried ASW piping outside of the intake structure, which is not attached to the circulating water tunnels, is vulnerable to the effects of tsunami and storm waves. An evaluation was conducted by Bechtel Corporation for PG&E to determine what protective measures were required to protect this buried ASW piping. This evaluation is described in Reference 40. Based on this evaluation, erosion protection, consisting of gabion mattresses, reinforced concrete pavement above this buried piping, and an armored embankment southeast of the intake structure, were designed and installed to resist the effects of tsunami and storm waves.

The ability of the breakwater to resist damage to the intake structure caused by collisions of marine vessels was demonstrated by Kircher et al. (Reference 41) as described in Section 2.2.3.1. The structural integrity of the intake structure to resist extreme wave attack (design flood event) in the unlikely event of degradation of the breakwater was reviewed by model tests conducted by O. J. Lillevang (Reference 16) and Dr. Fredric Raichlen (Reference 20). Data from the model study were used by E. N. Matsuda (Reference 21) to structurally analyze the ability of the intake structure to resist the most extreme wave forces. Matsuda determined that, with minor modifications, the intake structure would not be structurally damaged by the most extreme wave forces that might occur even in the unlikely event the entire breakwater were to be degraded to zero feet MLLW. The modifications were completed in 1983.

In addition to the structural evaluations discussed above, the potential effects of splash and spray of the sea waves on PG&E Design Class I equipment were evaluated. Splashing of water up to and above the top of the ventilation shaft (52 feet MLLW) for the ASW pump rooms was observed during the performance of the scale model testing (Reference 16). The testing demonstrated that the ventilation shaft extensions remained free of the upward splashed water as they are set back from the seaward edge of the concrete vent huts at a considerable distance from the seaward edge of the intake structure, and the openings face away from the sea.

Although the air intake would not be inundated by splashing of water, it could be subject to windborne spray. This spray could potentially wet the vent openings and enter the ASW pump rooms. As described in the following subsections, testing and analysis showed that it is not credible that the water level in a pump room would exceed the maximum design flood level for the room.

Additional tests, using the 1:45 scale model of the intake structure and intake basin, were performed by Offshore Technology Corporation to determine the potential for ingestion of water by the ASW pump room ventilation shafts (Reference 30). Wave splash behavior in the vicinity of the ventilation shafts was recorded using high-speed motion pictures, still photography, and visual observation. Subsequent to the testing, analyses were conducted to evaluate the effect of the splashing on the ASW pumps (Reference 18). The conclusion of this analysis was that the combination of degraded breakwater, tsunami, high tide, severe storm, and extreme winds in the offshore direction necessary to result in a critical volume of water being ingested is not credible (Reference 18).

The ASW pumps are protected against flooding for the maximum wave height under tsunami and storm wave conditions even if the entire length of the breakwater were degraded to MLLW. Since there is no assurance that the breakwater would not degrade below MLLW, even though Wiegel (Reference 33) indicates that this is very unlikely, the DCPP Equipment Control Guidelines (Reference 29) include requirements to monitor the condition of the breakwater, to implement corrective action when limited damage is sustained, and to identify the limiting condition for operation relative to the configuration of the breakwaters.

2.4.7.7 Background and Evolution of the Tsunami Design Basis

The background and evolution of the tsunami design basis have been documented in detail in NRC Supplemental Safety Evaluation Reports (SSERs) 1, 5, 7, 13, and 17.

2.4.8 ICE FLOODING

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

As described in Section 2.3, the mild climate and general lack of freezing temperatures in this region make regional ice formation highly unlikely, and it was, therefore, not considered.

2.4.9 COOLING WATER CANALS AND RESERVOIRS

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

The Pacific Ocean is the source of cooling water for the plant. This cooling water system contains no canals or reservoirs.

2.4.10 CHANNEL DIVERSIONS

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

Upstream diversions associated with rivers, where low flow has an impact on dependable cooling water sources, is not a factor for this site.

2.4.11 FLOODING PROTECTION REQUIREMENTS

The site arrangement, with the plant situated on a coastal terrace 85 feet above MSL, virtually eliminates all risks from flooding.

Roofs of PG&E Design Class I buildings have a drainage system designed in accordance with the Uniform Plumbing Code for an adjusted regional PMP of 4 inches/hour. In addition, overflow scuppers are provided in parapet walls at roof level to prevent ponding of accumulated rainwater in excess of drain capacity. Yard areas around PG&E Design Class I buildings are graded to provide positive slope away from buildings. Storm runoff is overland and unobstructed. It is, therefore, not possible for ponding from local PMP to flood PG&E Design Class I buildings.

2.4.12 LOW WATER CONSIDERATIONS

2.4.12.1 Low Flow in Rivers and Streams

There are no rivers or streams involved in plant operations; therefore, low flow conditions were not evaluated.

2.4.12.2 Low Water Resulting from Surges, Seiches, or Tsunamis

Low water, as a result of tsunami drawdown occurring coincident with low tide and short-period storm waves, is projected by Marine Advisers (Reference 24) to result in a possible low water elevation of 9 feet below MLLW.

2.4.12.3 Historical Low Water

As discussed in Section 2.4.7.2, there is no historical record for the site. Regional ocean low water history is reported in Reference 24.

2.4.12.4 Future Control

Flowrate factors generally associated with plants situated on rivers are not applicable to DCPP.

2.4.12.5 Plant Requirements

The only PG&E Design Class I system impacted by tsunami drawdown is the ASW. To ensure adequate water supply to the ASW system in the event a tsunami downsurge occurs, the arrangement of the intake structure provides free access to the ocean. In the event of a low water elevation of 9 feet below MLLW, each ASW pump will provide approximately 85 percent of the design flow due to increased static head losses (while operating in the one-pump one-heat exchanger alignment) (refer to Section 9.2.7.3.1). This is a temporary condition and would not result in a significant increase in component cooling water (CCW) temperature.

2.4.12.6 Heat Sink Dependability Requirements

The ASW pumps are designed to operate with the water level down to 17.4 feet below MLLW, substantially below the minimum water level of 9 feet below MLLW that might occur during a tsunami. Therefore, operation of the ASW system would not be interrupted by low water levels.

Cavitation (with the potential to significantly reduce system flow) is predicted to occur when operating with one ASW pump supplying two CCW heat exchangers during a tsunami drawdown. In the event a tsunami is indicated (by a tsunami warning or a severe earthquake) with two CCW heat exchangers in service, a loss of suction would be indicated by low ASW pump discharge pressure and/or low CCW heat exchanger differential pressure (D/P), low ASW bay level, or fluctuating pump motor current. Operator action would be required to remove one of the CCW heat exchangers from service to reduce system flow and decrease pump suction head requirements.

2.4.13 ENVIRONMENTAL ACCEPTANCE OF EFFLUENTS

Deep Well 0-2 is the source for groundwater for use at the DCPP site only, and there is no public use of this groundwater (as discussed in Section 2.4.14). No other significant groundwater source exists in this area. No detailed analysis of acceptance of effluents by surface or groundwater is relevant. The releases to the environment via the discharge canal are described in Chapter 11.

Estimated releases of activity from the liquid waste system are discussed in Section 11.2.6, and dilution factors for dilution of liquid wastes are discussed in Section 11.2.8. The release points for liquid waste are shown in Figure 11.2-9. A flow diagram for the design basis case for liquid waste processing is shown in Figure 11.2-2. The numbered waste input streams have their annual flow and isotopic spectra listed in Tables 11.2-3 and 11.2-5. The numbered process streams are listed in Tables 11.2-8 and 11.2-9, with flows and isotopic concentrations.

The possibility of accidental releases and the consequent dispersion of such releases are discussed in Chapter 15. Because of the location of the plant on the ocean and the separation of intake and discharge structures, insignificant recirculation occurs.

2.4.14 GROUNDWATER

2.4.14.1 Description and Onsite Use

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

Groundwater at the site is limited to Deep Well 0-2. No other significant groundwater has been encountered. Three small springs were encountered during excavation for plant construction; two of these were wet spots and the third had a flow of less than thirty gallons per minute. The water was analyzed and found to be very hard (1050 mg/l $CaCO_3$ and high in dissolved residue (2148 mg/l). Groundwater and domestic water supplies are not affected by the operation of the plant. (Draft Environmental Statement of the Directorate of Licensing, United States Atomic Energy Commission, December 1972.) There is no public use of onsite groundwater.

2.4.14.2 Monitoring and Safeguard Requirements

Process and effluent streams are monitored wherever a potential release of radioactivity exists during all modes of plant operation.

Differential temperature across the condenser is monitored as a condition of the national pollution discharge elimination system (NPDES) permit.

2.4.15 TECHNICAL SPECIFICATIONS AND EMERGENCY OPERATION REQUIREMENTS

Technical Specifications that describe the safe operation or shutdown requirements for the plant are contained in Appendix A to the operating license.

2.4.16 SAFETY EVALUATION

2.4.16.1 General Design Criterion 2, 1967 – Performance Standards

The PG&E Design Class I structures, systems, and components essential to the prevention of accidents or to mitigate their consequences are designed to withstand or are protected from the effects of flooding. Refer to Sections 2.4.3.2.1, 2.4.3.2.2, 2.4.6.7, 2.4.11, 2.4.12.1, 2.4.12.4, 2.4.13, 2.4.14.1, and 2.4.14.2.

2.4.16.2 Regulatory Guide 1.59, Revision 2, August 1977 – Design Basis Floods for Nuclear Power Plants

The PG&E Design Class I structures, systems, and components are designed to withstand and continue to perform their function during the worst site-related flood probable to occur. Refer to Sections 2.4.3.2.2, 2.4.3.2.3, 2.4.6.7, 2.4.7, 2.4.7.1, 2.4.7.1.1, 2.4.7.1.2, 2.4.7.1.3, 2.4.7.1.4, 2.4.7.1.5, 2.4.7.3.1, 2.4.7.3.2, 2.4.7.4, 2.4.7.6, 2.4.12.2, 2.4.12.3, 2.4.12.5, and 2.4.12.6.

2.4.16.3 Regulatory Guide 1.102, Revision 1, September 1976 – Flood Protection for Nuclear Power Plants

The PG&E Design Class I structures, systems, and components are appropriately protected from damage caused by flooding. Refer to Sections 2.4.3.2.3, 2.4.6.7, 2.4.7.6, and 2.4.12.6.

2.4.16.4 Regulatory Guide 1.125, Revision 1, October 1978 – Physical Models for Design and Operation of Hydraulic Structures and Systems for Nuclear Power Plants

Hydraulic modeling of the site intake breakwaters, systems, and structures is appropriately designed, verified, tested, and documented to accurately describe the behavior of these plant facilities. Refer to Sections 2.4.3.2.3, 2.4.6.7, 2.4.7.14, 2.4.7.5, and 2.4.7.6.

2.4.17 REFERENCES

1. Joint Feasibility Report by the State of California Department of Water Resources and the United States Department of the Interior, Office of Saline Water, 1972.

2. A. O. Waananen, Open File Report, Water Resources Division, Geological Survey, U.S. Department of the Interior, 1977.

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4. Floods of January and February 1969 in Southern California, Los Angeles District, Corps of Engineers, Department of the Army, 1969.

5. PG&E, Ocean Wave History, Appendix E, of the Preliminary Safety Analysis Report (PSAR) for Nuclear Unit No. 2, San Francisco, California, 1967.

6. United States Weather Bureau (USWB), Hydrometeorological Report (HMR) No. 36, Interim Report - Probable Maximum Precipitation in California, and modification thereto suggested in Revisions of October 1969 to Hydrometeorological Report No. 36, Interim Report - Probable Maximum Precipitation in California, 1969.

7. L. R. Beard, Optimization Techniques for Hydrologic Engineering, U.S. Army Corps of Engineers Technical Paper No. 2, 1966.

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9. Deleted

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16. O. J. Lillevang, et al., The Height Limiting Effect of Sea Floor Terrain Features and of Hypothetically Extensively Reduced Breakwaters on Wave Action at Diablo Canyon Sea Water Intake, California, 1982.

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20. F. Raichlen, The Investigation of Wave-Structure Interactions for the Cooling Water Intake Structure of the Diablo Canyon Nuclear Power Plant, California, 1982.

21. E. N. Matsuda, Wave Effects on the Intake Structure, DCPP, California, 1983.

22. O. J. Lillevang, Letter/Report dated May 20, 1982, to R. V. Bettinger.

23. H. Bolton Seed, Letter/Report dated September 22, 1981, to R. V. Bettinger.

24. An Evaluation of Tsunami Potential at the Diablo Canyon Site, Marine Advisers, Inc., Report A-253, 1966. (Appendix E of the PSAR).

25. O. J. Lillevang, A Basin Intake for Cooling Water at Diablo Canyon Power Plant, 1969. (Appendix 2.4A to Diablo Canyon Power Plant Final Safety Analysis Report as amended through August 1980). (See also Reference 27 of Section 2.3.)

26. Deleted

27. Li-San Hwang, et al., Earthquake Generated Water Waves at the Diablo Canyon Power Plant, 1974. (Appendix D of Appendix 2.4C to Diablo Canyon Power Plant Final Safety Analysis Report as amended through August 1980). (See also Reference 27 of Section 2.3.)

28. Li-San Hwang, et al., Earthquake Generated Water Waves at the Diablo Canyon Power Plant, (Part Two), 1975. (Appendix E of Appendix 2.4C to Diablo Canyon Power Plant Final Safety Analysis Report as amended through August 1980). (See also Reference 27 of Section 2.3.)

29. DCPP Equipment Control Guideline 17.3, "Flood Protection," Pacific Gas and Electric Company.

30. J. I. Collins and W. G. Groskopf, Hydraulic Model Study of Diablo Canyon Intake Structure, Test Results - Ingestion Studies, OTC Corporation, 1983.

31. Deleted in Revision 22.

32. U. S. Coast and Geodetic Survey, Report on the Seismicity of the Nuclear Plant at the Diablo Canyon Site, September 1967.

33. R. L. Wiegel, Breakwater Damage by Severe Storm Waves and Tsunami Waves, March 5, 1982.

34. Hydraulic Model Study of Diablo Canyon Intake Structure Test Results, December 1982, OTC-82-42.

35. Department of Engineering Memorandum, "Meeting with AEC Staff and Consultants, November 21, 1967," Pacific Gas and Electric Company, December 4, 1967.

36. F. Raichlen, "Wave Induced Effects in a Cooling Water Basin," Chapter 196, Proceedings of International Coastal Engineering Conference, 1986.

37. PG&E Calculation No. 52.18.13.1, "Combined Runup Depths for Tsunami and Storm Waves," 1997.

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40. Diablo Canyon Power Plant - Auxiliary Saltwater Cooling System Erosion Protection for New Bypass Piping, Bechtel Corporation, October 1996.

41. C. A. Kircher, et al., Frequency of Vessel Impact with the Diablo Canyon Intake Structures, December 10, 1982.

42. Regulatory Guide 1.59, Revision 2, Design Basis Floods for Nuclear Power Plants, USNRC, August 1977.

43. Regulatory Guide 1.125, Revision 1, Physical Models for Design and Operation of Hydraulic Structures and Systems for Nuclear Power Plants, USNRC, October 1978.

2.4.18 REFERENCE DRAWINGS

Figures representing controlled engineering drawings are incorporated by reference and are identified in Table 1.6-1. The contents of the drawings are controlled by DCPP procedures.

2.5 GEOLOGY AND SEISMOLOGY

This section presents the findings of the regional and site-specific geologic and seismologic investigations of the Diablo Canyon Power Plant (DCPP) site. Information presented is in compliance with the criteria in Appendix A of 10 CFR Part 100, as described below, and meets the format and content recommendations of Regulatory Guide 1.70, Revision 1 (Reference 39). Because the development of the seismic inputs for DCPP predates the issuance of 10 CFR Part 100, Appendix A, "Seismic and Geologic Siting Criteria for Nuclear Power Plants," the DCPP earthquakes are plant specific.

To capture the historical progress of the geotechnical and seismological investigations associated with the DCPP site, information pertaining to the following three time periods is described herein:

(1) Original Design Phase: investigations performed in support of the Preliminary Safety Analysis Report, prior to the issuance of the Unit 1 construction permit (1967), through the early stages of the construction of Unit 1 (1971). The Design Earthquake and Double Design Earthquake ground motions are associated with this phase. These earthquakes are similar to the regulatory ground motion level that the NRC subsequently developed in 10 CFR Part 100 Appendix A as the "Operating Basis Earthquake (OBE)" ground motion and the "Safe Shutdown Earthquake (SSE)" ground motion, respectively.

(2) Hosgri Evaluation Phase: investigations performed in response to the identification of the offshore Hosgri fault zone (1971) through the issuance of the Unit 1 operating license (1984). The 1977 Hosgri Earthquake ground motions are associated with this phase. The Hosgri Evaluation Phase does not affect or change the investigations and conclusions of the Original Design Phase.

(3) Long Term Seismic Program (LTSP) Evaluation Phase: investigations performed in response to the License Condition Item No. 2.C.(7) of the Unit 1 operating license (1984) through the removal of the License Condition (1991), including current on-going investigations. The 1991 L TSP ground motion is associated with this phase. The LTSP Evaluation Phase does not affect or change the investigations and conclusions of either the Original Design Phase or the Hosgri Evaluation Phase.

<u>Overview</u>

Locations of earthquake epicenters within 200 miles of the plant site, and faults and earthquake epicenters within 75 miles of the plant site for either magnitudes or intensities, respectively, are shown in Figures 2.5-2, 2.5-3, and 2.5-4 (through 1972). A geologic and tectonic map of the region surrounding the site is shown in Figure 2.5-5, and detailed information about site geology is presented in Figures 2.5-8 through 2.5-16. Geology and seismology are discussed in detail in Sections 2.5.2 through 2.5.5. Additional information on site geology is contained in References 1 and 2.

Detailed supporting data pertaining to this section are presented in Appendices 2.5A, 2.5B, 2.5C, and 2.5D of Reference 27 in Section 2.3. Geologic and seismic information from investigations that responded to Nuclear Regulatory Commission (NRC) licensing review questions are presented Appendices 2.5E and 2.5F of the same reference. A brief synopsis of the information presented in Reference 27 (Section 2.3) is given below. The DCPP site is located in San Luis Obispo County approximately 190 miles south of San Francisco and 150 miles northwest of Los Angeles, California. It is adjacent to the Pacific Ocean, 12 miles west-southwest of the city of San Luis Obispo, the county seat. The plant site location and topography are shown in Figure 2.5-1.

The site is located near the mouth of Diablo Creek which flows out of the San Luis Range, the dominant feature to the northeast. The Pacific Ocean is southwest of the site. Facilities for the power plant are located on a marine terrace that is situated between the mountain range and the ocean.

The terrace is bedrock overlain by surficial deposits of marine and nonmarine origin. PG&E Design Class I structures at the site are situated on bedrock that is predominantly stratified marine sedimentary rocks and volcanics, all of Miocene age. A more extensive discussion of the regional geology is presented in Section 2.5.2.1 and site geology in Section 2.5.2.2.

Several investigations were performed at the site and in the vicinity of the site to determine: potential vibratory ground motion characteristics, existence of surface faulting, and stability of subsurface materials and cut slopes adjacent to Seismic Category I structures. Details of these investigations are presented in Sections 2.5.2 through 2.5.5. Consultants retained to perform these studies included: Earth Science Associates (geology and seismicity), John A. Blume and Associates (seismic design and foundation materials dynamic response), Harding-Lawson and Associates (stability of cut slope), Woodward-Clyde-Sherard and Associates (soil testing), and Geo-Recon, Incorporated (rock seismic velocity determinations). The findings of these consultants are summarized in this section and the detailed reports are included in Appendices 2.5A, 2.5B, 2.5C, 2.5D, 2.5E, and 2.5F of Reference 27 in Section 2.3.

Geologic investigation of the Diablo Canyon coastal area, including detailed mapping of all natural exposures and exploratory trenches, yielded the following basic conclusions:

(1) The area is underlain by sedimentary and volcanic bedrock units of Miocene age. Within this area, the power plant site is underlain almost wholly by sedimentary strata of the Monterey Formation, which dip northward at moderate to very steep angles. More specifically, the reactor site is underlain by thick-bedded to almost massive Monterey sandstone

that is well indurated and firm. Where exposed on the nearby hillslope, this rock is markedly resistant to erosion.

(2) The bedrock beneath the main terrace area, within which the power plant site has been located, is covered by 3 to 35 feet of surficial deposits. These include marine sediments of Pleistocene age and nonmarine sediments of Pleistocene and Holocene age. In general, they are thickest in the vicinity of the reactor site.

(3) The interface between the unconsolidated terrace deposits and the underlying bedrock comprises flat to moderately irregular surfaces of Pleistocene marine planation and intervening steeper slopes that also represent erosion in Pleistocene time.

(4) The bedrock beneath the power plant site occupies the southerly flank of a major syncline that trends west to northwest. No evidence of a major fault has been recognized within or near the coastal area, and bedrock relationships in the exploratory trenches positively indicate that no such fault is present within the area of the power plant site.

(5) Minor surfaces of disturbance, some of which plainly are faults, are present within the bedrock that underlies the power plant site. None of these breaks offsets the interface between bedrock and the cover of terrace deposits, and none of them extends upward into the surficial cover. Thus, the latest movements along these small faults must have antedated erosion of the bedrock section in Pleistocene time.

(6) No landslide masses or other gross expressions of ground instability are present within the power plant site or on the main hillslope east of the site. Some landslides have been identified in adjacent ground, but these are minor features confined to the naturally oversteepened walls of Diablo Canyon.

(7) No water of subsurface origin was encountered in the exploratory trenches, and the level of permanent groundwater beneath the main terrace area probably is little different from that of the adjacent lower reaches of the deeply incised Diablo Creek.

2.5.1. DESIGN BASIS

2.5.1.1 General Design Criterion 2, 1967 Performance Standards

DCPP systems, structures, and components have been located, designed and analyzed to withstand those forces that might result from the most severe natural earthquake phenomena.

2.5.1.2 License Condition 2.C(7) of DCPP Facility Operating License DPR-80 Rev. 44 (LTSP), Elements (1), (2) and (3)

DCPP developed and implemented a program to re-evaluate the seismic design bases used for the Diablo Canyon Power Plant.

The program included the following three Elements that were completed and accepted by the NRC (References 40, 41, and 43):

(1) The identification, examination, and evaluation of all relevant geologic and seismic data, information, and interpretations that have become available since the 1979 ASLB hearing in order to update the geology, seismology and tectonics in the region of the Diablo Canyon Nuclear Power Plant. If needed to define the earthquake potential of the region as it affects the Diablo Canyon Plant, PG&E has also re-evaluated the earlier information and acquired additional data.

(2) DCPP has re-evaluated the magnitude of the earthquakes used to determine the seismic basis of the Diablo Canyon Nuclear Plant using the information from Element 1.

(3) DCPP has re-evaluated the ground motion at the site based on the results obtained from Element 2 with full consideration of site and other relevant effects.

As a condition of the NRC's closeout of License Condition 2.C.(7), PG&E committed to several ongoing activities in support of the LTSP, as discussed in a public meeting between PG&E and the NRC on March 15, 1991 (Reference 53), described as the "Framework for the Future," in a letter to the NRC, dated April17, 1991 (Reference 50), and affirmed by the NRC in SSER 34 (Reference 43). These ongoing activities are discussed in Section 2.5.7.

2.5.1.3 10 CFR Part 100, March 1966- Reactor Site Criteria

During the determination of the location of the Diablo Canyon Power Plant, consideration was given to the physical characteristics of the site, including seismology and geology.

2.5.2 BASIC GEOLOGIC AND SEISMIC INFORMATION

This section presents the basic geologic and seismic information for DCPP site and surrounding region. Information contained herein has been obtained from literature studies, field investigations, and laboratory testing and is to be used as a basis for evaluations required to provide a safe design for the facility. The basic data contained in this section and in Reference 27 of Section 2.3 are referenced in several other sections of this FSAR Update. Additional information, developed during the Hosgri and LTSP evaluations, is described in Sections 2.5.3.9.3 and 2.5.3.9.4, respectively.

2.5.2.1 Regional Geology

2.5.2.1.1 Regional Physiography

Diablo Canyon is in the southern Coast Range which is a part of the California Coast Ranges section of the Pacific Border physiographic province (refer to Figure 2.5-1). The region surrounding the power plant site consists of mountains, foothills, marine terraces, and valleys. The dominant features are the San Luis Range adjacent to the site to the northeast, the Santa Lucia Range farther inland, the lowlands of the Los Osos and San Luis Obispo Valleys separating the San Luis and Santa Lucia Ranges, and the marine terrace along the coastal margin of the San Luis Range.

Landforms of the San Luis Range and the adjacent marine terrace produce the physiography at the site and in the region surrounding the site. The westerly end of the San Luis Range is a mass of rugged high ground that extends from San Luis Obispo Creek and San Luis Obispo Bay on the east and is bounded by the Pacific Ocean on the south and west. Except for its narrow fringe of coastal terraces, the range is featured by west-northwesterly-trending ridge and canyon topography. Ridge crest altitudes range from about 800 to 1800 feet. Nearly all of the slopes are steep, and they are modified locally by extensive slump and earthflow landslides.

Most of the canyons have narrow-bottomed, V-shaped cross sections. Alluvial fans and talus aprons are prominent features along the bases of many slopes and at localities where ravines debouch onto relatively gentle terrace surfaces. The coastal terrace belt extends between a steep mountain-front backscarp and a near-vertical sea cliff 40 to 200 feet in height. Both the bedrock benches of the terraces and the present offshore wave-cut bench are irregular in detail, with numerous basins and rock projections.

The main terrace along the coastal margin of the San Luis Range is a gently to moderately sloping strip of land as much as 2000 feet in maximum width. The more landward parts of its surface are defined by broad aprons of alluvial deposits. This cover thins progressively in a seaward direction and is absent altogether in a few places along the present sea cliff. The main terrace represents a series of at least three wave-cut rock benches that have approximate shoreline-angle elevations of 70, 100, and 120 feet.

Owing to both the prevailing seaward slopes of the rock surfaces and the variable thickness of overlying marine and nonmarine cover, the present surface of the main terrace ranges from 70 to more than 200 feet in elevation. Remnants of higher terraces exist at scattered locations along upper slopes and ridge crests. The most extensive among these is a series of terrace surfaces at altitudes of 300+, 400+, and 700+ feet at the west end of the ridge between Coon and Islay Creeks, north of Point Buchon. A surface described by Headlee (Reference 19) as a marine terrace at an altitude of about

700 feet forms the top of San Luis Hill. Remnants of a lower terrace at an altitude of 30 to 45 feet are preserved at the mouth of Diablo Canyon and at several places farther north.

Owing to contrasting resistance to erosion among the various bedrock units of the San Luis Range, the detailed topography of the wave-cut benches commonly is very irregular. As extreme examples, both modern and fossil sea stacks rise as much as 100 feet above the general levels of adjacent marine-eroded surfaces at several localities.

2.5.2.1.2 Regional Geologic and Tectonic Setting

2.5.2.1.2.1 Geologic Setting

The San Luis Range is underlain by a synclinal section of Tertiary sedimentary and volcanic rocks, which have been downfolded into a basement of Mesozoic rocks now exposed along its southwest and northeast sides. Two zones of faulting have been recognized within the range. The Edna fault zone trends along its northeast side, and the Miguelito fault zone extends into the range from the vicinity of Avila Bay. Minor faults and bedding-plane shears can be seen in the parts of the section that are well exposed along the sea cliff fringing the coastal terrace benches. None of these faults shows evidence of geologically recent activity, and the most recent movements along those in the rocks underlying the youngest coastal terraces can be positively dated as older than 80,000 to 120,000 years. Geologic and tectonic maps of the region surrounding the site are shown in Figures 2.5-5 (2 sheets), 2.5-6, 2.5-8, and 2.5-9.

2.5.2.1.2.2 Tectonic Features of the Central Coastal Region

DCPP site lies within the southern Coast Ranges structural province, and approximately upon the centerline axis of the northwest-trending block of crust that is bounded by the San Andreas fault on the northeast and the continental margin on the southwest. This crustal block is characterized by northwest-trending structural and geomorphic features, in contrast to the west-trending features of the Transverse Ranges to the south. A major geologic boundary within the block is associated with the Sur-Nacimiento and Rinconada faults, which separate terrains of contrasting basement rock types. The ground southwest of the Sur-Nacimiento zone and the southerly half of the Rinconada fault, referred to as the Coastal Block, is underlain by Franciscan basement rocks of dominantly oceanic types, whereas that to the northeast, referred to as the Salinia Block, is underlain by granitic and metamorphic basement rocks of continental types. Page (Reference 10) outlined the geology of the Coast Ranges, describing it generally in terms of "core complexes" of basement rocks and surrounding sections of younger sedimentary rocks. The principal Franciscan core complex of the southern Coast Range crops out on the coastal side of the Santa Lucia Range from the vicinity of San Luis Obispo to Point Sur, a distance of 120 miles. Its complex features reflect numerous episodes of deformation that evidently included folding, faulting, and the tectonic emplacement of extensive bodies of ultrabasic rocks. Other core complexes consisting of granitic and metamorphic basement rocks are exposed in the southern Coast Ranges in the ground between the Sur-Nacimiento and Rinconada and in the San Andreas fault zones. The locations of these areas of basement rock exposure are shown in Figure 2.5-6 and in Figure 1 of Appendix 2.5D of Reference 27 in Section 2.3.

Younger structural features include thick folded basins of Tertiary strata and the large faults that form structural boundaries between and within the core complexes and basins.

The structure of the southern Coast Ranges has evolved during a lengthy history of deformation extending from the time when the ancestral Sur-Nacimiento zone was a site for subduction (a Benioff zone) along the then-existing continental margin, through subsequent parts of Cenozoic time when the San Andreas fault system was the principal expression of the regional stress-strain system. The latest episodes of major deformation involved folding and faulting of Pliocene and older sediments during mid-Pliocene time, and renewed movements along preexisting faults during early or mid-Pliocene time. Present tectonic activity within the region is dominated by interaction between the Pacific and American crustal plates on opposite sides of the San Andreas fault and by continuing vertical uplift of the Coast Ranges. In the regional setting of DCPP site, the major structural features addressed during the original design phase are the San Andreas, Rinconada-San Marcos-Jolon, Sur-Nacimiento, and Santa Lucia Bank faults. Additional faults were identified during the Hosgri evaluation and LTSP evaluation phases, discussed in Sections 2.5.3.9.3 and 2.5.3.9.4, respectively. The San Simeon fault may also be included with this group. These original design phase faults are described as follows:

1. San Andreas Fault

The San Andreas fault is recognized as a major transform fault of regional dimensions that forms an active boundary between the Pacific and North American crustal plates. Cumulative slip along the San Andreas fault may have amounted to several hundred miles, and a substantial fraction of the total slip has occurred during late Cenozoic time. The fault has spectacular topographic expression, generally lying within a rift valley or along an escarpment mountain front, and having associated sag ponds, low scarps, right-laterally deflected streams, and related manifestations of recent activity.

The most recent episode of large-scale movement along the reach of the San Andreas fault that is closest to the San Luis Range occurred during the great Fort Tejon earthquake of 1857. Geologic evidence pertinent to the behavior of the fault during this and earlier seismic events was studied in great detail by Wallace (Reference 15 and 32) who reported in terms of infrequent great earthquakes accompanied by ground rupture of 10 to 30 feet, with intervening periods of near total quiescence. Allen (Reference 16 suggested that such behavior has been typical for this reach of the San Andreas fault and has been fundamentally different from the behavior of the fault along the reach farther northwest, where creep and numerous small earthquakes have occurred. He further suggested that release of accumulating strain energy might have been facilitated by the presence of large amounts of serpentine in the fault zone to the northwest, and retarded by the locking effect of the broad bend of the fault zone where it crosses the Transverse Ranges to the southeast.

Movement is currently taking place along large segments of the San Andreas fault. The active reach of the fault between Parkfield and San Francisco is currently undergoing relative movement of at least 3 to 4 cm/yr, as determined geodetically and analyzed by Savage and Burford (Reference 33). When the movement that occurs during the episodes of fault displacement in the western part of the Basin and Ranges Province is added to the minimum of 3 to 4 cm/yr of continuously and intermittently released strain, the total probably amounts to at least 5 to 6 cm/yr. This may account for essentially all of the relative motion between the Pacific and North American plates at present. In the Transverse Ranges to the south, this strain is distributed between lateral slip along the San Andreas system and east-west striking lateral slip faulting, thrust faulting, and folding. North of the latitude of Monterey Bay and south of the Transverse Ranges, transcurrent movement is again concentrated along the San Andreas system, but in those regions, it is distributed among several major strands of the system.

2. Sur-Nacimiento Fault Zone

The Sur-Nacimiento fault zone has been regarded as the system of faults that extends from the vicinity of Point Sur, near the northwest end of the Santa Lucia Range, to the Big Pine fault in the western Transverse Ranges, and that separates the granitic-metamorphic basement of the Salinian Block from the Franciscan basement of the Coastal Block. The most prominent faults that are included within this zone are, from northwest to southeast, the Sur, Nacimiento, Rinconada, and (south) Nacimiento faults. The Sur fault, which extends as far northward as Point Sur on land, continues to the northwest in the offshore continental margin. At its southerly end, the zone terminates where the (south) Nacimiento fault is cut off by the Big Pine fault. The overall length of the Sur-Nacimiento fault zone between Point Sur and the Transverse Ranges is about 180 miles. The 60 mile long Nacimiento fault, between points of juncture with the Sur and Rinconada faults, forms the longest segment within this zone. Page (Reference 11) stated that:

> "It is unlikely that the Nacimiento fault proper has displaced the ground surface in Late Quaternary time, as there are no indicative offsets of streams, ridges, terrace deposits, or other topographic features. The Great Valley-type rocks on the northeast side must have been down-dropped against the older Franciscan rocks on the southwest, yet they commonly stand higher in the topography. This implies relative quiescence of the Late Quaternary time, allowing differential erosion to take place. In a few localities, the northeast side is the low side, and this inconsistency favors the same conclusion. In addition to the foregoing circumstances, the fault is offset by minor cross-faults in a manner suggesting that little, if any, Late Quaternary near-surface movement had occurred along the main fracture."

Hart (Reference 14), on the other hand, stated that: ". . . youthful topographic features (offset streams, sag ponds, possible fault scarplets, and apparently oversteepened slopes) suggest movement along both (Sur-Nacimiento and Rinconada) fault zones." The map compiled by Jennings (Reference 23), however, shows only the Rinconada with a symbol indicating "Quaternary fault displacement."

The results of photogeologic study of the region traversed by the Sur-Nacimiento fault zone tend to support Page's view. A pronounced zone of fault-controlled topographic lineaments can be traced from the northwest end of the Nacimiento fault southeastward to the Rinconada (south Nacimiento), East Huasna, and West Huasna faults. Only along the Rinconada, however, are there topographic features that seem to have originated through fault disturbances of the ground surface rather than through differential erosion along zones of shearing and juxtaposition of differing rocks. Richter (Reference 13) noted that some historic seismicity, particularly the 1952 Bryson earthquake, appears to have originated along the Nacimiento fault. This view is supported by recent work of S. W. Smith (Reference 30) that indicates that the Bryson shock and the epicenters of several smaller, more recent earthquakes were located along or near the trace of the Nacimiento.

3. Rinconada (Nacimiento)-San Marcos-Jolon-San Antonio Fault System

A system of major faults extends northwestward, parallel to the San Andreas fault, from a point of junction with the Big Pine fault in the western Transverse Ranges. This system includes several faults that have been mapped as separate features and assigned individual names. Dibblee (Reference 27) however, has suggested that these faults are part of a single system, provisionally termed the Rinconada fault zone after one of its more prominent members. He also proposed abandoning the name Nacimiento for the large fault that constitutes the most southerly part of this system, as it is not continuous with the Nacimiento fault to the north, near the Nacimiento River. The newly defined Rinconada fault system comprises the old (south) Nacimiento, Rinconada, and San Marcos faults. Dibblee proposed that the system also include the Espinosa and Reliz faults, to the north, but detailed work by Durham (Reference 28) does not seem to support this interpretation. Instead, the system may extend into Lockwood Valley and die out there along the Jolon and San Antonio faults. All the faults of the Rinconada system have undergone significant movement during middle and late Cenozoic time, though the entire system did not behave as a unit. Dibblee pointed out that: "Relative vertical displacements are controversial, inconsistent, reversed from one segment to another; the major movement may be strike slip, as on the San Andreas fault."

Regarding the structural relationship of the Rinconada fault to nearby faults, Dibblee wrote as follows:

> "Thrust or reverse faults of Quaternary age are associated with the Rinconada fault along much of its course on one or both sides, within 9 miles, especially in areas of intense folding. In the northern part several, including the San Antonio fault, are

present along both margins of the range of hills between the Salinas and Lockwood Valleys . . . . along which this range was elevated in part. Near the southern part are the major southwest-dipping South Cuyama and Ozena faults along which the Sierra Madre Range was elevated against Cuyama Valley, with vertical displacements possibly up to 8000 feet. All these thrust or reverse faults dip inward toward the Rinconada fault and presumably either splay from it at depth, or are branches of it. These faults, combined with the intense folding between them, indicated that severe compression accompanied possible transcurrent movement along the Rinconada fault."

"The La Panza fault along which the La Panza Range was elevated .... in Quaternary time, is a reverse fault that dips northeast under the range, and is not directly related to the Rinconada fault.

"The Big Pine fault against which the Rinconada fault abuts . . . is a high angle left-lateral transcurrent fault active in Quaternary time (Reference 35). The Pine Mountain fault south of it . . . . is a northeast-dipping reverse fault along which the Pine Mountain Range was elevated in Quaternary time. This fault may have been reactivated along an earlier fault that may have been continuous with the Rinconada fault, but displaced about 8 miles from it by left slip on the Big Pine fault (Reference 12) in Quaternary time."

"The Rinconada and Reliz faults were active after deposition of the Monterey Shale and Pancho Rico Formation, which are severely deformed adjacent and near the faults. The faults were again active after deposition of the Paso Robles Formation but to a lesser degree. These faults do not affect the alluvium or terrace deposits. There are no offset stream channels along these faults. However, in two areas several canyons and streams are deviated, possibly by right-lateral movement on the (Espinosa and San Marcos segments of the) Rinconada fault. There are no indications that these faults are presently active."

4. San Simeon Fault

The fault here referred to as the San Simeon fault trends along the base of the peninsula that lies north of the settlement of San Simeon. This fault is on land for a distance of 12 miles between its only outcrop, north of Ragged Point, and Point San Simeon. It may extend as much as 16 miles farther to the southeast, to the vicinity of Point Estero. This possibility is suggested by the straight reach of coastline between Cambria and Point Estero, which is directly aligned with the onshore trend of the fault; its linear form may well have been controlled by a zone of structural weakness associated with the inferred southerly part of the fault. South of Port Estero, however, there is no evidence of faulting observable in the seismic reflection profiles across Estero Bay, and the trend defined by the Los Osos Valley-Estero Bay series of lower Miocene or Oligocene intrusives extends across the San Simeon trend without deviation.

North of Point Piedras Blancas, Silver (Reference 26) reports a fault with about 5 kilometers of vertical separation between the 4-kilometer-thick Tertiary section in the offshore basin and the nearby 1-kilometer-high exposure of Franciscan basement rocks in the coastline mountain front. The existence of a fault in this region is also indicated by the 30- milligal gravity anomaly between the offshore basin and the onshore ranges (Plate II of Appendix 2.5D of Reference 27 in Section 2.3). This postulated fault may well be a northward extension of the San Simeon fault. If this is the case, the San Simeon fault may have a total length of as much as 60 miles.

Between Point San Simeon and Ragged Point, the San Simeon fault lies along the base of a broad peninsula, the surface of which is characterized by elevated marine terraces and younger, steep-walled ravines and canyons. The low, terraced topography of the peninsula contrasts sharply with that of the steep mountain front that rises immediately behind it. Clearly, the ground west of the main fault represents a part of the sea floor that has been locally arched up.

This has resulted in exposure of the fault, which elsewhere is concealed underwater off the shoreline.

The ground between the San Simeon fault and the southwest coastline of the Piedras Blancas peninsula is underlain by faulted blocks and slivers of Franciscan rocks, serpentinites, Tertiary sedimentary breccia and volcanic rocks, and Miocene shale. The faulted contacts between these rock masses trend somewhat more westerly than the trend of the San Simeon fault. One north-dipping reverse fault, which separates serpentinite from graywacke, has broken marine terrace deposits in at least two places, one of them in the basal part of the lowest and youngest terrace. Movement along this branch fault has therefore occurred less than 130,000 years before the present, although the uppermost, youngest Pleistocene deposits are apparently not broken. Prominent topographic lineations defined by northwest-aligned ravines that incise the upper terrace surface, on the other hand, apparently have originated through headward gully erosion along faults and faulted contacts, rather than through the effects of surface faulting.

The characteristics of the San Simeon fault can be summarized as follows: The fault may be related to a fault along the coast to the north that displays some 5 kilometers of vertical displacement. Near San Simeon, it exhibits probable Pleistocene right-lateral strike-slip movement of as much as 1500 feet near San Simeon, although it apparently does not break dune sand deposits of late Pleistocene or early Holocene age. A branch reverse fault, however, breaks upper Pleistocene marine terrace deposits. The San Simeon fault may extend as far south as Point Estero, but it dies out before crossing the northern part of Estero Bay.

5. Santa Lucia Bank Fault

South of the latitude of Point Piedras Blancas, the western boundary of the main offshore Santa Maria Basin is defined by the east-facing scarp along the east side of the

Santa Lucia Bank. This scarp is associated with the Santa Lucia Bank fault, the structure that separates the subsided block under the basin from the structural high of the bank. The escarpment that rises above the west side of the fault trace has a maximum height of about 450 feet, as shown on U.S. Coast and Geodetic Survey (USC&GS) Bathymetric Map 1306N-20.

The Santa Lucia Bank fault can be traced on the sea floor for a distance of about 65 miles. Extensions that are overlapped by upper Tertiary strata continue to the south for at least another 10 miles, as well as to the north. The northern extension may be related to another, largely buried fault that crosses and may intersect the trend of the Santa Lucia Bank fault. This second fault extends to the surface only at points north of the latitude of Point Piedras Blancas.

West of the Santa Lucia Bank fault, between N latitudes 34°30' and 30°, several subparallel faults are characterized by apparent surface scarps. The longest of these faults trends along the upper continental slope for a distance of as much as 45 miles, and generally exhibits a west-facing scarp. Other faults are present in a zone about 30 miles long lying between the 45 mile fault and the Santa Lucia Bank fault. These faults range from 5 to 15 or more miles in length, and have both east-and west-facing scarps.

This zone of faulting corresponds closely in space with the cluster of earthquake epicenters around N latitude 34°45' and 121°30'W longitude, and it probably represents the source structure for those shocks (Figure 2.5-3).

2.5.2.1.2.3 Tectonic Features in the Vicinity of the DCPP Site

Geologic relationships between the major fold and fault structures in the vicinity of Diablo Canyon are shown in Figures 2.5-5, 2.5-6, and 2.5-7, and are described and illustrated in Appendix 2.5D of Reference 27 of Section 2.3. The San Luis Ranges-Estero Bay area is characterized structurally by west-northwest-trending folds and faults. These include the San Luis-Pismo syncline and the bordering Los Osos Valley and Point San Luis antiformal highs, and the West Huasna, Edna, and San Miguelito faults. A few miles offshore, the structural features associated with this trend merge into a north-northwest-trending zone of folds and faults that is referred to herein as the offshore Santa Maria Basin East Boundary zone of folding and faulting. The general pattern of structural highs and lows of the onshore area is warped and stepped downward to the west across this boundary zone, to be replaced by more northerly-trending folds in the lower part of the offshore basin section. The overall relationship between the onshore Coast Ranges and the offshore continental margin is one of differential uplift and subsidence. The East Boundary zone represents the structural expression of the zone of inflection between these regions of contrasting vertical movement.

In terms of regional relationships, structural style, and history of movement, the faults in the San Luis Ranges-Estero Bay vicinity, identified during the original design phase, may be characterized as follows:

1. West Huasna Fault

This fault zone separates the large downwarp of the Huasna syncline on the northeast from Franciscan assemblage rocks of the Los Osos Valley antiform and the Tertiary section of the southerly part of the San Luis-Pismo syncline on the southwest. The West Huasna fault is thought to join with the Suey fault to the south. Differences in thicknesses and facies relationships between units of apparently equivalent age on opposite sides of the fault are interpreted as indicating lateral movement along the fault; however, the available evidence regarding the amount and even the relative sense of displacement is not consistent. The West Huasna shows no evidence of late Quaternary activity.

2. Edna Fault Zone

The Edna fault zone lies along a west-northwesterly trend that extends obliquely from the West Huasna fault at its southeast end to the hills of the San Luis Range south of Morro Bay. Several isolated breaks that lie on a line with the trend are present in the Tertiary strata beneath the south part of Estero Bay, east of the Santa Maria Basin East Boundary fault zone across the mouth of the bay.

The Edna fault is typically a zone of two or more anastomosing branches that range in width from 1/2 mile to as much as 1-1/2 miles. Although individual strands are variously oriented and exhibit various senses of amounts of movement, the zone as a whole clearly expresses high-angle dip-slip displacement (down to the southwest). The irregular traces of major strands suggest that little, if any, strike-slip movement has occurred. Preliminary geologic sections shown by Hall and Surdam (Reference 21) and Hall (Reference 20) imply that the total amount of vertical separation ranges from 1500 to a few thousand feet along the central part of the fault zone. The amount of displacement across the main fault trend evidently decreases to the northwest, where the zone is mostly overlapped by upper Tertiary strata.

It may be, however, that most of the movement in the Baywood Park vicinity has been transferred to the north-trending branch of the Edna, which juxtaposes Pliocene and Franciscan rocks where last exposed. In the northwesterly part of the San Luis Range, the Edna fault forms much of the boundary between the Tertiary and basement rock sections. Most of the measurable displacements along this zone of rupture occurred during or after folding of the Pliocene Pismo Formation but prior to deposition of the lower Pleistocene Paso Robles Formation. Some additional movement has occurred during or since early Pleistocene time, however, because Monterey strata have been faulted against Paso Robles deposits along at least one strand of the Edna near the head of Arroyo Grande valley. This involved steep reverse fault movement, with the southwest side raised, in contrast to the earlier normal displacement down to the southwest.

Search has failed to reveal dislocation of deposits younger than the Paso Robles Formation, disturbance of late Quaternary landforms, or other evidence of Holocene or late Pleistocene activity.

3. San Miguelito Fault Zone

Northwesterly-trending faults have been mapped in the area between Pismo Beach and Arroyo Grande, and from Avila Beach to the vicinity of the west fork of Vineyard Canyon, north of San Luis Hill. Because these faults lie on the same trend, appear to reflect similar senses of movement, and are "separated" only by an area of no exposure along the shoreline between Pismo Beach and Avila Beach, they may well be part of a more or less continuous zone about 10 miles long. As on the Edna fault, movements along the San Miguelito fault appear to have been predominantly dip-slip, but with displacement down on the northeast. Hall's preliminary cross section indicates total vertical separation of about 1400 feet. The fault is mapped as being overlain by unbroken deposits of the Paso Robles Formation near Arroyo Grande.

Field checking of the ground along the projected trend of the San Miguelito fault zone northwest of Vineyard Canyon in the San Luis Range has substantiated Hall's note that the fault cannot be traced west of that area.

Detailed mapping of the nearly continuous sea cliff exposures extending across this trend northeast of Point Buchon has shown there is no faulting along the San Miguelito trend at the northwesterly end of the range. Like the Edna fault zone, the San Miguelito fault zone evidently represents a zone of high-angle dip-slip rupturing along the flank of the San Luis-Pismo syncline.

4. East Boundary Zone of the Offshore Santa Maria Basin

The boundary between the offshore Santa Maria Basin and the onshore features of the southern Coast Ranges is a 4 to 5 wide zone of generally north-northwest-trending folds, faults, and onlap unconformities referred to as the "Hosgri fault zone" by Wagner (Reference 31). The geology of this boundary zone has been investigated in detail by means of extensive seismic reflection profiling, high resolution surface profiling, and side scan sonar surveying.

More general information about structural relationships along the boundary zone has been obtained from the pattern of Bouguer Gravity anomaly values that exist in its vicinity. These data show the East Boundary zone to consist of a series of generally parallel north-northwest-trending faults and folds, developed chiefly in upper Pliocene strata that flank upwarped lower Pliocene and older rocks. The zone extends from south of the latitude of Point Sal to north of Point Piedras Blancas. Within the zone, individual fault breaks range in length from less than 1000 feet up to a maximum of about 30 miles. The overall length of the zone is approximately 90 miles, with about 60 miles of relatively continuous faulting.

The apparent vertical component of movement is down to the west across some faults and down to the east across others. Along the central reach of the zone, opposite the San Luis Range, a block of ground has been dropped between the two main strands of the fault to form a graben structure. Within the graben, and at other points along the East Boundary zone, bedding in the rock has been folded down toward the upthrown side of the west side down fault. This feature evidently is an expression of "reverse drag" phenomena.

The axes of folds in the ground on either side of the principal fault breaks can be traced for distances of as much as 22 miles. The fold axes typically are nearly horizontal; maximum axial plunges seem to be 5° or less. The structure and onlap relationships of the upper Pliocene, as reflected in the configuration of the unconformity at its base, are such that it consistently rises from the offshore basin and across the boundary zone via a series of upwarps, asymmetric folds, and faults. This configuration seems to correspond generally to a zone of warping and partial disruption along the boundary between relatively uplifting and subsiding regions.

2.5.2.1.3 Geologic History

The geologic history reflected by the rocks, structural features, and landforms of the San Luis Range is typical of that of the southern Coast Ranges of California in its length and complexity. Six general episodes for which there is direct evidence can be tabulated as follows:

Age	Episode	Evidence
Late Mesozoic	Development of Franciscan and Upper Cretaceous rock assemblages	Franciscan and other Mesozoic rocks
Late Mesozoic - Early Tertiary	Early Coast Ranges deformation	Structural features pre-served in the Mesozoic rocks
Mid-Tertiary	Uplift and erosion	Erosion surface at the base of the Tertiary section
Mid- and late-Tertiary	Accumulation of Miocene and Pliocene sedimentary and volcanic rocks	Vaqueros, Rincon, Obispo, Point Sal, Monterey, and Pismo Formation and associated volcanic intrusive, and brecciated rocks
Pliocene	Folding and faulting associated with the Pliocene Coast Ranges deformation	Folding and faulting of the Tertiary and basement rocks

Pleistocene	Uplift and erosion, development of successive tiers of wave-cut-benches alluvial fan, talus, and landslide deposition.	Pleistocene and Holocene deposits, present land-forms.

The earliest recognizable geologic history of the southern Coast Ranges began in Mesozoic time, during the Jurassic period when eugeosynclinal deposits (graywacke sandstone, shale, chert, and basalt) accumulated in an offshore trench developed in oceanic crust.

Some time after the initiation of Franciscan sedimentation, deposition of a sequence of miogeosynclinal or shelf sandstones and shales, known as the Great Valley Sequence, began on the continental crust, at some distance to the east of the Franciscan trench. Deposition of both sequences continued into Cretaceous time, even while the crustal basement section on which the Great Valley strata were being deposited was undergoing plutonism involving emplacement of granitic rocks. Subsequently, the Franciscan assemblage, the Great Valley Sequence, and the granite-intruded basement rocks were tectonically juxtaposed. The resulting terrane consisted generally of granitic basement thrust over intensely deformed Franciscan, with Great Valley Sequence strata overlying the basement, but thrust over and faulted into the Franciscan.

The processes that were involved in the tectonic juxtaposition evidently were active during the Mesozoic, and continued into the early Tertiary. Page (Reference 25) has shown that they were completed by no later than Oligocene time, so that the dual core complex basement of the southern Coast Ranges was formed by then.

The Miocene and later geologic history of the southern Coast Ranges region began with deposition of the Vaqueros and Rincon Formations on a surface eroded on the Franciscan and Great Valley core complex rocks.

Following deposition and some deformation and erosion of these formations, the stratigraphic unit that includes the Point Sal and Obispo Formations as approximately contemporaneous facies was laid down. The Obispo consists of a section of tuffaceous sandstone and mudstone, with lesser amounts of shale, and lensing layers of vitric and lithic-crystal tuff. Locally, the unit is featured by masses of clastic-textured tuffaceous rock that exhibit cross-cutting intrusive relations with the bedded parts of the formation. The Obispo and Point Sal were folded and locally eroded prior to initiation of the main episode of upper Miocene and Pliocene marine sedimentation.

During late middle Miocene to late Miocene time, deposition of the thick sections of silica-rich shale of the Monterey Formation began. Deposition of this formation and equivalent strata took place throughout much of the coastal region of California, but apparently was centered in a series of offshore basins that all developed at about the same time, some 10 to 12 million years ago. Local volcanism toward the latter part of this time is shown by the presence of diabase dikes and sills in the Monterey. Near the end of the Miocene, the Monterey strata were subjected to compressional deformation resulting in folding, in part with great complexity, and in faulting. Near the old continental margin, represented by the Sur-Nacimiento fault zone, the deformation was most intense, and was accompanied by uplift. This apparently resulted in the first development of many of the large folds of the southern Coast Ranges including the Huasna and San Luis-Pismo synclines, and in the partial erosion of the folded Monterey section in areas of uplift. The pattern of regional uplift of the Coast Ranges and subsidence of the offshore basins, with local upwarping and faulting in a zone of inflection along the boundary between the two regions, apparently became well established during the episode of late Miocene and Mio-Pliocene diastrophism.

Sedimentation resumed in Pliocene time throughout much of the region of the Miocene basins, and several thousand feet of siltstone and sandstone was deposited. This was the last significant episode of marine sedimentation in the region of the present Coast Ranges. Pliocene deposits in the region of uplift were then folded, and there was renewed movement along most of the preexisting larger faults.

Differential movements between the Coast Ranges uplift and the offshore basins were again concentrated along the boundary zone of inflection, resulting in upwarping and faulting of the basement, Miocene, and Pliocene sections. Relative displacement across parts of this zone evidently was dominantly vertical, because the faulting in the Pliocene has definitely extensional character, and Miocene structures can be traced across the zone without apparent lateral offset. The basement and Tertiary sections step down seaward, away from the uplift, along a system of normal faults having hundreds to nearly a thousand feet of dip-slip offset. A second, more seaward system of normal faults is antithetic to the master set and exhibits only tens to a few hundreds of feet of displacement. Strata between these faults locally exhibit reverse drag downfolding toward the edge of the Pliocene basin, whereas the section is essentially undeformed farther offshore. This style of deformation indicates a passive response, through gravity tectonics, to the onshore uplift.

The Plio-Pleistocene uplift was accompanied by rapid erosion, with consequent nearby deposition of clastic sediments such as the Paso Robles Formation in valleys throughout the southern Coast Ranges. The high-angle reverse and normal faulting observed by Compton (Reference 38) in the northern Santa Lucia Range also occurred farther south, probably more or less contemporaneously with accumulation of the continental deposits. Much of the Quaternary faulting other than that related to the San Andreas right lateral stress-strain system may well have occurred at this time.

Tectonic activity during the Quaternary has involved continued general uplift of the southern Coast Ranges, with superimposed local downwarping and continued movement along faults of the San Andreas system. The uplift is shown by the general high elevation and steep youthful topography that characterizes the Coast Ranges and by the widespread uplifted marine and stream terraces. Local downwarping can be seen in valleys, such as the Santa Maria Valley, where thick sections of Plio-Pleistocene and younger deposits have accumulated. Evidence of significant late Quaternary fault movement is seen in the topography along the Rinconada-San Marcos, Espinosa, San Simeon, and Santa Lucia Bank faults, as well as along the San Andreas itself. Only along the San Andreas, however, is there evidence of Holocene or contemporary movement.

The latest stage in the evolution of the San Luis Range has extended from mid-Pleistocene time to the present, and has involved more or less continuous interaction between apparent uplift of the range and alternating periods of erosion or deposition, especially along the coast, during times of relatively rising, falling, or unchanging sea level. The development of wave-cut benches and the accumulation of marine deposits on these benches have provided a reliable guide to the minimum age of latest displacements along breaks in the underlying bedrock. Detailed exploration of the interfaces between wave-cut benches and overlying marine deposits at the site of DCPP has shown that no breaks extend across these interfaces. This demonstrates that the youngest faulting or other bedrock breakage in that area antedated the time of terrace cutting, which is on the order of 80,000 to 120,000 years before the present.

The bedrock section and the surficial deposits that formerly capped this bedrock on which the power plant facilities are located have been studied in detail to determine whether they express any evidence of deformation or dislocation ascribable to earthquake effects.

The surficial geologic materials at the site consisted of a thin, discontinuous basal section of rubbly marine sand and silty sand, and an overlying section of nonmarine rocky sand and sandy clay alluvial and colluvial deposits. These deposits were extensively exposed by exploratory trenches, and were examined and mapped in detail. No evidence of earthquake-induced effects such as lurching, slumping, fissuring, and liquefaction was detected during this investigation.

The initial movement of some of the landslide masses now present in Diablo Canyon upstream from the switchyard area may have been triggered by earthquake shaking. It is also possible that some local talus deposits may represent earthquake-triggered rock falls from the sea cliff or other steep slopes in the vicinity.

Deformation of the rock substrata in the site area may well have been accompanied by earthquake activity at the time of its occurrence in the geologic past. There is no evidence, however, of post-terrace earthquake effects in the bedrock where the power plant is being constructed.

2.5.2.1.4 Stratigraphy of the San Luis Range and Vicinity

The geologic section exposed in the San Luis Range comprises sedimentary, igneous, and tectonically emplaced ultrabasic rocks of Mesozoic age, sedimentary, pyroclastic, and hypabyssal intrusive rocks of Tertiary age, and a variety of surficial deposits of Quaternary age. The lithology, age, and distribution of these rocks were studied by Headlee and more recently have been mapped in detail by Hall. The geology of the San Luis Range is shown in Figure 2.5-6 with a geologic cross section constructed using exploratory oil wells shown in Figure 2.5-7. The geologic events that resulted in the stratigraphic units described in this section are discussed in Section 2.5.2.1.3, Geologic History.

2.5.2.1.4.1 Basement Rocks

An assemblage of rocks typical of the Coast Ranges basement terrane west of the Nacimiento fault zone is exposed along the south and northeast sides of the San Luis Range. As described by Headlee, this assemblage includes quartzose and greywacke sandstone, shale, radiolarian chert, intrusive serpentine and diabase, and pillow basalt. Some of these rocks have been dated as Upper Cretaceous from contained microfossils, including pollen and spores, and Headlee suggested that they may represent dislocated parts of the Great Valley Sequence. There is contrasting evidence, however, that at least the pillow basalt and associated cherty rocks may be more typically Franciscan. Certainly, such rocks are characteristic of the Franciscan terrane. Further, a potassium-argon age of 156 million years, equivalent to Upper Jurassic, has been determined for a core of similar rocks obtained from the bottom of the Montodoro Well No. 1 near Point Buchon.

2.5.2.1.4.2 Tertiary Rocks

Five formational units are represented in the Tertiary section of the San Luis Range. The lower part of this section comprises rocks of the Vaqueros, Rincon, and Obispo Formations, which range in age from lower Miocene through middle Miocene. These strata crop out in the vicinity of Hazard Canyon, at the northwest end of the range, and in a broad band along the south coastal margin of the range. In both areas the Vaqueros rests directly on Mesozoic basement rocks. The core of the western San Luis Range is underlain by the Upper Miocene Monterey Formation, which constitutes the bulk of the Tertiary section. The Upper Miocene to Lower Pliocene Pismo Formation crops out in a discontinuous band along the southwest flank and across the west end of the range, resting with some discordance on the Monterey section and elsewhere directly on older Tertiary or basement rocks.

The coastal area in the vicinity of Diablo Canyon is underlain by strata that have been variously correlated with the Obispo, Point Sal, and Monterey Formations. Headlee, for example, has shown the Point Sal as overlying the Obispo, whereas Hall has considered these two units as different facies of a single time-stratigraphic unit. Whatever the exact stratigraphic relationships of these rocks might prove to be, it is clear that they lie above the main body of tuffaceous sedimentary rocks of the Obispo Formation and below the main part of the Monterey Formation. The existence of intrusive bodies of both tuff breccia and diabase in this part of the section indicates either that local volcanic activity continued beyond the time of deposition of the Obispo Formation, or that the section represents a predominantly sedimentary facies of the upper part of the Obispo Formation. In either case, the strata underlying the power plant site range downward through the Obispo Formation and presumably include a few hundred feet of the Rincon and Vaqueros Formations resting upon a basement of Mesozoic rocks.

A generalized description of the major units in the Tertiary section follows, and a more detailed description of the rocks exposed at the power plant site is included in a later section.

The Vaqueros Formation has been described by Headlee as consisting of 100 to 400 feet of resistant, massive, coarse-grained, calcareously cemented bioclastic sandstone. The overlying Rincon Formation consists of 200 to 300 feet of dark gray to chocolate brown calcareous shale and mudstone.

The Obispo Formation (or Obispo Tuff) is 800 to 2000 feet thick and comprises alternating massive to thick-bedded, medium to fine grained vitric-lithic tuffs, finely laminated black and brown marine siltstone and shale, and medium grained light tan marine sandstone. Headlee assigned to the Point Sal Formation a section described as consisting chiefly of medium to fine grained silty sandstone, with several thin silty and fossiliferous limestone lenses; it is gradational upward into siliceous shale characteristic of the Monterey Formation. The Monterey Formation itself is composed predominantly of porcelaneous and finely laminated siliceous and cherty shales.
The Pismo Formation consists of massive, medium to fine grained arkosic sandstone, with subordinate amounts of siltstone, sandy shale, mudstone, hard siliceous shale, and chert.

2.5.2.1.4.3 Quaternary Deposits

Deposits of Pleistocene and Holocene age are widespread on the coastal terrace benches along the southwest margin of the San Luis Range, and they exist farther onshore as local alluvial and stream-terrace deposits, landslide debris, and various colluvial accumulations. The coastal terrace deposits include discontinuous thin basal sections of marine silt, sand, gravel, and rubble, some of which are highly fossiliferous, and generally much thicker overlying sections of talus, alluvial-fan debris, and other deposits of landward origin. All of the marine deposits and most of the overlying nonmarine accumulations are of Pleistocene age, but some of the uppermost talus and alluvial deposits are Holocene. Most of the alluvial and colluvial materials consist of silty clayey sand with irregularly distributed fragments and blocks of locally exposed rock types. The landslide deposits include chaotic mixtures of rock fragments and fine-grained matrix debris, as well as some large masses of nearly intact to thoroughly disrupted bedrock.

A more detailed description of surficial deposits that are present in the vicinity of the power plant site is included in a later section.

2.5.2.1.5 Structure of the San Luis Range and Vicinity

2.5.2.1.5.1 General Features

The geologic structure of the San Luis Range-Estero Bay and adjacent offshore area is characterized by a complex set of folds and faults (Figures 2.5-5, 2.5-6, and 2.5-7). Tectonic events that produced these folds and faults are discussed in Section 2.5.2.1.3, Geologic History. The San Luis Range-Estero Bay and adjacent offshore area lies within the zone of transition from the west-trending Transverse Range structural province to the northwest-trending Coast Ranges province. Major structural features are the long narrow downfold of the San Luis-Pismo syncline and the bordering antiformal structural highs of Los Osos Valley on the northeast, and of Point San Luis and the adjacent offshore area on the southwest. This set of folds trends obliquely into a north-northwest aligned zone of basement upwarping, folding, and high-angle normal faulting that lies a few miles off the coast. The main onshore folds can be recognized, by seismic reflection and gravity techniques, in the structure of the buried, downfaulted Miocene section that lies across (west of) this zone.

Lesser, but yet important structural features in this area include smaller zones of faulting and trends of volcanic intrusives. The Edna and San Miguelito fault zones disrupt parts of the northeast and southwest flanks of the San Luis-Pismo syncline. A southward extension of the San Simeon fault, the existence of which is inferred on the basis of the linearity of the coastline between Cambria and Point Estero, and of the gravity gradient in that area, may extend into, and die out within, the northern part of Estero Bay. An aligned series of plugs and lensoid masses of Tertiary volcanic rocks that intrude the Franciscan Formation along the axis of the Los Osos Valley antiform extends from the outer part of Estero Bay southeastward for 22 miles (Figure 2.5-6).

These features define the major elements of geologic structure in the San Luis Range-Estero Bay area. Other structural elements include the complex fold and fault structures within the Franciscan core complex rocks and the numerous smaller folds within the Tertiary section.

2.5.2.1.5.2 San Luis-Pismo Syncline

The main synclinal fold of the San Luis Range, referred to here as the San Luis-Pismo syncline, trends about N60°W and forms a structural trend more than 15 miles in length. The fold system comprises several parallel anticlines and synclines across its maximum onshore width of about 5 miles. Individual folds of the system typically range in length from hundreds of feet to as much as 10,000 feet. The folds range from zero to more than 30° in plunge, and have flank dips as steep as 90°. Various kinds of smaller folds exist locally, especially flexures and drag folds associated with tuff intrusions and with zones of shear deformation.

Near Estero Bay, the major fold extends to a depth of more than 6000 feet. Farther south, in the central part of the San Luis Range, it is more than 11,000 feet deep. Parts of the northeast flank of the fold are disrupted by faults associated with the Edna fault zone. Local breaks along the central part of the southwest flank have been referred to as the San Miguelito fault zone.

2.5.2.1.5.3 Los Osos Valley Antiform

The body of Franciscan and Great Valley Sequence rocks that crops out between the San Luis-Pismo and Huasna synclines is here referred to as the Los Osos Valley antiform. This composite structure extends southward from the Santa Lucia Range, across the central and northern part of Estero Bay, and thence southeastward to the point where it is faulted out at the juncture of the Edna and the West Huasna fault zones.

Notable structural features within this core complex include northwest- and west-northwest- trending-faults that separate Franciscan melange, graywacke, metavolcanic, and serpentinite units. The serpentinites have been intruded or dragged within faults, apparently over a wide range of scales. One of the more persistent zones of serpentinite bodies occurs along a trend which extends west-northwestward from the West Huasna fault. It has been suggested that movement from this fault may have taken place within this serpentine belt. The range of hills that lies between the coast and Highway 1 between Estero Bay and Cambria is underlain by sandstone and minor shale of the Great Valley Sequence, referred to as the Cambria slab, which has been underthrust by Franciscan rocks. The thrust contact extends southeastward under Estero Bay near Cayucos. This contact is probably related to the fault contact between Great Valley and Franciscan rocks located just north of San Luis Obispo, which Page has shown to be overlain by unbroken lower Miocene strata.

A prominent feature of the Los Osos Valley antiform is the line of plugs and lensoid masses of intrusive Tertiary volcanic rocks. These distinctive bodies are present at isolated points along the approximate axis of the antiform over a distance of 22 miles, extending from the center of outer Estero Bay to the upper part of Los Osos Valley (Figure 2.5-6). The consistent trend of the intrusives provides a useful reference for assessing the possibility of northwest-trending lateral slip faulting within Estero Bay. It shows that such faulting has not extended across the trend from either the inferred San Simeon fault offshore south extension, or from faults in the ground east of the San Simeon trend.

2.5.2.1.5.4 Edna and San Miguelito Fault Zones

These fault zones are described in Section 2.5.2.1.2.3.

2.5.2.1.5.5 Adjacent Offshore Area and East Boundary of the Offshore Santa Maria Basin

The stratigraphy and west-northwest-trending structure that characterize the onshore region from Point Sal to north of Point Estero have been shown by extensive marine geophysical surveying to extend into the adjacent offshore area as far as the north-northwest trending structural zone that forms a boundary with the main offshore Santa Maria Basin. Owing to the irregular outline of the coast, the width of the offshore shelf east of this boundary zone ranges from 2-1/2 to as much as 12 miles. The shelf area is narrowest opposite the reach of coast between Point San Luis and Point Buchon, and widest in Estero Bay and south of San Luis Bay.

The major geologic features that underlie the near-shore shelf include, from south to north, the Casmalia Hills anticline, the broad Santa Maria Valley downwarp, the anticlinal structural high off Point San Luis, the San Luis-Pismo syncline, and the Los Osos Valley antiform.

The form of these features is defined by the outcrop pattern and structure of the older Pliocene, Miocene, and basement core complex rocks. The younger Pliocene strata that constitute the upper 1000 to 2000 feet of section in the adjacent offshore Santa Maria Basin are partly buttressed and partly faulted against the rocks that underlie the near-shore shelf, and they unconformably overlap the boundary zone and parts of the shelf in several areas.

The boundaries between the San Luis-Pismo syncline and the adjacent Los Osos Valley and Point San Luis antiforms can be seen in the offshore area to be expressed chiefly as zones of inflection between synclinal and anticlinal folds, rather than as zones of fault rupture such as occurs farther south along the Edna and San Miguelito faults. Isolated west-northwest- trending faults of no more than a few hundred feet displacement are located along the northeast flank of the syncline in Estero Bay. These faults evidently are the northwesternmost expressions of breakage along the Edna fault trend.

The main San Luis-Pismo synclinal structure opens to the northwest, attaining a maximum width of 8 or 9 miles in the southerly part of Estero Bay. The Point San Luis high, on the other hand, is a domal structure, the exposed basement rock core of which is about 10 miles long and 5 miles wide.

The general characteristics of the Santa Maria Basin East Boundary zone have been described in Section 2.5.2.1.2.3. As was noted there, the zone is essentially an expression of the boundary between the synclinorial downwarp of the offshore basin and the regional uplift of the southern Coast Ranges. In the vicinity of the San Luis Range, the zone is characterized by pronounced upwarping and normal faulting of the basement and overlying Tertiary rock sections. Both modes of deformation have contributed to the structural relief of about 500 feet in the Pliocene section, and of 1500 feet or more in the basement rocks, across this boundary. Successively younger strata are banked unconformably against the slopes that have formed from time to time in response to the relative uplifting of the ground east of the boundary zone.

A series of near-surface structural troughs forms prominent features within the segment of the boundary zone structure that extends between the approximate latitudes of

Arroyo Grande and Estero Bay. This trough structure apparently has formed through the extension and subsidence of a block of ground in the zone where the downwarp of the offshore basin has pulled away from the Santa Lucia uplift. Continued subsidence of this block has resulted in deformation and partial disruption of the buttress unconformity between the offshore Pliocene section and the near-shore Miocene and older rocks. This deformation is expressed by normal faulting and reverse drag type downfolding of the Pliocene strata adjacent to the contact, along the east side of the trough.

On the opposite, seaward side of the trough, a series of antithetic down-to-the-east normal faults of small displacement has formed in the Pliocene strata west of the contact zone. These faults exhibit only a few tens of feet displacement, and they seem to exhibit constant or even decreasing displacement downward.

The structural evolution of the offshore area near Estero Bay and the San Luis Range involved episodes of compressional deformation that affected the upper Tertiary section similarly on opposite sides of the boundary zone. The section on either side exhibits about the same intensity and style of folding. Major folds, such as the San Luis-Pismo syncline and the Piedras Blancas anticline, can be traced into the ground across the boundary zone.

The internal structure of the zone, including the presence of several on-lap unconformities in the adjacent Pliocene section, shows that, at least during Pliocene and early Pleistocene time, the boundary zone has been the inflection line between the Coast Ranges uplift and the offshore Santa Maria Basin downwarp.

Evidence that uplift has continued through late Pleistocene time, at least in the vicinity of the San Luis Range, is given by the presence of successive tiers of marine terraces along the seaward flank of the range. The wave-cut benches and back scarps of these terraces now exist at elevations ranging from about -300 feet (below sea level) to more than 300 feet above sea level.

The ground within which the East Boundary zone lies has been beveled by the post-Wisconsin marine transgression, and so the zone generally is not expressed topographically. Small topographic features, such as a seaward topographic step-up of the sea floor surface across the east-down fault at the BBN (Reference 37) (offshore) survey line 27 crossing, in Estero Bay, and several possible fault-line notch back scarps, however, may represent minor topographic expressions of deformation within the zone.

2.5.2.1.6 Structural Stability

The potential for surface or subsurface subsidence, uplift, or collapse at the site or in the region surrounding the site, is discussed in Section 2.5.5, Stability of Subsurface Materials.

2.5.2.1.7 Regional Groundwater

Groundwater in the region surrounding the site is used as a backup source due to its poor quality and the lack of a significant groundwater reservoir. Section 2.4.13 states that most of the groundwater at the site or in the area around the site is either in the alluvial deposits of Diablo Creek or seeps from springs encountered in excavations at the site.

2.5.2.2 Site Geology

2.5.2.2.1 Site Physiography

The site consists of approximately 750 acres near the mouth of Diablo Creek and is located on a sloping coastal terrace, ranging from 60 to 150 feet above sea level. The terrace terminates at the Pacific Ocean on the southwest and extends toward the San Luis Mountains on the northeast. The terrace consists of bedrock overlain by surficial deposits of marine and nonmarine origin.

The remainder of this section presents a detailed description of site geology.

2.5.2.2.2 General Features

The area of the DCPP site is a coastal tract in San Luis Obispo County approximately 6.5 miles northwest of Point San Luis. It lies immediately southeast of the mouth of Diablo Canyon, a major westward-draining feature of the San Luis Range, and about a mile southeast of Lion Rock, a prominent offshore element of the highly irregular coastline.

The ground being developed as a power plant site occupies an extensive topographic terrace about 1000 feet in average width. In its pregrading, natural state, the gently undulating surface of this terrace sloped gradually southwestward to an abrupt termination along a cliff fronting the ocean; in a landward, or northeasterly, direction, it rose with progressively increasing slope to merge with the much steeper front of a foothill ridge of the San Luis Range. The surface ranged in altitude from 65 to 80 feet along the coastline to a maximum of nearly 300 feet along the base of the hillslope to the northeast, but nowhere was its local relief greater than 10 feet. Its only major interruption was the steep-walled canyon of lower Diablo Creek, a gash about 75 feet in average depth.

The entire subject area is underlain by a complex sequence of stratified marine sedimentary rocks and tuffaceous volcanic rocks, all of Tertiary (Miocene) age. Diabasic intrusive rocks are locally exposed high on the walls of Diablo Canyon at the edge of the area. Both the sedimentary and volcanic rocks have been folded and otherwise disturbed over a considerable range of scales.

Surficial deposits of Quaternary age are widespread. In a few places, they are as thick as 50 feet, but their average thickness probably is on the order of 20 feet over the terrace areas and 10 feet or less over the entire mapped ground. The most extensive deposits underlie the main topographic terrace.

Like many other parts of the California coast, the Diablo Canyon area is characterized by several wave-cut benches of Pleistocene age. These surfaces of irregular but generally low relief were developed across bedrock by marine erosion, and they are ancient analogues of the benches now being cut approximately at sea level along the present coast. They were formed during periods when the sea level was higher, relative to the adjacent land, than it is now. Each is thinly and discontinuously mantled with marine sand, gravel, and rubble similar to the beach and offshore deposits that are accumulating along the present coastline. Along its landward margin each bears thicker and more localized coarse deposits similar to the modern talus along the base of the present sea cliff.

Both the ancient wave-cut benches and their overlying marine and shoreline deposits have been buried beneath silty to gravelly detritus derived from landward sources after the benches were, in effect, abandoned by the ocean. This nonmarine cover is essentially an apron of coalescing fan deposits and other alluvial debris that is thickest adjacent to the mouths of major canyons.

Where they have been deeply trenched by subsequent erosion, as along Diablo Canyon in the map areas, these deposits can be seen to have buried some of the benches so deeply that their individual identities are not reflected by the present (pregrading) rather smooth terrace topography. Thus, the surface of the main terrace is defined mainly by nonmarine deposits that conceal both the older benches of marine erosion and some of the abruptly rising ground that separates them (refer to Figures 2.5-8 and 2.5-10).

The observed and inferred relationships among the terrace surfaces and the wave-cut benches buried beneath them can be summarized as follows:

Wave-cut Bench		Terrace Surface	
Altitude, feet	Location	Altitude, feet	Location
170-175	Small remnants on sides of Diablo Canyon	Mainly 170-190	Sides of Diablo Canyon and upper parts of main terrace; in places separated from lower parts of terrace by scarps
145-155	Very small remnants on sides of Diablo Canyon	Mainly 150-170	
120-130 90-100 65-80	Subparallel benches elongate in a northwest-southeast direction but with considerable aggregate width; wholly	Mainly 70-160	Most of main terrace, a wide-spread surface on a composite section of nonmarine deposits; no well-defined scarps

	beneath main terrace surface	50-100	Small remnants above modern sea cliff
30-45	Small remnants above modern sea cliff		No depositional terrace
Approx. 0	Small to moderately large areas along present coastline.		

Within the subject area the wave-cut benches increase progressively in age with increasing elevation above present sea level; hence, their order in the above list is one of decreasing age. By far, the most extensive of these benches slopes gently seaward from a shoreline angle that lies at an elevation of 100 feet above present sea level.

The geology of the power plant site is shown in the site geologic maps, Figures 2.5-8 and 2.5-9, and geologic section, Figure 2.5-10.

2.5.2.2.3 Stratigraphy

2.5.2.2.3.1 Obispo Tuff

The Obispo Tuff, which has been classified either as a separate formation or as a member of the Miocene Monterey Formation, is the oldest bedrock unit exposed in the site area. Its constituent rocks generally are well exposed, appear extensively in the coastward parts of the area, and form nearly all of the offshore prominences and shoals. They are dense to highly porous, and thinly layered to almost massive. Their color ranges from white to buff in fresh exposures, and from yellowish to reddish brown on weathered surfaces, many of which are variegated in shades of brown. Outcrop surfaces have a characteristic "punky" to crusty appearance, but the rocks in general are tough, cohesive, and relatively resistant to erosion.

Several pyroclastic rock types constitute the Obispo Tuff ("To" on map, Figure 2.5-8) in and near the subject area. By far, the most widespread is fine-grained vitric tuff with rare to moderately abundant tabular crystals of sodic plagioclase. The constituent glass commonly appears as fresh shards, but in many places it has been partly or completely devitrified. Crystal tuffs are locally prominent, and some of these are so crowded with 1/8 to 3/8 inch crystals of plagioclase that they superficially resemble granitoid plutonic rocks. Other observed rock types include pumiceous tuffs, pumice-pellet tuff breccias, perlitic vitreous tuffs, tuffaceous siltstones and mudstones, and fine-grained tuff breccias with fragments of glass and various Monterey rocks. No massive flow rocks were recognized anywhere in the exposed volcanic section.

In terms of bulk composition, the pyroclastic rocks appear to be chiefly soda rhyolites and soda quartz latites. Their plagioclase, which ranges from calcic albite to sodic oligoclase, commonly is accompanied by lesser amounts of quartz as small rounded crystals and irregular crystal fragments. Biotite, zircon, and apatite also are present in many of the specimens that were examined under the microscope. Most of the tuffaceous rocks, and especially the more vitreous ones, have been locally to pervasively altered. Products of silicification, zeolitization, and pyritization are readily recognizable in many exposures, where the rocks generally are traversed by numerous thin, irregular veinlets and layers of cherty to opaline material. Veinlets and thin, pod-like concentrations of gypsum also are widespread. Where pyrite is present, the rocks weather yellowish to brownish and are marked by gossan-like crusts.

The various contrasting rock types are simply interlayered in only a few places; much more typical are abutting, intertonguing, and irregularly interpenetrating relationships over a wide range of scales. Septa and inclusions of Monterey rocks are abundant, and a few of them are large enough to be shown separately on the accompanying geologic map (Figure 2.5-8). Highly irregular inclusions, a few inches to several feet in maximum dimension, are so densely packed together in some places that they form breccias with volcanic matrices.

The Obispo Tuff is underlain by mudstones of early Miocene (pre-Monterey) age, on which it rests with a highly irregular contact that appears to be in part intrusive. This contact lies offshore in the vicinity of the power plant site, but it is exposed along the seacoast to the southeast.

In a gross way, the Obispo underlies the basal part of the Monterey formation, but many of its contacts with these sedimentary strata are plainly intrusive. Moreover, individual sills and dikes of slightly to thoroughly altered tuffaceous rocks appear here and there in the Monterey section, not uncommonly at stratigraphic levels well above its base (refer to Figures 2.5-8 and 2.5-13). The observed physical relationships, together with the local occurrence of diatoms and foraminifera within the principal masses of volcanic rocks, indicate that much of the Obispo Tuff in this area probably was emplaced at shallow depths beneath the Miocene sea floor during accumulation of the Monterey strata. The tuff unit does not appear to represent a single, well-defined eruptive event, nor is it likely to have been derived from a single source conduit.

2.5.2.2.3.2 Monterey Formation

Stratified marine rocks variously correlated with the Monterey Formation, Point Sal Formation, and Obispo Tuff underlie most of the subject area, including all of that portion intended for power plant location. They are almost continuously exposed along the crescentic sea cliff that borders Diablo Cove, and elsewhere they appear in much more localized outcrops. For convenience, they are here assigned to the Monterey Formation ("Tm" on map, Figure 2.5-8) in order to delineate them from the adjacent more tuffaceous rocks so typical of the Obispo Tuff.

The observed rock types, listed in general order of decreasing abundance, are silty and tuffaceous sandstone, siliceous shale, shaly siltstone and mudstone, diatomaceous shale, sandy to highly tuffaceous shale, calcareous shale and impure limestone, bituminous shale, fine- to coarse-grained sandstone, impure vitric tuff, silicified limestone and shale, and tuff-pellet sandstone. Dark colored and relatively fine-grained strata are most abundant in the lowest part of the section, as exposed along the east side of Diablo Cove, whereas lighter colored sandstones and siliceous shales are dominant at stratigraphically higher levels farther north. In detail, however, the different rock types are interbedded in various combinations, and intervals of uniform lithology rarely are thicker than 30 feet. Indeed, the closely-spaced alternations of contrasting strata yield a prominent rib-like pattern of outcrop along much of the sea cliff and shoreline bench forming the margin of Diablo Cove.

The sandstones are mainly fine- to medium-grained, and most are distinctly tuffaceous. Shards of volcanic glass generally are recognizable under the microscope, and the very fine-grained siliceous matrix may well have been derived largely through alteration of original glassy material. Some of the sandstone contains small but megascopically visible fragments of pumice, perlitic glass, and tuff, and a few beds grade along strike into submarine tuff breccia. The sandstones are thinly to very thickly layered; individual beds 6 inches to 4 feet thick are fairly common, and a few appear to be as thick as 15 feet. Some of them are hard and very resistant to erosion, and they typically form subdued but nearly continuous elongated projections on major hillslopes (Figure 2.5-8).

The siliceous shales are buff to light gray platy rocks that are moderately hard to extremely hard according to their silica content, but they tend to break readily along bedding and fracture surfaces. The bituminous rocks and the siltstones and mudstones are darker colored, softer, and grossly more compact. Some of them are very thinly bedded or laminated, others appear almost massive or form matrices for irregularly ellipsoidal masses of somewhat sandier material. The diatomaceous, tuffaceous, and sandy rocks are lighter colored. The more tuffaceous types are softer, and the diatomaceous ones are soft to the degree of punkiness; both kinds of rocks are easily eroded, but are markedly cohesive and tend to retain their gross positions on even the steepest of slopes.

The siliceous shale and most of the hardest, highly silicified rocks weather to very light gray, and the dark colored, fine-grained rocks tend to bleach when weathered. The other types, including the sandstones, weather to various shades of buff and light brown. Stains of iron oxides are widespread on exposures of nearly all the Monterey rocks, and are especially well developed on some of the finest-grained shales that contain disseminated pyrite. All but the hardest and most thick-bedded rocks are considerably broken to depths of as much as 6 feet in the zone of weathering on slopes other than the present sea cliff, and the broken fragments have been separated and displaced by surface creep to somewhat lesser depths.

2.5.2.2.3.3 Diabasic Intrusive Rocks

Small, irregular bodies of diabasic rocks are poorly exposed high on the walls of Diablo Canyon at and beyond the northeasterly edge of the map area. Contact relationships are readily determined at only a few places where these rocks evidently are intrusive into the Monterey Formation. They are considerably weathered, but an ophitic texture is recognizable. They consist chiefly of calcic plagioclase and augite, with some olivine, opaque minerals, and zeolitic alteration products.

2.5.2.2.3.4 Masses of Brecciated Rocks

Highly irregular masses of coarsely brecciated rocks, a few feet to many tens of feet in maximum dimension, are present in some of the relatively siliceous parts of the Monterey section that adjoin the principal bodies of Obispo Tuff. The fracturing and dislocation is not genetically related to any recognizable faults, but instead seems to have been associated with emplacement of the volcanic rocks; it evidently was accompanied by, or soon followed by, extensive silicification. Many adjacent fragments in the breccias are closely juxtaposed and have matching opposed surfaces, so that they plainly represent no more than coarse crackling of the brittle rocks. Other fragments, though angular or subangular, are not readily matched with adjacent fragments and hence may represent significant translation within the entire rock masses.

The ratio of matrix materials to coarse fragments is very low in most of the breccias and nowhere was it observed to exceed about 1:3. The matrices generally comprise smaller angular fragments of the same Monterey rocks that are elsewhere dominant in the breccias, and they characteristically are set in a siliceous cement. Tuffaceous matrices, with or without Monterey fragments, also are widespread and commonly show the effects of pervasive silicification. All the exposed breccias are firmly cemented, and they rank among the hardest and most resistant units in the entire bedrock section.

A few 3 to 18 inch beds of sandstone have been pulled apart to form separate tabular masses along specific stratigraphic horizons in higher parts of the Monterey sequence. Such individual tablets, which are boudins rather than ordinary breccia fragments, are especially well exposed in the sea cliff at the northern corner of Diablo Cove. They are flanked by much finer-grained strata that converge around their ends and continue essentially unbroken beyond them. This boudinage or separation and stringing out of sandstone beds that lie within intervals of much softer and more shaly rocks has resulted from compression during folding of the Monterey section. Its distribution is stratigraphically controlled and is not systematically related to recognizable faults in the area.

2.5.2.2.3.5 Surficial Deposits

1. Coastal Terrace Deposits

The coastal wave-cut benches of Pleistocene age, as described in a foregoing section, are almost continuously blanketed by terrace deposits (Qter in Figure 2.5-8) of several contrasting types and modes of origin. The oldest of these deposits are relatively thin and patchy in their occurrence, and were laid down along and adjacent to ancient beaches during Pleistocene time. They are covered by considerably thicker and more extensive nonmarine accumulations of detrital materials derived from various landward sources.

The marine deposits consist of silt, sand, gravel, and cobbly to bouldery rubble. They are approximately 2 feet in average thickness over the entire terrace area and reach a maximum observed thickness of about 8 feet. They rest directly upon bedrock, some of which is marked by numerous holes attributable to the action of boring marine mollusks, and they commonly contain large rounded cobbles and boulders of Monterey and Obispo rocks that have been similarly bored. Lenses and pockets of highly fossiliferous sand and gravel are present locally.

The marine sediments are poorly to very well sorted and loose to moderately well consolidated. All of them have been naturally compacted; the degree of compaction varies according to the material, but it is consistently greater than that observed in any of the associated surficial deposits of other types. Near the inner margins of individual wave-cut benches the marine deposits merge landward into coarser and less well-sorted debris that evidently accumulated along the bases of ancient sea cliffs or other shoreline slopes. This debris is locally as much as 12 feet thick; it forms broad but very short aprons, now buried beneath younger deposits, that are ancient analogues of the talus accumulations along the inner margin of the present beach in Diablo Cove. One of these occurrences, identified as "fossil Qtb" in the geologic map of Figure 2.5-8, is well exposed high on the northerly wall of Diablo Canyon.

A younger, thicker, and much more continuous nonmarine cover is present over most of the coastal terrace area. It consistently overlies the marine deposits noted above, and, where these are absent, it rests directly upon bedrock. It is composed in part of alluvial detritus contributed during Pleistocene time from Diablo Canyon and several smaller drainage courses, and it thickens markedly as traced sourceward toward these canyons. The detritus represents a series of alluvial fans, some of which appear to have partly coalesced with adjacent ones. It is chiefly fine- to moderately-coarse-grained gravel and rubble characterized by tabular fragments of Monterey rocks in a rather abundant silty to clayey matrix. Most of it is thinly and regularly stratified, but the distinctness of this layering varies greatly from place to place.

Slump, creep, and slope-wash deposits, derived from adjacent hillsides by relatively slow downhill movement over long periods of time, also form major parts of the nonmarine terrace cover. All are loose and uncompacted. They comprise fragments of Monterey rocks in dark colored clayey matrices, and their internal structure is essentially chaotic. In some places they are crudely interlayered with the alluvial fan deposits, and elsewhere they overlie these bedded sediments. On parts of the main terrace area not reached by any of the alluvial fans, a cover of slump, creep, and slope-wash deposits, a few inches to nearly 10 feet thick, rests directly upon either marine terrace deposits or bedrock.

Thus, the entire section of terrace deposits that caps the coastal benches of Pleistocene marine erosion is heterogeneous and internally complex; it includes contributions of detritus from contrasting sources, from different directions at different times, and via several basically different modes of transport and deposition.

2. Stream-terrace Deposits

Several narrow, irregular benches along the walls of Diablo Canyon are veneered by a few inches to 6 feet of silty gravels that are somewhat coarser but otherwise similar to the alluvial fan deposits described above. These stream-terrace deposits (Qst) originally occupied the bottom of the canyon at a time when the lower course of Diablo Creek had been cut downward through the alluvial fan sediments of the main terrace and well into the underlying bedrock. Subsequent deepening of the canyon left remnants of the deposits as cappings on scattered small terraces.

3. Landslide Deposits

The walls of Diablo Canyon also are marked by tongue- and bench-like accumulations of loose, rubbly landslide debris (Qls), consisting mainly of highly broken and jumbled masses of Monterey rocks with abundant silty and soily matrix materials. These landslide bodies represent localized failure on naturally oversteepened slopes, generally confined to fractured bedrock in and immediately beneath the zone of weathering. Individual bodies within the mapped area are small, with probable maximum thicknesses no greater than 20 feet. All of them lie outside the area intended for power plant construction.

Landslide deposits along the sea cliff have been recognized at only one locality, on the north side of Diablo Cove about 400 feet northwest of the mouth of Diablo Canyon. Here slippage has occurred along bedding and fracture surfaces in siliceous Monterey rocks, and it has been confined essentially to the axial region of a well-defined syncline (refer to Figure 2.5-8). Several episodes of sliding are attested by thin, elongate masses of highly broken ground separated from one another by well-defined zones of dislocation. Some of these masses are still capped by terrace deposits. The entire composite accumulation of debris is not more than 35 feet in maximum thickness, and ground failure at this locality does not appear to have resulted in major recession of the cliff. Elsewhere within the mapped area, landsliding along the sea cliff evidently has not been a significant process.

Large landslides, some of them involving substantial thickness of bedrock, are present on both sides of Diablo Canyon not far northeast of the power plant area. These occurrences need not be considered in connection with the plant site, but they have been regarded as significant factors in establishing a satisfactory grading design for the switchyard and other up-canyon installations. They are not dealt with in this section.

4. Slump, Creep, and Slope-wash Deposits

As noted earlier, slump, creep, and slope-wash deposits (Qsw) form parts of the nonmarine sedimentary blanket on the main terrace. These materials are shown separately on the geologic map only in those limited areas where they have been considerably concentrated along well-defined swales and are readily distinguished from other surficial deposits. Their actual distribution is much wider, and they undoubtedly are present over a large fraction of the areas designated as Qter; their average thickness in such areas, however, is probably less than 5 feet.

Angular fragments of Monterey rocks are sparsely to very abundantly scattered through the slump, creep, and slope-wash deposits, whose most characteristic feature is a fine-grained matrix that is dark colored, moderately rich in clay minerals, and extremely soft when wet. Internal layering is rarely observable and nowhere is sharply expressed. The debris seems to have been rather thoroughly intermixed during its slow migration down hillslopes in response to gravity. That it was derived mainly from broken materials in the zone of weathering is shown by several exposures in which it grades downward through soily debris into highly disturbed and partly weathered bedrock, and thence into progressively fresher and less broken bedrock.

5. Talus and Beach Deposits

Much of the present coastline in the subject area is marked by bare rock, but Diablo Cove and a few other large indentations are fringed by narrow, discontinuous beaches and irregular concentrations of sea cliff talus. These deposits (Qtb) are very coarse grained. Their total volume is small, and they are of interest mainly as modern analogues of much older deposits at higher levels beneath the main terrace surface.

The beach deposits consist chiefly of well-rounded cobbles. They form thin veneers over bedrock, and in Diablo Cove they grade seaward into patches of coarse pebbly sand. The floors of both Diablo Cove and South Cove probably are irregular in detail and are featured by rather hard, fresh bedrock that is discontinuously overlain by irregular thin bodies of sand and gravel. The distribution and abundance of kelp suggest that bedrock crops out over large parts of these cove areas where the sea bottom cannot be observed from onshore points.

6. Stream-laid Alluvium

Stream-laid alluvium (Qal) occurs as a strip along the present narrow floor of Diablo Canyon, where it is only a few feet in average thickness. It is composed of irregularly intertongued silt, sand, gravel, and rubble. It is crudely to sharply stratified, poorly to well sorted, and, in general, somewhat compacted. Most of it is at least moderately porous.

7. Other Deposits

Earlier inhabitation of the area by Indians is indicated by several midden deposits that are rich in charcoal and fragments of shells and bones. The most extensive of these occurrences marks the site of a long-abandoned village along the edge of the main terrace immediately northwest of Diablo Canyon. Others have been noted on the main terrace just east of the mouth of Diablo Canyon, on the shoreward end of South Point, and at several places in and near the plant site.

2.5.2.2.4 Structure

2.5.2.2.4.1 Tectonic Structures Underlying the Region Surrounding the Site

The dominant tectonic structure in the region of the power plant site is the San Luis-Pismo downwarp system of west-northwest-trending folds. This structure is bounded on the northeast by the antiformal basement rock structure of the Los Osos and San Luis Valley trend. The west-northwest-trending Edna fault zone lies along the northeast flank of the range, and the parallel Miguelito fault extends into the southeasterly end of the range. A north-northwest- trending structural discontinuity that may be a fault has been inferred or interpolated from widely spaced traverses in the offshore, extending within about 5 miles of the site at its point of closest approach. To the west of this discontinuity, the structure is dominated by north to north-northwest-trending folds in Tertiary rocks. These features are illustrated in Figure 2.5-3 and described in this section.

Tectonic structures underlying the site and region surrounding the site are identified in the above and following sections, and they are shown in Figures 2.5-3, 2.5-5, 2.5-8, 2.5-10, 2.5-15, and 2.5-16. They are listed as follows:

2.5.2.2.4.2 Tectonic Structures Underlying the Site

The rocks underlying the DCPP site have been subjected to intrusive volcanic activity and to later compressional deformation that has given rise to folding, jointing and fracturing, minor faulting, and local brecciation. The site is situated in a section of moderately to steeply north-dipping strata, about 300 feet south of an east-west-trending synclinal fold axis (Figures 2.5-8 and 2.5-10). The rocks are jointed throughout, and they contain local zones of closely spaced high-angle fractures (Figure 2.5-16).

A minor fault zone extends into the site from the west, but dies out in the vicinity of the Unit 1 turbine building. Two other minor faults were mapped for distances of 35 to more than 200 feet in the bedrock section exposed in the excavation for the Unit 1 containment structure. In addition to these features, cross-cutting bodies of tuff and tuff brecia, and cemented "crackle breccia" could be considered as tectonic structures.

Exact ages of the various tectonic structures at the site are not known. It has been clearly demonstrated, however, that all of them are truncated by, and therefore antedate, the principal marine erosion surface that underlies the coastal terrace bench. This terrace can be correlated with coastal terraces to the north and south that have been dated as 80,000 to 120,000 years old. The tectonic structures probably are related to the Pliocene-lower Pleistocene episode of Coast Ranges deformation, which occurred more than 1 million years ago.

The bedrock units within the entire subject area form part of the southerly flank of a very large syncline that is a major feature of the San Luis Range. The northerly-dipping sequence of strata is marked by several smaller folds with subparallel trends and flank-to-flank dimensions measured in hundreds of feet. One of these, a syncline with gentle to moderate westerly plunge, is the largest flexure recognized in the vicinity of the power plant site. Its axis lies a short distance north of the site and about 450 feet northeast of the mouth of Diablo Canyon (Figures 2.5-8 and 2.5-10). East of the canyon this fold appears to be rather open and simple in form, but farther west it probably is complicated by several large wrinkles and may well lose its identity as a single feature. Some of this complexity is clearly revealed along the northerly margin of Diablo Cove, where the beds exposed in the sea cliff have been closely folded along east to northeast trends. Here a tight syncline (shown in Figure 2.5-8) and several smaller folds can be recognized, and steep to near-vertical dips are dominant in several parts of the section.

The southerly flank of the main syncline within the map area steepens markedly as traced southward away from the fold axis. Most of this steepening is concentrated within an across-strike distance of about 300 feet as revealed by the strata exposed in the sea cliff southeastward from the mouth of Diablo Canyon; farther southward the beds of sandstone and finer-grained rocks dip rather uniformly at angles of 70° or more. A slight overturning through the vertical characterizes the several hundred feet of section exposed immediately north of the Obispo Tuff that underlies South Point and the north shore of South Cove (refer to Figure 2.5-8). Thus the main syncline, though simple in gross form, is distinctly asymmetric. The steepness of its southerly flank may well have resulted from buttressing, during the folding, by the relatively massive and competent unit of tuffaceous rocks that adjoins the Monterey strata at this general level of exposure.

Smaller folds, corrugations, and highly irregular convolutions are widespread among the Monterey rocks, especially the finest-grained and most shaley types. Some of these flexures trend east to southeast and appear to be drag features systematically related to the larger-scale folding in the area. Most, however, reflect no consistent form or trend, range in scale from inches to only a few feet, and evidently are confined to relatively soft rocks that are flanked by intervals of harder and more massive strata. They constitute crudely tabular zones of contortion within which individual rock layers can be traced for short distances but rarely are continuous throughout the deformed ground.

Some of this contortion appears to have derived from slumping and sliding of unconsolidated sediments on the Miocene sea floor during accumulation of the

Monterey section. Most of it, in contrast, plainly occurred at much later times, presumably after conversion of the sediments to sedimentary rocks, and it can be most readily attributed to highly localized deformation during the ancient folding of a section that comprises rocks with contrasting degrees of structural competence.

2.5.2.2.4.3 Faults

Numerous faults with total displacements ranging from a few inches to several feet cut the exposed Monterey rocks. Most of these occur within, or along the margins of, the zones of contortion noted above. They are sharp, tight breaks with highly diverse attitudes, and they typically are marked by 1/16-inch or less of gouge or microbreccia. Nearly all of them are curving or otherwise somewhat irregular surfaces, and many can be seen to terminate abruptly or to die out gradually within masses of tightly folded rocks. These small faults appear to have been developed as end products of localized intense deformation caused by folding of the bedrock section. Their unsystematic attitudes, small displacements, and limited effects upon the host rocks identify them as second-order features, i.e., as results rather than causes of the localized folding and convolution with which they are associated.

Three distinctly larger and more continuous faults also were recognized within the mapped area. They are well exposed on the sea cliff that fringes Diablo Cove (refer to Figure 2.5-8), and each lies within a zone of moderately to severely contorted fine-grained Monterey strata. Each is actually a zone, 6 inches to several feet wide, within which two or more subparallel tight breaks are marked by slickensides, 1/4-inch or less of gouge, and local stringers of gypsum. None of these breaks appears to be systematically related to individual folds within the adjoining rocks. None of them extends upward into the overlying blanket of Quaternary terrace deposits.

One of these faults, exposed on the north side of the cove, trends north-northwest essentially parallel to the flanking Monterey beds, but it dips more steeply than these beds. Another, exposed on the east side of the cove, trends east-southeast and is essentially vertical; thus, it is essentially parallel to the structure of the host Monterey section. Neither of these faults projects toward the ground intended for power plant construction. The third fault, which appears on the sea cliff at the mouth of Diablo Canyon, trends northeast and projects toward the ground in the northernmost part of the power plant site. It dips northward somewhat more steeply than the adjacent strata.

Total displacement is not known for any of these three faults on the basis of natural exposures, but it could amount to as much as tens of feet. That these breaks are not major features, however, is strongly suggested by their sharpness, by the thinness of gouge along individual surfaces of slippage, and by the essential lack of correlation between the highly irregular geometry of deformation in the enclosing strata and any directions of movement along the slip surfaces.

The possibility that these surfaces are late-stage expressions of much larger-scale faulting at this general locality was tested by careful examination of the deformed rocks that they transect. On megascopic scales, the rocks appear to have been deformed much more by flexing than by rupture and slippage, as evidenced by local continuity of numerous thin beds that denies the existence of pervasive faulting within much of the ground in question. That the finer-grained rocks are not themselves fault gouged was confirmed by examination of 34 samples under the microscope.

Sedimentary layering, recognized in 27 of these samples, was observed to be grossly continuous even though dislocated here and there by tiny fractures. Moreover, nearly all the samples were found to contain shards of volcanic glass and/or the tests of foraminifera; some of these delicate components showed effects of microfracturing and a few had been offset a millimeter or less along tiny shear surfaces, but none appeared to have been smeared out or partially obliterated by intense shearing or grinding. Thus, the three larger faults in the area evidently were superimposed upon ground that already had been deformed primarily by small-scale and locally very intense folding rather than by pervasive grinding and milling.

It is not known whether these faults were late-stage results of major folding in the region or were products of independent tectonic activity. In either case, they are relatively ancient features, as they are capped without break by the Quaternary terrace deposits exposed along the upper part of the sea cliff. They probably are not large-scale elements of regional structure, as examination of the nearest areas of exposed bedrock along their respective landward projections revealed no evidence of substantial offsets among recognizable stratigraphic units.

Seaward projection of one or more of these faults might be taken to explain a possible large offset of the Obispo Tuff units exposed on North Point and South Point. The notion of such an offset, however, would rest upon the assumption that these two units are displaced parts of an originally continuous body, for which there is no real evidence. Indeed, the two tuff units are bounded on their northerly sides by lithologically different parts of the Monterey Formation; hence, they were clearly originally emplaced at different stratigraphic levels and are not directly correlative.

2.5.2.2.5 Geological Relationships at the Units 1 and 2 Power Plant Site

2.5.2.2.5.1 Geologic Investigations at the Site

The geologic relationships at DCPP site have been studied in terms of both local and regional stratigraphy and structure, with an emphasis on relationships that could aid in dating the youngest tectonic activity in the area. Geologic conditions that could affect the design, construction, and performance of various components of the plant installation also were identified and evaluated. The investigations were carried out in three main phases, which spanned the time between initial site selection and completion of foundation construction.

2.5.2.2.5.2 Feasibility Investigation Phase

Work directed toward determining the pertinent general geologic conditions at the plant site comprised detailed mapping of available exposures, limited hand trenching in areas with critical relationships, and petrographic study of the principal rock types. The results of this feasibility program were presented in a report that also included recommendations for determining suitability of the site in terms of geologic conditions. Information from this early phase of studies is included in the preceding four sections and illustrated in Figures 2.5-8, 2.5-9, and 2.5-10.

2.5.2.2.5.3 Suitability Investigation Phase

The record phase of investigations was directed toward testing and confirming the favorable judgments concerning site feasibility. Inasmuch as the principal remaining uncertainties involved structural features in the local bedrock, additional effort was made to expose and map these features and their relationships. This was accomplished through excavation of large trenches on a grid pattern that extended throughout the plant area, followed by photographing the trench walls and logging the exposed geologic features. Large-scale photographs were used as a mapping base, and the recorded data were then transferred to controlled vertical sections at a scale of 1 inch = 20 feet. The results of this work were reported in three supplements to the original geologic report (Reference 1). Supplementary Reports I and III presented data and interpretation based on trench exposures in the areas of the Unit 1 and Unit 2 installations, respectively. Supplementary Report II described the relationships of small bedrock faults exposed in the exploratory trenches and in the nearby sea cliff.
During these suitability investigations, special attention was given to the contact between bedrock and overlying terrace deposits in the plant site area. It was determined that none of the discontinuities present in the bedrock section displaces either the erosional surface developed across the bedrock or the terrace deposits that rest upon this surface. The pertinent data are presented farther on in this section and illustrated in Figures 2.5-11, 2.5-12, 2.5-13, and 2.5-14.

2.5.2.2.5.4 Construction Geology Investigation Phase

Geologic work done during the course of construction at the plant site spanned an interval of 5 years, which encompassed the period of large-scale excavation. It included detailed mapping of all significant excavations, as well as special studies in some areas of rock bolting and other work involving rock reinforcement and temporary instrumentation. The mapping covered essentially all parts of the area to be occupied by structures for Units 1 and 2, including the excavations for the circulating water intake and outlet, the turbine-generator building, the auxiliary building, and the containment structures. The results of this mapping are described farther on and illustrated in Figures 2.5-15 and 2.5-16.

2.5.2.2.5.5 Exploratory Trenching Program, Unit 1 Site

Four exploratory trenches were cut beneath the main terrace surface at the power plant site, as shown in Figures 2.5-8, 2.5-11, 2.5-12, and 2.5-13. Trench AF (Trench A), about 1080 feet long, extended in a north-northwesterly direction and thus was roughly parallel to the nearby margin of Diablo Cove. Trench BE (Trench B), 380 feet long, was parallel to Trench A and lay about 150 feet east of the northerly one-third of the longer trench. Trenches C and D, 450 and 490 feet long, respectively were nearly parallel to each other, 130 to 150 feet apart, and lay essentially normal to Trenches A and B. The two pairs of trenches crossed each other to form a "#" pattern that would have been symmetrical were it not for the long southerly extension of Trench A. They covered the area intended for Unit 1 power plant construction, and the intersection of Trenches B and C coincided in position with the center of the Unit 1 nuclear reactor structure.

All four trenches, throughout their aggregate length of approximately 2400 feet, revealed a section of surficial deposits and underlying bedrock that corresponds to the two-ply sequence of surficial deposits and Monterey strata exposed along the sea cliff in nearby Diablo Cove. The trenches ranged in depth from 10 feet to nearly 40 feet, and all had sloping sides that gave way downward to essentially vertical walls in the bedrock encountered 3 to 8 feet above their floors.

To facilitate detailed geologic mapping, the easterly walls of Trenches A and B and the southerly walls of Trenches C and D were trimmed to near-vertical slopes extending upward from the trench floors to levels well above the top of bedrock. These walls subsequently were scaled back by means of hand tools in order to provide fresh, clean exposures prior to mapping of the contact between bedrock and overlying unconsolidated materials.

1. Bedrock

The bedrock that was continuously exposed in the lowest parts of all the exploratory trenches lies within a portion of the Montery Formation characterized by a preponderance of sandstone. It corresponds to the part of the section that crops out in lower Diablo Canyon and along the sea cliff souteastward from the canyon mouth. The sandstone ranges from light gray through buff to light reddish brown, from silty to markedly tuffaceous, and from thin-bedded and platy to massive. The distribution and thickness of beds can be readily appraised from sections along Trenches A and B (Figure 2.5-12) that show nearly all individual bedding surfaces that could be recognized on the ground.

The sandstone ranges from very hard to moderately soft, and some of it feels slightly punky when struck with a pick. All of it is, however, firm and very compact. In general, the most platy parts of the sequence are also the hardest, but the soundest rock in the area is almost massive sandstone of the kind that underlies the site of the intended reactor structure. This rock is well exposed on the nearby hillslope adjoining the main terrace area, where it has been markedly resistant to erosion and stands out as distinct low ridges.

Tuff, consisting chiefly of altered volcanic glass, forms irregular sills and dikes in several parts of the bedrock section. This material, generally light gray to buff, is compact but distinctly softer than the enclosing sandstone. Individual bodies are 1/2 inch to 4 feet thick. They are locally abundant in Trench C west of Trench A, and in Trench A southward beyond the end of the section in Figure 2.5-12. They are very rare or absent in Trenches B and D, and in the easterly parts of Trench C and the northerly parts of Trench A. These volcanic rocks probably are related to the Obispo Tuff as described earlier, but all known masses of typical Obispo rocks in this area lie at considerable distances west and south of the ground occupied by the trenches.

2. Bedrock Structure

The stratification of the Monterey rocks dips northward wherever it was observable in the trenches, in general, at angles of 35 to 55°. Thus, the bedrock beneath the power plant site evidently lies on the southerly flank of the major syncline noted and described earlier. Zones of convolution and other expressions of locally intense folding were not recognized, and probably are much less common in this general part of the section than in other, previously described parts that include intervals of softer and more shaley rocks.

Much of the sandstone is traversed by fractures. Planar, curving, and irregular surfaces are well represented, and, in places, they are abundant and closely spaced. All prominent fractures and many of the minor and discontinuous ones are shown in the sections of Figure 2.5-12. Also shown in these sections are all recognized slip joints, shear surfaces, and faults, i.e., all surfaces along which the bedrock has been displaced. Such features are most abundant in Trenches A and C near their intersection, in Trench D west of the intersection with Trench A, and near the northerly end of Trench B.

Most of the surfaces of movement are hairline features with or without thin films of clay and/or gypsum. Displacements range from a small fraction of an inch to several inches. The other surfaces are more prominent, with well-defined zones of gouge and fine-grained breccia ordinarily 1/8 inch or less in thickness. Such zones were observed to reach a maximum thickness of nearly 1/2 inch along two small faults, but only as local lenses or pockets. Exposures were not sufficiently extensive in three dimensions for definitely determining the magnitude of slip along the more prominent faults, but all of these breaks appeared to be minor features. Indeed, no expressions of major faulting were recognized in any of the trenches despite careful search, and the continuous bedrock exposures precluded the possibility that such features could have been readily overlooked.

A northeast-trending fault that appears on the sea cliff at the mouth of Diablo Canyon projects toward the ground in the northernmost part of the power plant site, as noted in a foregoing section. No zone of breaks as prominent as this one was identified in the trench exposures, and any distinct northeastward continuation of the fault would necessarily lie north of the trenched ground. Alternatively, this fault might well separate northeastward into several smaller faults; some or all of these could correspond to some or all of the breaks mapped in the northerly parts of Trenches A and B.

3. Terrace Deposits

Marine terrace deposits of Pleistocene age form a cover, generally 2 to 5 feet thick, over the bedrock that lies beneath the power plant site. This cover was observed to be continuous in Trench C and the northerly part of Trench A, and to be nearly continuous in the other two trenches. Its lithology is highly variable, and includes bouldery rubble, loose beach sand, pebbly silt, silty to clayey sand with abundant shell fragments, and soft clay derived from underlying tuffaceous rocks. Nearly all of these deposits are at least sparsely fossiliferous, and, in a few places, they consist mainly of shells and shell fragments. Vertebrate fossils, chiefly vertebral and rib materials representing large marine mammals, are present locally; recognized occurrences are designated by the symbol X in the sections of Figure 2.5-12.

At the easterly ends of Trenches C and D, the marine deposits intergrade and intertongue in a landward direction with thicker and coarser accumulations of poorly sorted debris. This material evidently is talus that was formed along the base of an ancient sea cliff or other shoreline slope. In some places, the marine deposits are overlain by nonmarine terrace sediments with a sharp break, but elsewhere the contact between these two kinds of deposits is a dark colored zone, a few inches to as much as 2 feet thick, that appears to represent a soil developed on the marine section. Fragments of these soily materials appear here and there in the basal parts of the nonmarine section.

The nonmarine sediments that were exposed in Trenches B, C, and D and in the northerly part of Trench A are mainly alluvial deposits derived in ancient times from Diablo Canyon. They consist of numerous tabular fragments of Monterey rocks in a relatively dark colored silty to clayey matrix, and, in general, they are distinctly bedded and moderately to highly compact. As indicated in the sections of Figure 2.5-12, they thicken progressively in a north-northeastward direction, i.e., toward their principal source, the ancient mouth of Diablo Canyon.

Slump, creep, and slope-wash deposits, which constitute the youngest major element of the terrace section, overlie the alluvial fan gravels and locally are interlayered with them. Where the gravels are absent, as in the southerly part of Trench A, this younger cover rests directly upon bedrock. It is loose and uncompacted, internally chaotic, and is composed of fragments of Monterey rocks in an abundant dark colored clayey matrix.

All the terrace deposits are soft and unconsolidated, and hence are much less resistant to erosion than is the underlying bedrock. Those appearing along the walls of exploratory trenches were exposed to heavy rainfall during two storms, and showed some tendency to wash and locally to rill. Little slumping and no gross failure were noted in the trenches, however, and it was not anticipated that these materials would cause special problems during construction of a power plant.

4. Interface Between Bedrock and Surficial Deposits

As once exposed continuously in the exploratory trenches, the contact between bedrock and overlying terrace deposits represents a broad wave-cut platform of Pleistocene age. This buried surface of ancient marine erosion ranges in altitude between extremes of 82 and 100 feet, and more than three-fourths of it lies within the more limited range of 90 to 100 feet. It terminates eastward against a moderately steep shoreline slope, the lowest parts of which were encountered at the extreme easterly ends of Trenches C and D, and beyond this slope is an older buried bench at an altitude of 120 to 130 feet.

Available exposures indicate that the configuration of the erosional platform is markedly similar, over a wide range of scales, to that of the platform now being cut approximately at sea level along the present coast. Grossly viewed, it slopes very gently in a seaward (westerly) direction and is marked by broad, shallow channels and by upward projections that must have appeared as low spines and reefs when the bench was being formed (Figures 2.5-12 and 2.5-13). The most prominent reef, formerly exposed in Trenches B and D at and near their intersection, is a wide, westerly-trending projection that rises 5 to 15 feet above neighboring parts of the bench surface. It is composed of massive sandstone that was relatively resistant to the ancient wave erosion.

As shown in the sections and sketches of Figure 2.5-12, the surface of the platform is nearly planar in some places but elsewhere is highly irregular in detail. The small-scale irregularities, generally 3 feet or less in vertical extent, including knob, spine, and rib like projections and various wave-scoured pits, crevices, notches, and channels. The upward projections clearly correspond to relatively hard, resistant beds or parts of beds in the sandstone section. The depressions consistently mark the positions of relatively soft silty or shaley sandstone, of very soft tuffaceous rocks, or of extensively jointed rocks. The surface traces of most faults and some of the most prominent joints are in sharp depressions, some of them with overhanging walls. All these irregularities of detail have modern analogues that can be recognized on the bedrock bench now being cut along the margins of Diablo Cove.

The interface between bedrock and overlying surficial deposits is of particular interest in the trenched area because it provides information concerning the age of youngest fault movements within the bedrock section. This interface is nowhere offset by faults revealed in the trenches, but instead has been developed irregularly across these faults after their latest movements. The consistency of this general relationship was established by highly detailed tracing and inspection of the contact as freshly exhumed by scaling of the trench walls. Gaps in exposure of the interface necessarily were developed at the four intersections of trenches; at these localities, the bedrock was carefully laid bare so that all joints and faults could be recognized and traced along the trench floors to points where their relationships with the exposed interface could be determined.

Corroborative evidence concerning the age of the most recent fault displacements stems from the marine deposits that overlie the bedrock bench and form the basal part of the terrace section. That these deposits rest without break across the traces of faults in the underlying bedrock was shown by the continuity of individual sedimentary beds and lenses that could be clearly recognized and traced.

Further, some of the faults are directly capped by individual boulders, cobbles, pebbles, shells, and fossil bones, none of which have been affected by fault movements. Thus, the most recent fault displacements in the plant site area occurred prior to marine planation of the bedrock and deposition of the overlying terrace sediments. As pointed out earlier, the age of the most recent faulting in this area is therefore at least 80,000 years and more probably at least 120,000 years. It might be millions of years.

2.5.2.2.5.6 Exploratory Trenching Program, Unit 2 Site

Eight additional trenches were cut beneath the main terrace surface south of Diablo Canyon (Figure 2.5-13) in order to extend the scope of subsurface exploration to include all ground in the Unit 2 plant site. As in the area of the Unit 1 plant site, the trenches formed two groups; those in each group were parallel with one another and were oriented nearly normal to those of the other group. The excavations pertinent to the Unit 2 plant site can be briefly identified as follows:

1. North-northwest Alignment

a. Trench EJ, 240 feet long, was a southerly extension of older Trench BE (originally designated as Trench B).

b. Trench WU, 1300 feet long, extended southward from Trench DG (originally designated as Trench D), and its northerly part lay about 65 feet east of Trench EJ. The northernmost 485 feet of this trench was mapped in connection with the Unit 2 trenching program.

c. Trench MV, 700 feet long, lay about 190 feet east of Trench WU. The northernmost 250 feet of this trench was mapped in connection with the Unit 2 trenching program.

d. Trench AF (originally designated as Trench A) was mapped earlier in connection with the detailed study of the Unit 1 plant site. A section for this trench, which lay about 140 feet west of Trench EJ, was included with others in the report on the Unit 1 trenching program.

2. East-northeast Alignment

a. Trench KL, about 750 feet long, lay 180 feet south of Trench DG (originally designated as Trench D) and crossed Trenches AF, EJ, and WU.

b. Trench NO, about 730 feet long, lay 250 feet south of Trench KL and crossed Trenches AF, WU, and MV.

These trenches, or parts thereof, covered the area intended for the Unit 2 power plant construction, and the intersection of Trenches WU and KL coincided in position with the center of the Unit 2 nuclear reactor structure.

All five additional trenches, throughout their aggregate length of nearly half a mile, revealed a section of surficial deposits and underlying Monterey bedrock that corresponded to the two-ply sequence of surficial deposits and Monterey strata exposed in the older trenches and along the sea cliff in nearby Diablo Cove. The trenches ranged in depth from 10 feet (or less along their approach ramps) to nearly 35 feet, and all had sloping sides that gave way downward to essentially vertical walls in the bedrock encountered 3 to 22 feet above their floors. To facilitate detailed geologic mapping, the easterly walls of Trenches EJ, WU, and MV and the southerly walls of Trenches KL and NO were trimmed to near-vertical slopes extending upward from the trench floors to levels well above the top of bedrock. These walls subsequently were scaled back by means of hand tools in order to provide fresh, clean exposures prior to mapping of the contact between bedrock and overlying unconsolidated materials.

The geologic sections shown in Figures 2.5-12 and 2.5-13 correspond in position to the vertical portions of the mapped trench walls. Relationships exposed at higher levels on sloping portions of the trench walls have been projected to the vertical planes of the sections. Centerlines of intersecting trenches are shown for convenience, but the planes of the geologic sections do not contain the centerlines of the respective trenches.

3. Bedrock

The bedrock that was continuously exposed in the lowest parts of all the exploratory trenches lies within a part of the Monterey Formation characterized by a preponderance of sandstone. It corresponds to the portion of the section that crops out along the sea cliff southward from the mouth of Diablo Canyon. The sandstone is light to medium gray where fresh, and light gray to buff and reddish brown where weathered. It ranges from silty to markedly tuffaceous, with tuffaceous units tending to dominate southward and southwestward from the central parts of the trenched area (refer to geologic section in Figure 2.5-13). Much of the sandstone is thin-bedded and platy, but the most siliceous parts of the section are characterized by a strata a foot or more in thickness. Individual beds commonly are well defined by adjacent thin layers of more silty material.

Bedding is less distinct in the more tuffaceous parts of the section, some of which seem to be almost massive. These rocks typically are broken by numerous tight fractures disposed at high angles to one another so that, where weathered, their appearance is coarsely blocky rather than layered.

As broadly indicated in the geologic sections, the sandstone ranges from very hard to moderately soft, and some of it feels slightly punky when struck with a pick. All of it, however, is firm and very compact. In general, the most platy parts of the sequence are relatively hard, but the hardest and soundest rock in the area is thick-bedded to almost massive sandstone of the kind at and immediately north of the site for the intended reactor structure. This resistant rock is well exposed as distinct low ridges on the nearby hillslope adjoining the main terrace area.

Tuff, consisting chiefly of altered volcanic glass, is abundant within the bedrock section. Also widely scattered, but much less abundant, is tuff breccia, consisting typically of small fragments of older tuff, pumice, or Monterey rocks in a matrix of fresh to altered volcanic glass. These materials, which form sills, dikes, and highly irregular intrusive masses, are generally light gray to buff, gritty, and compact but distinctly softer than much of the enclosing sandstone. Individual bodies range from stringers less than a quarter of an inch thick to bulbous or mushroom-shaped masses with maximum exposed dimensions measured in tens of feet. As shown on the geologic sections, they are abundant in all the trenches.

These volcanic rocks probably are related to the Obispo Tuff, large masses of which are well exposed west and south of the trenched ground. The bodies exposed in the trenches doubtless represent a rather lengthy period of Miocene volcanism, during which the Monterey strata were repeatedly invaded by both tuff and tuff breccia. Indeed, several of the mapped tuff units were themselves intruded by dikes of younger tuff, as shown, for example, in Sections KL and NO.

4. Bedrock Structure

The stratification of the Monterey rocks dips northward wherever it was observable in the trenches, in general, at angles of 45 to 85°. The steepness of dip increases progressively from north to south in the trenched ground, a relationship also noted along the sea cliff southward from the mouth of Diablo Canyon. Thus, the bedrock beneath the power plant site evidently lies on the southerly flank of the major syncline that was described previously. Zones of convolution and other expressions of locally intense folding were not recognized, and they probably are much less common in this general part of the section than in other (previously described) parts that include intervals of softer and more shaley rocks.

Much of the sandstone is traversed by fractures. Planar, curving, and irregular surfaces are well represented, and in places they are abundant and closely spaced. All prominent fractures and nearly all of the minor and discontinuous ones are shown on the geologic sections (Figure 2.5-13). Also shown in these sections are all recognized shear surfaces, faults, and other discontinuities along which the bedrock has been displaced. Such features are nowhere abundant in the trench exposures.

Most of the surfaces of movement are hairline breaks with or without thin films of clay, calcite, and/or gypsum. Displacements range from a small fraction of an inch to several inches. A few other surfaces are more prominent, with well-defined zones of fine-grained breccia and/or infilling mineral material ordinarily 1/8 inch or less in thickness. Such zones were observed to reach maximum thicknesses of 3/8 to 1/2 inch along three small faults, but only as local lenses or pockets.

Exposures are not sufficiently extensive in three dimensions for definitely determining the magnitude of slip along all the faults, but for most of them it is plainly a few inches or less. None of them appears to be more than a minor break in a bedrock section that has been folded on a large scale. Indeed, no expressions of major faulting were recognized in any of the trenches despite careful search, and the continuous bedrock exposures preclude the possibility that such features could be readily overlooked.

Most surfaces of past movement probably were active during times when the Monterey rocks were being deformed by folding, when rupture and some differential movements would be expected in a section comprising such markedly differing rock types. Some of the fault displacements may well have been older, as attested in two places by relationships involving small faults, the Monterey rocks, and tuff.

In Trench WU south of Trench KL, for example, sandstone beds were seen to have been offset about a foot along a small fault. A thin sill of tuff occupies the same stratigraphic horizon on opposite sides of this fault, but the sill has not been displaced by the fault. Instead, the tuff occupies a short segment of the fault to effect the slight jog between its positions in the strata on either side. Intrusion of the tuff plainly postdated all movements along this fault.

5. Terrace Deposits

Marine terrace deposits of Pleistocene age form covers, generally 2 to 5 feet thick, but locally as much as 12 feet thick, over the bedrock that lies beneath the Unit 2 plant site. These covers were observed to be continuous in some parts of all the trenches, and thin and discontinuous in a few other parts. Elsewhere, the marine sediments were absent altogether, as in the lower and more southerly parts of Trenches EJ and WU and in the lower and more westerly parts of Trenches KL and NO.

The range in lithology of these deposits is considerable, and includes bouldery rubble, gravel composed of well-rounded fragments of shells and/or Monterey rocks, beach sand, loose accumulations of shells, pebbly silt, silty to clayey sand with abundant shell fragments, and soft clay derived from underlying tuffaceous rocks. Nearly all of the deposits are at least sparsely fossiliferous, and many of them contain little other than shell material. Vertebrate fossils, chiefly vertebral and rib materials representing large marine mammals, are present locally.

The trenches in and near the site of the reactor structure exposed a buried narrow ridge of hard bedrock that once projected westward as a bold promontory along an ancient sea coast, probably at a time when sea level corresponded approximately to the present 100 foot contour (refer to Figure 2.5-11). Along the flanks of this promontory and the face of an adjoining buried sea cliff that extends southeastward through the area in which Trenches MV and NO intersected, the marine deposits intergrade and intertongue with thicker and coarser accumulations of poorly sorted debris. This rubbly material evidently is talus that was formed and deposited along the margins of the ancient shoreline cliff.

Similar gradations of older marine deposits into older talus deposits were observable at higher levels in the easternmost parts of Trenches KL and NO, where the rubbly materials doubtless lie against a more ancient sea cliff that was formed when sea level corresponded to the present 140 foot contour. The cliff itself was not exposed, however, as it lies slightly beyond the limits of trenching.

In many places, the marine covers are overlain by younger nonmarine terrace sediments with a sharp break, but elsewhere the contact between these two kinds of deposits is a zone of dark colored material, a few inches to as much as 6 feet thick, that represents weathering and development of soils on the marine sections. Fragments of these soily materials are present here and there in the basal parts of the nonmarine section. Over large areas, the porous marine deposits have been discolored through infiltration by fine-grained materials derived from the overlying ancient soils.

The nonmarine accumulations, which form the predominant fraction of the entire terrace cover, consist mainly of slump, creep, and slope-wash debris that is characteristically loose, uncompacted, and internally chaotic. These relatively dark colored deposits are fine grained and clayey, but they contain sparse to very abundant fragments of Monterey rocks generally ranging from less than an inch to about 2 feet in maximum dimension. Toward Diablo Canyon they overlie and, in places, intertongue with silty to clayey gravels that are ancient contributions from Diablo Creek when it flowed at levels much higher than its present one. These "dirty" alluvial deposits appeared only in the most northerly parts of the more recently trenched terrace area, and they are not distinguished from other parts of the nonmarine cover on the geologic sections (Figure 2.5-13).

All the terrace deposits are soft and unconsolidated, and hence are much less resistant to erosion than is the underlying bedrock. Those appearing along the walls of the exploratory trenches showed some tendency to wash and locally to rill when exposed to heavy rainfall, but little slumping and no gross failure were noted in the trenches.

6. Interface Between Bedrock and Surficial Deposits

As exposed continuously in the exploratory trenches, the contact between bedrock and overlying terrace deposits represents two wave-cut platforms and intervening slopes, all of Pleistocene age. The broadest surface of ancient marine erosion ranges in altitude from 80 to 105 feet, and its shoreward margin, at the base of an ancient sea cliff, lies uniformly within 5 feet of the 100 foot contour. A higher, older, and less extensive marine platform ranges in altitude from 130 to 145 feet, and most of it lies within the ranges of 135 to 140 feet. As noted previously, these are two of several wave-cut benches in this coastal area, each of which terminates eastward against a cliff or steep shoreline slope and westward at the upper rim of a similar but younger slope.

Available exposures indicate that the configurations of the erosional platforms are markedly similar, over a wide range of scales, to that of the platform now being cut approximately at sea level along the present coast. Grossly viewed, they slope very gently in a seaward (westerly) direction and are marked by broad, shallow channels and by upward projections that must have appeared as low spines and reefs when the benches were being formed. The most prominent reefs, which rise from a few inches to about 5 feet above neighboring parts of the bench surfaces, are composed of hard, thick-bedded sandstone that was relatively resistant to ancient wave erosion.
As shown in the geologic sections (Figure 2.5-13), the surfaces of the platforms are nearly planar in some places but elsewhere are highly irregular in detail. The small scale irregularities, generally 3 feet or less in vertical extent, include knob-, spine-, and rib-like projections and various wave-scoured pits, notches, crevices, and channels. Most of the upward projections closely correspond to relatively hard, resistant beds or parts of beds in the sandstone section. The depressions consistently mark the positions of relatively soft silty or shaley sandstone, of very soft tuffaceous rocks, or of extensively jointed rocks. The surface traces of most faults and some of the most prominent joints are in sharp depressions, some of them with overhanging walls. All these irregularities of detail have modern analogues that can be recognized on the bedrock bench now being cut along the margins of Diablo Cove.

The interface between bedrock and overlying surficial deposits provides information concerning the age of youngest fault movements within the bedrock section. This interface is nowhere offset by faults that were exposed in the trenches, but instead has been developed irregularly across the faults after their latest movements. The consistency of this general relationship was established by highly detailed tracing and inspection of the contact as freshly exhumed by scaling of the trench walls. Gaps in exposure of the interface necessarily were developed at the intersections of trenches as in the exploration at the Unit 1 site. At such localities, the bedrock was carefully laid bare so that all joints and faults could be recognized and traced along the trench floors to points where their relationships with the exposed interface could be determined.

Corroborative evidence concerning the age of the most recent fault displacements stems from the marine deposits that overlie the bedrock bench and form a basal part of the terrace section. That these deposits rest without break across the traces of faults in the underlying bedrock was shown by the continuity of individual sedimentary beds and lenses that could be clearly recognized and traced. As in other parts of the site area, some of the faults are directly capped by individual boulders, cobbles, pebbles, shells, and fossil bones, none of which have been affected by fault movements. Thus, the most recent fault displacements in the plant site area occurred before marine planation of the bedrock and deposition of the overlying terrace sediments.

The age of the most recent faulting in this area is therefore at least 80,000 years. More probably, it is at least 120,000 years, the age most generally assigned to these terrace deposits along other parts of the California coastline. Evidence from the higher bench in the plant site area indicates a much older age, as the unfaulted marine deposits there are considerably older than those that occupy the lower bench corresponding to the 100 foot terrace. Moreover, it can be noted that ages thus determined for most recent fault displacements are minimal rather than absolute, as the latest faulting actually could have occurred millions of years ago.

During the Unit 2 exploratory trenching program, special attention was directed to those exposed parts of the wave-cut benches where no marine deposits are present, and hence where there are no overlying reference materials nearly as old as the benches themselves. At such places, the bedrock beneath each bench has been weathered to depths ranging from less than 1 inch to at least 10 feet, a feature that evidently corresponds to a lengthy period of surface exposure from the time when the bench was abandoned by the sea to the time when it was covered beneath encroaching nonmarine deposits derived from hillslopes to the east.

Stratification and other structural features are clearly recognizable in the weathered bedrock, and they obviously have exercised some degree of control over localization of the weathering. Moreover, in places where upward projections of bedrock have been gradually bent or rotationally draped in response to weathering and creep, their contained fractures and surfaces of movement have been correspondingly bent. Nowhere in such a section that has been disturbed by weathering have the materials been cut by younger fractures that would represent straight upward projections of breaks in the underlying fresh rocks. Nor have such fractures been observed in any of the overlying nonmarine terrace cover.

Thus, the minimum age of any fault movement in the plant site area is based on compatible evidence from undisplaced reference features of four kinds: (a) Pleistocene wave-cut benches developed on bedrock, (b) immediately overlying marine deposits that are very slightly younger, (c) zones of weathering that represent a considerable span of subsequent time, and (d) younger terrace deposits of nonmarine origin.

2.5.2.2.5.7 Bedrock Geology of the Plant Foundation Excavations

Bedrock was continuously exposed in the foundation excavations for major structural components of Units 1 and 2. Outlines and invert elevations of these large openings, which ranged in depth from about 5 to nearly 90 feet below the original ground surface, are shown in Figures 2.5-15 and 2.5-16. The complex pattern of straight and curved walls with various positions and orientations provided an excellent three-dimensional representation of bedrock structure. These walls were photographed at large scales as construction progressed, and the photographs were used directly as a geologic mapping base. The largest excavations also were mapped in detail on a surveyed planimetric base.

Geologic mapping of the plant excavations confirmed the conclusions based on earlier investigations at the site. The exposed section of Monterey strata was found to correspond in lithology and structure to what had been predicted from exposures at the mouth of Diablo Canyon, along the sea cliffs in nearby Diablo Cove, and in the test trenches. Thus, the plant foundation is underlain by a moderately to steeply north-dipping sequence of thin to thick bedded sandy mudstone and fine-grained sandstone. The rocks at these levels are generally fresh and competent, as they lie below the zone of intense near-surface weathering.

Several thin interbeds of claystone were exposed in the southwestern part of the plant site in the excavations for the Unit 2 turbine-generator building, intake conduits, and outlet structure. These beds, which generally are less than 6 inches thick, are distinctly softer than the flanking sandstone. Some of them show evidence of internal shearing.

Layers of tuffaceous sandstone and sills, dikes, and irregular masses of tuff and tuff breccia are present in most parts of the foundation area. They tend to increase in abundance and thickness toward the south, where they are relatively near the large masses of Obispo Tuff exposed along the coast south of the plant site.

Some of the tuff bodies are conformable with the enclosing sandstone, but others are markedly discordant. Most are clearly intrusive. Individual masses, as exposed in the excavations, range in thickness from less than 1 inch to about 40 feet. The tuff breccia, which is less abundant than the tuff, consists typically of small fragments of older tuff, pumice, or Monterey rocks in a matrix of fresh to highly altered volcanic glass. At the levels of exposure in the excavations, both the tuff and tuff breccia are somewhat softer than the enclosing sandstone.

The stratification of the Monterey rocks dips generally northward throughout the plant foundation area. Steepness of dips increases progressively and, in places, sharply from north to south, ranging from 10 to 15° on the north side of Unit 1 to 75 to 80° in the area of Unit 2. A local reversal in direction of dip reflects a small open fold or warp in the Unit 1 area. The axis of this fold is parallel to the overall strike of the bedding, and strata on the north limb dip southward at angles of 10 to 15°. The more general steepening of dips from north to south may reflect buttressing by the large masses of Obispo Tuff south of the plant site.

The bedrock of the plant area is traversed throughout by fractures, including various planar, broadly curving, and irregular breaks. A dominant set of steeply dipping to vertical joints trends northerly, nearly normal to the strike of bedding. Other joints are diversely oriented with strikes in various directions and dips ranging from 10° to vertical. Many fractures curve abruptly, terminate against other breaks, or die out within single beds or groups of beds.

Most of the joints are widely spaced, ranging from about 1 to 10 feet apart, but within several northerly trending zones, ranging in width from 10 to 20 feet, closely spaced near vertical fractures give the rocks a blocky or platy appearance. The fracture and joint surfaces are predominantly clean and tight, although some irregular ones are thinly coated with clay or gypsum. Others could be traced into thin zones of breccia with calcite cement.

Several small faults were mapped in the foundation excavations for Unit 1 and the outlet structure. A detailed discussion of these breaks and their relationship to faults that were mapped earlier along the sea cliff and in the exploratory trenches is included in the following section.

2.5.2.2.5.8 Relationships of Faults and Shear Surfaces

Several subparallel breaks are recognizable on the sea cliff immediately south of Diablo Canyon, where they transect moderately thick-bedded sandstone of the kind exposed in the exploratory trenches to the east. These breaks are nearly concordant with the bedrock stratification but, in general, they dip more steeply (refer to detailed structure section, Figure 2.5-14) and trend more northerly than the stratification. Their trend differs significantly from much of their mapped trace, as the trace of each inclined surface is markedly affected by the local steep topography. The indicated trend, which projects eastward toward ground north of the Unit 1 reactor site, has been summed from numerous individual measurements of strike on the sea cliff exposures, and it also corresponds to the trace of the main break as observed in nearly horizontal outcrop within the tidal zone west of the cliff.

The structure section shows all recognizable surfaces of faulting and shearing in the sea cliff that are continuous for distances of 10 feet or more. Taken together, they represent a zone of dislocation along which rocks on the north have moved upward with respect to those on the south as indicated by the attitude and roughness sense of slickensides. The total amount of movement cannot be determined by any direct means, but it probably is not more than a few tens of feet and could well be less than 10 feet. This is suggested by the following observed features:

(1) All individual breaks are sharp and narrow, and the strata between them are essentially undeformed except for their gross inclination.

(2) Some breaks plainly die out as traced upward along the cliff surface, and others merge with adjoining breaks. At least one well-defined break butts downward against a cross-break, which in turn butts upward against a break that branches and dies out approximately 20 feet away (refer to structure section, Figure 2.5-14, for details).

(3) Nearly all the breaks curve moderately to abruptly in the general direction of movement along them.

(4) Most of the breaks are little more than knife-edge features along which rock is in direct contact with rock, and others are marked by thin films of gouge. Maximum thickness of gouge anywhere observed is about 1/2 inch, and such exceptional occurrences are confined to short curving segments of the main break at the southerly margin of the zone.

(5) No fault breccia is present; instead, the zone represents transection of otherwise undeformed rocks by sharply-defined breaks. No bedrock unit is cut off and juxtaposed against a unit of different lithology along any of the breaks.

(6) Local prominence of the exposed breaks, and especially the main one, is due to slickensides, surface coatings of gypsum, and iron-oxide stains rather than to any features reflecting large-scale movements.

This zone of faulting cannot be regarded as a major tectonic element, nor is it the kind of feature normally associated with the generation of earthquakes. It appears instead to reflect second-order rupturing related to a marked change in dip of strata to the south, and its general sense of movement is what one would expect if the breaks were developed during folding of the Monterey section against what amounts to a broad buttress of Obispo Tuff farther south (refer to geologic map, Figure 2.5-8). That the fault and shear movements were ancient is positively indicated by upward truncation of the zone at the bench of marine erosion along the base of the overlying terrace deposits.

As indicated earlier, bedrock was continuously exposed along several exploratory trenches. This bedrock is traversed by numerous fractures, most of which represent no more than rupture and very small amounts of simple separation. The others additionally represent displacement of the bedrock, and the map in Figure 2.5-14 shows every exposed break in the initial set of trenches along which any amount of displacement could be recognized or inferred.

That the surfaces of movement constitute no more than minor elements of the bedrock structure was verified by detailed mapping of the large excavations for the plant structures. Detailed examination of the excavation walls indicated that the faults exposed in the sea cliff south of Diablo Canyon continue through the rock under the Unit 1 turbine-generator building, where they are expressed as three subparallel breaks with easterly trend and moderately steep northerly dips (Figure 2.5-15).
Stratigraphic separation along these breaks ranges from a few inches to nearly 5 feet, and, in general, decreases eastward on each of them. They evidently die out in the ground immediately west of the containment excavation, and their eastward projections are represented by several joints along which no offsets have occurred. Such joints, with eastward trend and northward dip, also are abundant in some of the ground adjacent to the faults on the south (Figure 2.5-15).

The easterly reach of the Diablo Canyon sea cliff faults apparently corresponds to the two most northerly of the north-dipping faults mapped in Trench A (Figure 2.5-14).

Dying out of these breaks, as established from subsequent large excavations in the ground east of where Trench A was located, explains and verifies the absence of faults in the exposed rocks of Trenches B and C. Other minor faults and shear surfaces mapped in the trench exposures could not be identified in the more extensive exposures of fresher rocks in the Unit 1 containment and turbine-generator building excavations. The few other minor faults that were mapped in these large excavations evidently are not sufficiently continuous to have been present in the exploratory trenches.

2.5.2.2.6 Site Engineering Properties

2.5.2.2.6.1 Field and Laboratory Investigations

In order to determine anticipated ground accelerations at the site, it was necessary to conduct field surveys and laboratory testing to evaluate the engineering properties of the materials underlying the site.

Bore holes were drilled into the rock upon which PG&E Design Class 1 structures are founded. The borings were located at or near the intersection of the then existing Unit 1 exploration trenches. (refer to Figures 2.5-11, 2.5-12, and 2.5-13 for exploratory trenching programs and boring locations.) These holes were cored continuously and representative samples were taken from the cores and submitted for laboratory testing.

The field work also included a reconnaissance to evaluate physical condition of the rocks that were exposed in trenches, and samples were collected from the ground surface in the trenches for laboratory testing. These investigations included seismic refraction measurements across the ground surface and uphole seismic measurements in the various drill holes to determine shear and compressional velocities of vertically propagated waves.

Laboratory testing, performed by Woodward-Clyde-Sherard & Associates, included unconfined compression tests, dynamic elastic moduli tests under controlled stress conditions, density and water content determinations, and Poisson's ratio tests. Tests were also carried out by Geo-Recon, Incorporated, to determine seismic velocities on selected rock samples in the laboratory. The results of seismic measurements in the field were used to construct a three-dimensional model of the subsurface materials beneath the plant site showing variations of shear wave velocity and compressional wave velocity both laterally and vertically. The seismic velocity data and elastic moduli determined from laboratory testing were correlated to determine representative values of elastic moduli necessary for use in dynamic analyses of structures.

Details of field investigations and results of laboratory testing and correlation of data are contained in Appendices 2.5A and 2.5B of Reference 27 in Section 2.3.

2.5.2.2.6.2 Summary and Correlation of Data

The foundation material at the site can be categorized as a stratified sequence of fine to very fine grained sandstone deeply weathered to an average elevation of 75 to 80 feet, mean sea level (MSL). The rock is closely fractured, with tightly closed or healed fractures generally present below elevation 75 feet. Compressional and shear wave velocity interfaces generally are at an average elevation of 75 feet, correlating with fracture conditions.

Time-distance plots and seismic velocity profiles presenting results of each seismic refraction line and time depth plots with results for each uphole seismic survey are included in Appendices 2.5A and 2.5B of Reference 27 in Section 2.3. Compressional wave velocities range from 2350 to 5700 feet per second and shear wave velocities from 1400 to 3600 feet per second as determined by the refraction survey. These same parameters range from 2450 to 9800 and 1060 to 6050 feet per second as determined by the uphole survey. For the Hosgri Evaluation an average shear wave velocity of 3600 feet per second is used at the foundation grade. An isometric diagram summarizing results of the refraction survey for Unit 1 is also included in Appendix 2.5A of Reference 27 in Section 2.3.

Table 1 of Appendix 2.5A of Reference 27 of Section 2.3 shows calculations of Poisson's ratio and Young's Modulus based on representative compressional and shear wave velocities from the field geophysical investigations and laboratory measurements of compressional wave velocities. Table 2 of Appendix 2.5A of the same reference presents laboratory test results including density, unconfined compressive strength, Poisson's ratio and calculated values for compressional and shear wave velocities, shear modulus, and constrained modulus. Secant modulus values in Table 2 were determined from cyclic stress-controlled laboratory tests.

Compressional wave velocity measurements were made in the laboratory of four selected core samples and three hand specimens from exposures in the trench excavations. Measured values ranged from 5700 to 9500 feet per second. A complete tabulation of these results can be found in Appendix 2.5A of Reference 27 of Section 2.3.

2.5.2.2.6.3 Dynamic Elastic Moduli and Poisson's Ratio

Laboratory test results are considered to be indicative of intact specimens of foundation materials. Field test results are considered to be indicative of the gross assemblage of foundation materials, including fractures and other defects. Load stress conditions are obtained by evaluating cyclic load tests. In-place load stress conditions and confinement of the material at depth are also influential in determining elastic behavior. Because of these considerations, originally recommended representative values for Young's Modulus of Elasticity and Poisson's ratio for the site were:

Depth Below Bottom of Trench	E	δ
0 to approximately 15 feet	44×10^6 lb/ft^2	0.20
Below 15 feet	148×10^6 lb/ft^2	0.18

A single value was selected for Young's Modulus below 15 feet because the initial analyses of the seismic response of the structures utilized a single value that was considered representative of the foundation earth materials as a whole.

More detailed seismic analyses were performed subsequent to the initial analyses. These analyses, discussed in Section 3.7.2, incorporated the finite element method and made it possible to model the rock beneath the plant site in a more refined manner by accounting for changes in properties with increasing depth. To determine the refined properties of the founding materials for these analyses, the test data were reviewed and consideration was given to: (a) strain range of the materials at the site, (b) overburden pressure and confinement, (c) load imposed by the structure, (d) observation of fracture condition and geometry of the founding rock in the open excavation, (e) decreases in Poisson's ratio with depth, and (f) significant advances in state-of-the-art techniques of testing and analysis in rock mechanics that had been made and which resulted in considerably more being known about the behavior of rock under seismic strains in 1970 than in 1968 or 1969.

For the purposes of developing the mathematical models that represented the rock mass, the foundation was divided into horizontal layers based on: (a) the estimated depth of disturbance of the foundation rock below the base of the excavation, (b) changes in rock type and physical condition as determined from bore hole logs, (c) velocity interfaces as determined by refraction geophysical surveys, and (d) estimated depth limit of fractures across which movement cannot take place because of confinement and combined overburden and structural load. Based on these considerations, the founding material properties as shown in Figure 2.5-19 were selected as being representative of the physical conditions in the founding rock.

2.5.2.2.6.4 Engineered Backfill

Backfill operations were carefully controlled to ensure stability and safety. All engineered backfill was placed in lifts not exceeding 8 inches in loose depth. Yard areas and roads were compacted to 95 percent relative compaction as determined by the method specified in ASTM D1557. Rock larger than 8 inches in its largest dimension that would not break down under the compactors was not permitted. Figures 2.5-17 and 2.5-18 show the plan and profile view of excavation and backfill for major plant structures.

2.5.2.2.6.5 Foundation Bearing Pressures

PG&E Design Class I structures were analyzed to determine the foundation pressures resulting from the combination of dead load, live load, and the double design earthquake (DDE). The maximum pressure was found to be 158 ksf and occurs under the containment structure foundation slab. This analysis assumed that the lateral seismic shear force will be transferred to the rock at the base of the slab which is embedded 11 feet into rock. This computed bearing pressure is considered conservative in that no passive lateral pressure was assumed to act on the sides of the slab. Based on the results of the laboratory tests of unconfined compressive strength of representative samples of rock at the site, which ranged from 800 to 1300 ksf, the calculated foundation pressure is well below the ultimate in situ rock bearing capacity.

Adverse hydrologic effects on the foundations of PG&E Design Class I structures (there are no PG&E Design Class I embankments) can be safely neglected at this site, since PG&E Design Class I structures are founded on a substantial layer of bedrock, and the groundwater level lies well below grade, at a level corresponding to that of Diablo Creek. Additionally, the computed factors of safety (minimum of 5 under DDE) of foundation pressures versus unconfined compressive strength of rock are sufficiently high to ensure foundation integrity in the unlikely event groundwater levels temporarily rose to foundation grade.

Soil properties such as grain size, Atterberg limits, and water content need not be considered since PG&E Design Class I structures and PG&E Design Class II structures housing PG&E Design Class I equipment are founded on rock.

2.5.3 VIBRATORY GROUND MOTION

2.5.3.1 Geologic Conditions of the Site and Vicinity

DCPP is situated at the coastline on the southwest flank of the San Luis Range, in the southern Coast Ranges of California. The San Luis Range branches from the main coastal mountain chain, the Santa Lucia Range, in the area north of the Santa Maria Valley and southeast of the plant site, and thence follows an alignment that curves toward the west. Owing to this divergence in structural grain, the range juts out from the regional coastline as a broad peninsula and is separated from the Santa Lucia Range by an elongated lowland that extends southeasterly from Morro Bay and includes Los Osos and San Luis Obispo Valleys. It is characterized by rugged west-northwesterly trending ridges and canyons, and by a narrow fringe of coastal terraces along its southwesterly flank.

Diablo Canyon follows a generally west-southwesterly course from the central part of the range to the north-central part of the terraced coastal strip. Detailed discussions of the lithology, stratigraphy, structure, and geologic history of the plant site and surrounding region are presented in Section 2.5.2.

2.5.3.2 Underlying Tectonic Structures

Evidence pertaining to tectonic and seismic conditions in the region of the DCPP site, developed during the original design phase, is summarized later in the section, and is illustrated in Figures 2.5-2, 2.5-3, 2.5-4, and 2.5-5. Table 2.5-1 includes a summary listing of the nature and effects of all significant historic earthquakes within 75 miles of the site that have been reported through the end of 1972. Table 2.5-2 shows locations of 19 selected earthquakes that have been investigated by S. W. Smith. Table 2.5-3 lists the principal faults in the region that were identified during the original design phase and indicates major elements of their histories of displacement, in geological time units.

Prior to the start of construction of DCPP, Benioff and Smith (reference 5) assessed the maximum earthquakes to be expected at the site, and John A. Blume and Associates (references 6 and 7) derived the site vibratory motions that could result from these maximum earthquakes, which form the basis of the Design Earthquake. An extensive discussion of the geology of the southern Coast Ranges, the western Transverse Ranges, and the adjoining offshore region is presented in Appendix 2.5D of Reference 27 of Section 2.3. Tectonic features of the central coastal region are discussed in Section 2.5.2.1.2, Regional Geologic and Tectonic Setting.

Additional information about the tectonic and seismic conditions was gathered during the Hosgri evaluation and LTSP evaluation phases, as discussed in Sections 2.5.3.9.3 and 2.5.3.9.4, respectively.

2.5.3.3 Behavior During Prior Earthquakes

Physical evidence that indicates the behavior of subsurface materials, strata, and structure during prior earthquakes is presented in Section 2.5.2.2.5. The section presents the findings of the exploratory trenching programs conducted at the site.

2.5.3.4 Engineering Properties of Materials Underlying the Site

A description of the static and dynamic engineering properties of the materials underlying the site is presented in Section 2.5.2.2.6, Site Engineering Properties.

2.5.3.5 Earthquake History

The seismicity of the southern Coast Ranges region is known from scattered records extending back to the beginning of the 19th century, and from instrumental records dating from about 1900. Detailed records of earthquake locations and magnitudes became available following installation of the California Institute of Technology and University of California (Berkeley) seismograph arrays in 1932.

A plot of the epicenters for all large historical earthquakes and for all instrumentally recorded earthquakes of Magnitude 4 or larger that have occurred within 200 miles of DCPP site, through the end of 1972, is given in Figure 2.5-2. Plots of all historically and instrumentally recorded epicenters and all mapped faults within about 75 miles of the site, known through the end of 1972, are shown in Figures 2.5-3 and 2.5-4.

A tabulated list of seismic events through the end of 1972, representing the computer printout from the Berkeley Seismograph Station records, supplemented with records of individual shocks of greater than Magnitude 4 that appear only in the Caltech records, is included as Table 2.5-1. Table 2.5-2 gives a summary of revised epicenters of a representative sample of earthquakes off the coast of California near San Luis Obispo, as determined by S. W. Smith.

2.5.3.6 Correlation of Epicenters With Geologic Structures

Studies of particular aspects of the seismicity of the southern Coast Ranges region have been made by Benioff and Smith, Richter, and Allen. From results of these studies, together with data pertaining to the broader aspects of the geology and seismicity of central and eastern California, it can be concluded that, although the southern Coast Ranges region may be subjected to vibratory ground motion from earthquakes originating along faults as distant as 200 miles or more, the region itself is traversed by faults capable of producing large earthquakes, and that the strongest shaking possible for sites within the region probably would be caused by earthquakes no more than a few tens of miles away. Therefore, only the seismicity of the southern Coast Ranges, the adjacent offshore area, and the western Transverse Ranges is reviewed in detail.

Figure 2.5-3 shows three principal concentrations of earthquake epicenters, three smaller or more diffuse areas of activity, and a scattering of other epicenters, for earthquakes recorded through 1972. The most active areas, in terms of numbers of shocks, are the reach of the San Andreas fault north of about 35°7' latitude, the offshore area near Santa Barbara, and the offshore Santa Lucia Bank area. Notable concentrations of epicenters also are located as occurring in Salinas Valley, at Point San Simeon, and near Point Conception. The scattered epicenters are most numerous in the general vicinities of the most active areas, but they also occur at isolated points throughout the region.

The reliability of the position of instrumentally located epicenters of small shocks in the central California region has been relatively poor in the past, owing to its position between the areas covered by the Berkeley and Caltech seismograph networks. A recent study by Smith, however, resulted in relocation of nineteen epicenters in the coastal and offshore region between the latitudes of Point Arguello and Point Sur. Studies by Gawthrop (reference 29) and reported in Wagner have led to results that seem to accord generally with those achieved by Smith.

The epicenters relocated by Smith and those recorded by Gawthrop are plotted in Figure 2.5-3. This plot shows that most of the epicenters recorded in the offshore region seem to be spatially associated with faults in the Santa Lucia Bank region, the East Boundary zone, and the San Simeon fault. Other epicenters, including ones for the 1952 Bryson shock, and several smaller shocks originally located in the offshore area, were determined to be centered on or near the Sur-Nacimiento fault north of the latitude of San Simeon.

2.5.3.7 Identification of Active Faults

Faults that have evidence of recent activity and have portions passing within 200 miles of the site, as known through the end of 1972, are identified in Section 2.5.2.1.2.

2.5.3.8 Description of Active Faults

Active faults that have any part passing within 200 miles of the site, as known through the end of 1972, are described in Section 2.5.2.1.2. Additional active faults were identified during the Hosgri and LTSP evaluation phases, as described in Sections 2.5.3.9.3 and 2.5.3.9.4, respectively.

2.5.3.9 Design and Licensing Basis Earthquakes

The seismic design and evaluation of DCPP is based on the earthquakes described in the following four subsections. Refer to Section 3.7 for the design criteria associated with the application of these earthquakes to the structures, systems, and components. The DE, DDE, and HE are design bases earthquakes and the LTSP is a licensing bases earthquake.

2.5.3.9.1 Design Earthquake

During the original design phase, Benioff and Smith, in reviewing the seismicity of the region around DCPP site, determined the maximum earthquakes that could reasonably be expected to affect the site. Their conclusions regarding the maximum size earthquakes that can be expected to occur during the life of the reactor are listed below:

(1) Earthquake A: A great earthquake may occur on the San Andreas fault at a distance from the site of more than 48 miles. It would be likely to produce surface rupture along the San Andreas fault over a distance of 200 miles with a horizontal slip of about 20 feet and a vertical slip of 3 feet. The duration of strong shaking from such an event would be about 40 seconds, and the equivalent magnitude would be 8.5.

(2) Earthquake B: A large earthquake on the Nacimiento (Rinconada) fault at a distance from the site of more than 20 miles would be likely to produce a 60 mile surface rupture along the Nacimiento fault, a slip of 6 feet in the horizontal direction, and have a duration of 10 seconds. The equivalent magnitude would be 7.25.

(3) Earthquake C: Possible large earthquakes occurring on offshore fault systems that may need to be considered for the generation of seismic sea waves are listed below:

Location	Length of Fault Break	Slip, feet	Magnitude	Distance to Site
Santa Ynez Extension	80 miles	10 horizontal	7.5	50 miles
Cape Mendocino, NW Extension of San Andreas fault	100 miles	10 horizontal	7.5	420 miles
Gorda Escarpment	40 miles	5 vertical or horizontal	7	420 miles

(4) Earthquake D: Should a great earthquake occur on the San Andreas fault, as described in "A" above, large aftershocks may occur out to distances of about 50 miles from the San Andreas fault, but those aftershocks which are not located on existing faults would not be expected to produce new surface faulting, and would be restricted to depths of about 6 miles or more and magnitudes of about 6.75 or less. The distance from the site to such aftershocks would thus be more than 6 miles.

The available information suggests that the faults in this region can be associated with contrasting general levels of seismic potential. These are as follows:

(1) Level I: Potential for great earthquakes involving surface faulting over distances on the order of 100 miles: seismic activity at this level should occur only on the reach of the San Andreas fault that extends between the locales of Cajon Pass and Parkfield. This was the source of the 1857 Fort Tejon earthquake, estimated to have been of Magnitude 8.

(2) Level II: Potential for large earthquakes involving faulting over distances on the order of tens of miles: seismic activity at this level can occur along offshore faults in the Santa Lucia Bank region (the likely source of the Magnitude 7.3 earthquake of 1927), and possibly along the Big Pine and Santa Ynez faults in the Transverse Ranges.

Although the Rinconada-San Marcos-Jolon, Espinosa, Sur-Nacimiento, and San Simeon faults do not exhibit historical or even Holocene activity indicating this level of seismic potential, the fault dimensions, together with evidence of late Pleistocene movements along these faults, suggest that they may be regarded as capable of generating similarly large earthquakes.

(3) Level III: Potential for earthquakes resulting chiefly from movement at depth with no surface faulting, but at least with some possibility of surface faulting of as much as a few miles strike length and a few feet of slip:

Seismic activity at this level probably could occur on almost any major fault in the southern Coast Ranges and adjacent regions.

From the observed geologic record of limited fault activity extending into Quaternary time, and from the historical record of apparently associated seismicity, it can be inferred that both the greater frequency of earthquake activity and larger shocks from earthquake source structures having this level of seismic potential probably will be associated with one of the relatively extensive faults. Faults in the vicinity of the San Luis Range that may be considered to have such seismic potential include the West Huasna, Edna, and offshore Santa Maria Basin East Boundary zone.

(4) Level IV: Potential for earthquakes and aftershocks resulting from crustal movements that cannot be associated with any near-surface fault structures: such earthquakes apparently can occur almost anywhere in the region.

This information forms the basis of the Design Earthquake, described in section 2.5.3.10.1.

2.5.3.9.2 Double Design Earthquake

During the original design phase, in order to assure adequate reserve seismic resisting capability of safety related structures, systems, and components, an earthquake producing two-times the acceleration values of the Design Earthquake was also considered (Reference 51).

2.5.3.9.3 Hosgri Earthquake

In 1976, subsequent to the issuance of the construction permit of Unit 1, PG&E was requested by the NRC to evaluate the plant's capability to withstand a postulated Richter Magnitude 7.5 earthquake centered along an offshore zone of geologic faulting, approximately 3 miles offshore, generally referred to as the "Hosgri fault." Details of the investigations associated with this fault are provided in Appendices 2.5D, 2.5E, and 2.5F of Reference 27 in Section 2.3. An overview is provided in Section 2.5.3.10.3. Note that the Shoreline Fault Zone (refer to Section 2.5.7.1) is considered to be a lesser included case under the Hosgri evaluation (Reference 55).

A further assessment of the seismic potential of faults mapped in the region of DCPP site was made following the extensive additional studies of on and offshore geology and is reported in Appendix 2.5D of Reference 27 of Section 2.3. This was done in terms of observed Holocene activity, to achieve assessment of what seismic activity is reasonably probable, in terms of observed late Pleistocene activity, fault dimensions, and style of deformation.

2.5.3.9.4 1991 Long Term Seismic Program Earthquake

PG&E performed a reevaluation of the seismic design bases of DCPP in response to License Condition No. 2.C.(7) of the Unit 1 Operating License. Details of this reevaluation, referred to as the Long Term Seismic Program, are provided in Section 2.5.7.

PG&E's evaluations included the development of significant additional data applicable to the geology, seismology, and tectonics of the DCPP region, including characterization of the Hosgri, Los Osos, San Luis Bay, Olson, San Simeon, and Wilmar Avenue faults. These faults were evaluated as potential seismic sources (Reference 40, Chapter 3). However, PG&E determined that the potential seismic sources of significance to the ground motions at the site are: the Hosgri and Los Osos fault zones, and the San Luis Bay fault, based on the probabilistic seismic hazard analysis; and the Hosgri fault zone, based on the deterministic analysis. Details are provided in Reference 40, Chapters 2 and 3, and summarized in SSER 34, Section 2.5.1, "Geology" and 2.5.2, "Seismology".

The NRC's review of PG&E's evaluations is documented in References 42 and 43.

2.5.3.10 Ground Accelerations and Response Spectra

The seismic design and evaluation of DCPP is based on the earthquakes described in the following four subsections. Refer to Section 3.7 for the design criteria associated with the application of the DE, DDE, and HE to the structures, systems, and components and the seismic margin assessment of the LTSP.

2.5.3.10.1 Design Earthquake

During the original design phase, the maximum ground acceleration that would occur at the DCPP site was estimated for each of the postulated earthquakes listed in Section 2.5.3.9, using the methods set forth in References 12 and 24. The plant site acceleration was primarily dependent on the following parameters: Gutenberg-Richter magnitude and released energy, distance from the earthquake focus to the plant site, shear and compressional velocities of the rock media, and density of the rock. Rock properties are discussed under Section 2.5.2.2.6, Site Engineering Properties.

The maximum rock accelerations that would occur at the DCPP site were estimated as:

Earthquake A . . . . 0.10 g	Earthquake C . . . . 0.05 g
Earthquake B . . . . 0.12 g	Earthquake D . . . . 0.20 g

In addition to the maximum acceleration, the frequency distribution of earthquake motions is important for comparison of the effects on plant structures and equipment. In general, the parameters affecting the frequency distribution are distance, properties of the transmitting media, length of faulting, focus depth, and total energy release. Earthquakes that might reach the site after traveling over great distances would tend to have their high frequency waves filtered out. Earthquakes that might be centered close to the site would tend to produce wave forms at the site having minor low frequency characteristics.

In order to evaluate the frequency distribution of earthquakes, the concept of the response spectrum is used.

For nearby earthquakes, the resulting response spectra accelerations would peak sharply at short periods and would decay rapidly at longer periods. Earthquake D would produce such response spectra. The March 1957 San Francisco earthquake as recorded in Golden Gate Park (S80°E component) was the same type. It produced a maximum recorded ground acceleration of 0.13 g (on rock) at a distance of about 8 miles from the epicenter. Since Earthquake D has an assigned hypocentral distance of 12 miles, it would be expected to produce response spectra similar in shape to those of the 1957 event.

Large earthquakes centered at some distance from the plant site would tend to produce response spectra accelerations that peak at longer periods than those for nearby smaller shocks. Such spectra maintain a higher spectral acceleration throughout the period range beyond the peak period. Earthquakes A and C are events that would tend to produce this type of spectra. The intensity of shaking as indicated by the maximum predicted ground acceleration shows that Earthquake C would always have lower spectral accelerations than Earthquake A.

Since the two shocks would have approximately the same shape spectra, Earthquake C would always have lower spectral accelerations than Earthquake A, and it is therefore eliminated from further consideration. The north-south component of the 1940 El Centro earthquake produced response spectra that emphasized the long period characteristics described above. Earthquake A, because of its distance from the plant site, would be expected to produce response spectra similar in shape to those produced by the El Centro event. Smoothed response spectra for Earthquake A were constructed by normalizing the El Centro spectra to 0.10 g. These spectra, however, show smaller accelerations than the corresponding spectra for Earthquake B (discussed in the next paragraph) for all building periods, and thus Earthquake A is also eliminated from further consideration.

Earthquake B would tend to produce response spectra that emphasize the intermediate period range inasmuch as the epicenter is not close enough to the plant site to produce large high frequency (short-period) effects, and it is too close to the site and too small in magnitude to produce large low frequency (long-period) effects. The N69°W component to the 1952 Taft earthquake produced response spectra having such characteristics. That shock was therefore used as a guide in establishing the shape of the response spectra that would be expected for Earthquake B.

Following several meetings with the AEC staff and their consultants, the following two modifications were made in order to make the criteria more conservative:

(1) The Earthquake D time-history was modified in order to obtain better continuity of frequency distribution between Earthquakes D and B.

(2) The accelerations of Earthquake B were increased by 25 percent in order to provide the required margin of safety to compensate for possible uncertainties in the basic earthquake data.

Accordingly, Earthquake D-modified was derived by modifying the S80°E component of the 1957 Golden Gate Park, San Francisco earthquake, and then normalizing to a maximum ground acceleration of 0.20 g. Smoothed response spectra for this earthquake are shown in Figure 2.5-21. Likewise, Earthquake B was derived by normalizing the N69°W component of the 1952 Taft earthquake to a maximum ground acceleration of 0.15 g. Smoothed response spectra for Earthquake B are shown in Figure 2.5-20. The maximum vibratory motion at the plant site would be produced by either Earthquake D-modified or Earthquake B, depending on the natural period of the vibrating body.

2.5.3.10.2 Double Design Earthquake

The maximum ground acceleration and response spectra for the Double Design Earthquake are twice those associated with the design earthquake, as described in Section 2.5.3.10.1 (Reference 51).

2.5.3.10.3 Hosgri Earthquake

As mentioned earlier, based on a review of the studies presented in Appendices 2.5D and 2.5E (of Reference 27 in Section 2.3) by the NRC and the United States Geologic Survey (USGS) (acting as the NRC's geological consultant), the NRC issued SSER 4 in May 1976. This supplement included the USGS conclusion that a magnitude 7.5 earthquake could occur on the Hosgri fault at a point nearest to the Diablo Canyon site. The USGS further concluded that such an earthquake should be described in terms of near fault horizontal ground motion using techniques and conditions presented in Geological Survey Circular 672. The USGS also recommended that an effective, rather than instrumental, acceleration be derived for seismic analysis.

The NRC adopted the USGS recommendation of the seismic potential of the Hosgri fault. In addition, based on the recommendation of Dr. N. M. Newmark, the NRC prescribed that an effective horizontal ground acceleration of 0.75g be used for the development of response spectra to be employed in a seismic evaluation of the plant. The NRC outlined procedures considered appropriate for the evaluation including an adjustment of the response spectra to account for the filtering effect of the large building foundations. An appropriate allowance for torsion and tilting was to be included in the analysis. A guideline for the consideration of inelastic behavior, with an associated ductility ratio, was also established.

The NRC issued SSER 5 in September 1976. This supplement included independently-derived response spectra and the rationale for their development. Parameters to be used in the foundation filtering calculation were delineated for each major structure. The supplement prescribed that either the spectra developed by Blume or Newmark would be acceptable for use in the evaluation with the following conditions:

(1) In the case of the Newmark spectra no reduction for nonlinear effects would be taken except in certain specific areas on an individual case basis.

(2) In the case of the Blume spectra a reduction for nonlinear behavior using a ductility ratio of up to 1.3 may be employed.

(3) The Blume spectra would be adjusted so as not to fall below the Newmark spectra at any frequency.

The development of the Blume ground response spectra, including the effect of foundation filtering, is briefly discussed below. The rationale and derivation of the Newmark ground response spectra is discussed in Appendix C to Supplement No. 5 of the SER.

The time-histories of strong motion for selected earthquakes recorded on rock close to the epicenters were normalized to a 0.75g peak acceleration. Such records provide the best available models for the Diablo Canyon conditions relative to the Hosgri fault zone. The eight earthquake records used are listed in the table below.

Earthquake	M	Depth, km	Recorded at	Epicentral Distance, km	Component	Peak Acceleration g
Helena 1935	6	5	Helena	3 to 8	EW	0.16
Helena 1935	6	5	Helena	3 to 8	NS	0.13
Daly City 1957	5.3	9	Golden Gate Park	8	N80W	0.13
Daly City 1957	5.3	9	Golden Gate Park	8	N10E	0.11
Parkfield 1966	5.6	7	Temblor 2	7	S25W	0.33
Parkfield 1966	5.6	7	Temblor 2	7	N65W	0.28
San Fernando 1971	6.6	13	Pacoima Dam	3	S14W	1.17
San Fernando 1971	6.6	13	Pacoima	3	N76W	1.08

The magnitudes are the greatest recorded thus far (September 1985) close in on rock stations and range from 5.3 to 6.6. Adjustments were made subsequently in the period range of the response spectrum above 0.40 sec for the greater long period energy expected in a 7.5M shock as compared to the model magnitudes.

The procedure followed was to develop 7 percent damped response spectra for each of the eight records normalized to 0.75g and then to treat the results statistically according to period bands to obtain the mean, the median, and the standard deviations of spectral response. At this stage, no adjustments for the size of the foundation or for ductility were made. The 7 percent damped response spectra were used as the basis for calculating spectra at other damping values.

Figures 2.5-29 and 2.5-30 show free-field horizontal ground response spectra as determined by Blume and Newmark, respectively, at damping levels from two to seven percent.

Figures 2.5-31 and 2.5-32 show vertical ground response spectra as determined by Blume and Newmark, respectively, for two to seven percent damping. The ordinates of vertical spectra are taken as two-thirds of the corresponding ordinates of the horizontal spectra. These response spectra, finalized in 1977, are described as the ”1977 Hosgri response spectra.” Note that the Shoreline Fault Zone (refer to Section 2.5.7.1) is considered to be a lesser included case under the Hosgri evaluation (Reference 55).

2.5.3.10.4 1991 Long Term Seismic Program Earthquake

As discussed in Section 2.5.3.9.4, the Long Term Seismic Program, in response to License Condition No. 2.C.(7) determined that the governing earthquake source for the deterministic seismic margins evaluation of DCPP (84th percentile ground motion response spectrum) is the Hosgri fault. Ground motions, and the corresponding free-field response spectra for a Richter Magnitude 7.2 earthquake centered along the Hosgri fault, approximately 4.5 km from DCPP, were developed by PG&E, as documented in Reference 40. This event is referred to as the “LTSP Earthquake.” As part of their review of Reference 40, the NRC concluded that spectra developed by PG&E could underestimate the ground motion (Reference 42). As a result, the final spectra, applicable to the LTSP evaluation of DCPP, is an envelope of that developed by PG&E and that developed by the NRC. Figures 2.5-33 and 2.5-34 show the 84th percentile ground motion response spectrum at 5% damping for the horizontal and vertical directions, respectively, described as the "1991 LTSP response spectra". These spectra define the current licensing basis for the LTSP.

Figure 2.5-35 shows a comparison of the horizontal 1991 LTSP response spectrum with the 1977 Newmark Hosgri spectrum (based on Reference 40, Figure 7-2). This comparison indicates that the 1977 Hosgri spectrum is greater than the 1991 LTSP spectrum at all frequencies less than about 15 Hz, but the 1991 LTSP spectrum exceeds the 1977 Hosgri spectrum by approximately 10 percent for frequencies above 15 Hz. This exceedance was accepted by the NRC in SSER 34 (Reference 42), Section 3.8.1.1 (Ground-Motion Input for Deterministic Evaluations):

> "On the basis of PG&E's margins evaluation discussed in Section 3.8.1.7 of this SSER, the staff concludes that these high-frequency spectral exceedances are not significant."

In addition, the NRC states in SSER 34 (Reference 42), Section 1.4 (Summary of Staff Conclusions):

> "The staff notes that the seismic qualification basis for Diablo Canyon will continue to be the original design basis plus the Hosgri evaluation basis, along with the associated analytical methods, initial conditions, etc. The LTSP has served as a useful check of the adequacy of the seismic margins and has generally confirmed that the margins are acceptable."

Therefore, the 1991 LTSP ground motion response spectra does not replace or modify, the DE, DDE, or 1977 Hosgri response spectra described above.

2.5.4 SURFACE FAULTING

2.5.4.1 Geologic Conditions of the Site

The geologic history and lithologic, stratigraphic, and structural conditions of the site and the surrounding area are described in Section 2.5.2 and are illustrated in the various figures included in Section 2.5.

2.5.4.2 Evidence for Fault Offset

Substantive geologic evidence, described under Section 2.5.2.2, Site Geology, indicates that the ground at and near the site has not been displaced by faulting for at least 80,000 to 120,000 years. It can be inferred, on the basis of regional geologic history, that minor faults in the site bedrock date from the mid-Pliocene or, at the latest, from mid-Pleistocene episodes of tectonic activity.

2.5.4.3 Identification of Active Faults

Three zones that include faults greater than 1000 feet in length were mapped within about 5 miles of the site. Two of these, the Edna and San Miguelito fault zones, were mapped on land in the San Luis Range. The third, consisting of several breaks associated with the offshore Santa Maria Basin East Boundary zone of folding and faulting, is described in Sections 2.5.2.1.2.3 and 2.5.2.1.5.5 under Regional Geologic and Tectonic Setting. The mapped trace of each of these structures is shown in Figures 2.5-3 and 2.5-4. Additional active faults that were identified through the studies associated with the Hosgri Evaluation and LTSP are discussed in Sections 2.5.3.9.3 and 2.5.3.9.4, respectively.

2.5.4.4 Earthquakes Associated With Active Faults

The earthquakes discussions are limited to those identified during the original design phase and do not include any earthquakes recorded since 1971.

The Edna fault or fault zone has been active at some time since the deposition of the Plio-Pleistocene Paso Robles Formation, which it displaces. It has no morphologic expression suggestive of late Pleistocene activity, nor is it known to displace late Pleistocene or younger deposits. Four epicenters of small (3.9 to 3M) shocks and 42 other epicenters for shocks of "small" or "unknown" intensity have been reported as occurring in the approximate vicinity of the Edna fault (Figures 2.5-3 and 2.5-4). Owing to the small size of the earthquakes that they represent, however, all of these epicenters are only approximately located. Further, they fall in the energy range of shocks that can be generated by fairly large construction blasts. At present, no conclusive evidence is available to determine whether the Edna fault could be classified as seismically active, or as geologically active in the sense of having undergone multiple movements within the last 500,000 years.

The San Miguelito fault has been mapped as not displacing the Plio-Pleistocene Paso Robles Formation. No instrumental epicenter has been reliably recorded from its vicinity, but the Berkeley Seismological Laboratory indicates Avila Bay as the presumed epicentral location for a moderately damaging (Intensity VII at Avila) earthquake that occurred on December 1, 1916. It seems likely, however, that this shock occurred along the offshore East Boundary zone rather than on the San Miguelito fault zone.

The East Boundary zone has an overall length of about 70 miles. Individual breaks within the zone are as much as 30 miles long, though the varying amount of displacement that occurs along specific breaks indicates that movement along them is not uniform, and it suggests that breakage may have occurred on separate, limited segments of the faults. The reach of the zone that is opposite DCPP site contains four fault breaks. These breaks range from 1 to 15 miles in length, and they have minimum distances of 2.1 to 4.5 miles from the site. The East Boundary zone is considered to be seismically active, since at least five instrumentally well located epicenters and as many as ten less reliably located other epicenters are centered along or near the zone. One of the breaks (located 3-1/2 miles offshore from the site) exhibits topographic expression that may represent a tectonic offset of the sea floor surface at a point along its trace 6 miles north of the site. Other faults in the East Boundary zone have associated erosion features, a few of which could possibly be partly of faultline origin.

The earthquake of December 1, 1916, though listed as having an epicentral location at Avila Bay, is considered more probably to have originated along either the East Boundary zone or, possibly, the Santa Lucia Bank fault. Effects of this shock at Avila included landsliding in Dairy Canyon, 2 miles north of town, and "...disturbance of waters in the Bay of San Luis Obispo." "...plaster in several cottages...was jarred loose...while some of the smokestacks on the (Union Oil Company) refinery were toppled over." It is apparently on this basis that the Berkeley listing of earthquakes assigns this shock a "large" intensity and places its approximate epicentral location at Port San Luis.

A small (Magnitude 2.9) shock that apparently originated near the East Boundary zone a short distance south of DCPP site was lightly felt at the site on September 24, 1974.

This shock, like most of those recorded along the East Boundary zone, was not damaging.

The minor fault zone that was mapped in the sea cliff at the mouth of Diablo Creek and in the excavation for the Unit 1 turbine building has an onshore length of about 550 feet, and it probably continues for some distance offshore. It has been definitely determined to be not active.

2.5.4.5 Correlation of Epicenters With Active Faults

Earthquake epicenters located within 50 miles of DCPP site, for earthquakes recorded through 1972, have been approximately located in the vicinity of each of the faults. The reported earthquakes are listed in Table 2.5-1 and as follows, and their indicated epicentral locations are shown in Figures 2.5-3 and 2.5-4:

Earthquake Epicenters Reported as Being Located Approximately in the Vicinities of San Luis Obispo, Avila, and Arroyo Grande

Date	Geographic N Latitude	Coordinates W Longitude	Magnitude	Intensity	Notes and Greenwich Mean Time (GMT)
7.10.1889	35.17°	120.58°			Arroyo Grande. Shocks for several days.
12.1.1916	35.17°	120.75°		VII	VII at Avila. Considerable glass broken and goods in stores thrown from shelves at San Luis Obispo. Water in bay disturbed, plaster in cottages jarred loose, smoke stacks of Union Oil refinery toppled over at Avila. Severe at Port San Luis. III at Santa Maria: 22:53:00
4.26.1950	35.20°	120.60°	3.5	V	V at Santa Maria. Also felt at Orcutt: 7:23:29
1.26.1971	35.20°	120.70°	3		Near San Luis Obispo: 21:53:53
1830 to 7.21.1931	35.25°	120.67°			42 epicenters

Earthquake Epicenters Reported as Being Located Approximately in the Vicinity of the Offshore Santa Maria Basin East Boundary Zone

Date	Geographic Coordinates N Latitude	W Longitude	Magnitude	Intensity	Notes and Greenwich Mean Time (GMT)
5.27.1935(30-1)	35.62°	121.64°	3	III	Felt at Templeton: 16:08:00
9.7.1939(30-6)	35.46°	121.50°	3		Off San Luis Obispo County; felt at Cambria: 2:50:30
1.27.1945	34.75°	120.67°	3.9		17:50:31
12.31.1948(30-10)	35.60°	121.23°	4.6		Felt along coast from Lompoc to Moss Landing. VI at San Simeon. V at Cayucos, Creston, Moss Landing, Piedras Blancas Light Station: 14:35:46
11.17.1949	34.80°	120.70°	2.8		IV at Santa Maria. Near Priest: 5:06:60
2.5.1955(30-23)	35.86°	121.15°	3.3		West of San Simeon: 7:10:19
6.21.1957(30-25A)	35.23°	120.95°	3.7		Off Coast. Felt in San Luis Obispo, Morro Bay: 20:46:42
8.18.1958	35.60°	121.30	3.4		Near San Simeon: 5:30:42
10.25.1967	35.73°	121.45°	2.6		Near San Simeon: 23:05:39.5

(Figures in parentheses refer to events relocated by S. W. Smith, refer to Table 2.5-2).

2.5.4.6 Description of Active Faults

Data pertaining to faults with lengths greater than 1000 feet and reaches within 50 miles of the site, as identified during the original design phase, are included in Section 2.5.2.1.5, Structure of the San Luis Range and Vicinity, and in Figures 2.5-3 and 2.5-4. These data indicate the fault lengths, relationship of the faults to regional tectonic structures, known history of displacements, outer limits, and whether the faults can be considered as active.

2.5.4.7 Results of Faulting Investigation

The site for Units 1 and 2 of DCPP was investigated in detail for faulting and other possibly detrimental geologic conditions. From studies made prior to design of the plant, it was determined that there was need to take into account the possibility of surface faulting in such design. The data on which this determination was based are presented in Section 2.5.2.2, Site Geology.

2.5.5 Stability of Subsurface Materials

The possibility of past or potential surface or subsurface ground subsidence, uplift, or collapse in the vicinity of DCPP was considered during the course of the geologic investigations for Units 1 and 2.

2.5.5.1 Geologic Features

The site is underlain by folded bedrock strata consisting predominantly of sandy mudstone and fine-grained sandstone. The existence of an unbroken and otherwise undeformed section of upper Pleistocene terrace deposits overlying a wave-cut bedrock bench at the site provides positive evidence that all folding and faulting in the bedrock antedated formation of the terrace. Local depressions and other irregularities on the bedrock surface plainly reflect erosion in an ancient surf zone.

The rocks that constitute the bedrock section are not subject to significant solution effects (i.e., development of cavities or channels that could affect the engineering or fluid conducting character of the rock) because the bedrock section does not contain thick or continuous bodies of soluble rock types such as limestone or gypsum. Voids encountered during excavation at the site were limited to thin zones of vuggy breccia and isolated vugs in some beds of calcareous mudstone. Areas where such minor vuggy conditions were present were noted at a few locations in the excavation for the Unit 2 containment and fuel handling structures (at plant grid coordinates N59, N597, E10, E005 and N59, N700, E10, E120).

The maximum size of any individual opening was 3 inches or less, and most were less than 1 inch in maximum dimension. Because of the limited extent and isolated nature of these small voids, they were not considered significant in foundation engineering or slope stability analyses.

It has been determined by field examination that no sea caves exist in the immediate vicinity of the site. The only cave like natural features in the area are shallow pits and hollows in some of the sea cliff outcrops of resistant tuff. These features generally have dimensions of a few inches to about 10 feet. They are superficial, and have originated through differential weathering of variably cemented rock.

Several exploratory wells have been drilled for petroleum within the San Luis Range, but no production was achieved and the wells were abandoned. The area is not now active in terms of either production or exploration. The location of the abandoned wells is shown in Figure 2.5-6, and the geologic relationships in the Range are illustrated in Section A-A' of Figure 2.5-6 and in Figure 2.5-7, Section D-D'. The nearest oil-producing area is the Arroyo Grande field, about 15 miles to the southeast.

The potential for future problems of ground instability at the site, because of nearby petroleum production, can be assessed in terms of the geologic potential for the occurrence of oil within, or offshore from, the San Luis Range. In addition, assessment can be made in terms of the geologic relationships in the site as contrasted with geologic conditions in places where oil field exploitation has resulted in deformation of the ground surface.

As shown in Figures 2.5-6 and 2.5-7, the San Luis Range has the structural form of a broad synclinal fold, which in turn is made up of several tightly compressed anticlines and synclines of lesser order. The configuration is not conducive to entrapment of hydrocarbon fluids, as such fluids tend to migrate upward through bedding and fracture-controlled zones of higher primary and secondary permeability until they reach a local trap or escape into the near surface or surface environment.

Within the San Luis Range, the only recognizable structural traps are in local zones where plunge reversals exist along the crests of the second-order anticlines. Such structures evidently were the actual or hoped-for targets for most of the exploratory wells that have been drilled in the San Luis Range, but none of these wells has produced enough oil or gas to record; thus, the traps have not been effective, or perhaps the strata are essentially lacking in hydrocarbon fluids. Other conditions that indicate poor petroleum prospects for the Range include the general absence of good reservoir rocks within the section and the relatively shallow basement of non petroliferous Franciscan rocks.

In the offshore, adjacent to the southerly flank of the San Luis Range, subsurface conditions are not well known, but are probably generally similar. Scattered data suggest that a structural high, perhaps defined by a west-northwest plunging anticline, may exist a few miles offshore from DCPP site. Such a feature could conceivably serve as a structural trap, if local closure were present along its axis; however, it seems unlikely that it would contain significant amounts of petroleum.

Available data pertaining to exploratory oil wells drilled in the region of the site are given here:

Exploratory Oil Wells in the Vicinity of DCPP Site

Data from exploratory wells drilled outside of oil and gas fields in California to December 31, 1963: Division of Oil and Gas, San Francisco.

Mount Diablo B. & M. T R Sec	Operator	Well No.	Elev, ft	Date Started	Total Depth, ft	Stratigraphy (depth in ft) Age at Bottom of Hole
31S 10E 3	Tidewater Oil Co.	"Montadoro" 1	365	April 1954	6,146	Monterey 0-3800; Obispo Tuff 3800: Franciscan; U. Jurassic
30S 10E 24	Gretna Corp.	"Maino-Gonzales" 1	275	March 1937	1,575	Franciscan; Jurassic
24	Wm. H. Provost	"Spooner" 1	325	July 1952	1,749	Jurassic
24	Shell Oil Co.	"Buchon"	-	-	-	-
34	A. O. Lewis	"Pecho" 1	177	May 1937	2,745	Monterey 0-2612; U. Miocene
30S 11E 9	Van Stone and Dallaston	"Souza" 1	42	Oct 1951	1,233	Franciscan; Jurassic
31S 11E 15	Tidewater Oil Co.	"Honolulu-Tidewater-U.S.L.-Heller “Lease" 1	1,614	Jan 1958	10,788	Monterey 0-4363; Pt. Sal 4363; Obispo Tuff 4722; Rincon Shale 5370; 2nd Tuff 5546; 2nd Rincon Shale 6354; 3rd Tuff 10,174; L. Miocene

For the purpose of assessing the potential for the occurrence of adverse oil field related ground deformation effects at DCPP site, in the unlikely event that petroleum should be discovered and produced at a nearby location, it is useful to review the nature and causes of such ground deformation, and the types of geologic conditions at places where it has been observed.

The general subject of surface deformation associated with oil and gas field operations has been reviewed by Yerkes and Castle (Reference 22), among others. Such deformation includes differential subsidence, development of horizontally compressive strain effects within the central parts of subsidence bowls and horizontally extensive strain effects around their margins, and development or activation of cracks and faults. Pull-apart cracks and normal faults may develop in the marginal zone of extensive strain, while reverse and thrust faults sometimes occur in the central, compressive part of subsidence bowls. These effects all can develop when extraction of petroleum, water, and sand, plus lowering of fluid pressures, result in compression within and adjacent to producing zones, and attendant subsidence of the overlying ground. Other effects, including rebound of the ground surface, fault activation, and earthquake generation, have resulted from injection of fluid into the ground for purposes of secondary recovery, subsidence control, and disposal of fluid waste.

In virtually all instances of ground-surface deformation associated with petroleum production, the producing field has been centered on an anticlinal structure, in general relatively broad and internally faulted. The strata in the producing and overlying parts of the section typically are poorly consolidated sandstone, siltstone, claystone, and shale of low structural competence. The field generally is one with relatively large production, with significant decline of fluid pressure in the producing zones.

The conditions just cited can be contrasted with those obtained in the vicinity of DCPP site, where the rocks lie along the flank of a major syncline. They consist of tight sandstone, tuffaceous sandstone, mudstone, and shale, together with large resistant masses of tuff and diabase. Bedding dips range from near horizontal to vertical and steeply overturned, as shown in Section D-D' of Figure 2.5-7 and Section A-B of Figure 2.5-10. This structural setting is unlike any reported from areas where oil-field-associated surface deformation has occurred.

The foregoing discussion leads to the following conclusions: (a) future development of a producing oil field in the vicinity of DCPP site is highly unlikely because of unfavorable geologic conditions, and (b) geologic conditions in the site vicinity are not conducive to the occurrence of surface deformation, even if nearby petroleum production could be achieved.

As was noted in Section 2.4, the rocks underlying the site do not constitute a significant groundwater reservoir, so that future development of deep rock water wells in the vicinity is not a reasonable possibility. The considerations pertaining to surface deformation resulting from water extraction are about the same as for petroleum extraction, so there is no likelihood that DCPP site could experience artificially induced and potentially damaging subsidence, uplift, collapse, or changes in subsurface effective stress related to pore pressure phenomena.

There are no mineral deposits of economic significance in the ground underlying the site.

Although some regional warping and uplift may well be taking place in the southern Coast Ranges, such deformation cannot be sufficiently rapid and local to impose significant effects on coastal installations. Apparent elevation of the San Luis Range has increased about 100 feet relative to sea level since the cutting of the main terrace bench at least 80,000 years ago.

Expressions of deformation preserved in the bedrock at the site include minor faults, folds, and zones of blocky fracturing in sandstone and intra-bed shearing in claystone. Zones of cemented breccia also are present, as is widespread evidence of disturbance adjacent to intrusive bodies of tuff. Local weakening of the rocks in some of these zones led to some problems during construction, but these were handled by conventional techniques such as overexcavation and rock bolting. No observed features of deformation are large or continuous enough to impose significant effects on the overall performance of the site foundation.

The foundation excavations for Units 1 and 2 were extended below the zone of intense near surface weathering so that the exposed bedrock was found to be relatively fresh and firm. The principal zones of structural weakness are associated with small bodies of altered tuff and with internally sheared beds of claystone. The claystone intra-bed shear was expressed by the development of numerous slickensided shear surfaces within parts of the beds, especially in places where the claystone had locally been squeezed into pod like masses. The shearing and local squeezing clearly are expressions of the preferential occurrence of differential adjustments in the relatively weaker claystone beds during folding of the section.

The claystone beds are localized in a part of the rock section that underlies the discharge structure and extends across the southerly part of the Unit 2 turbine-generator building, thence continuing easterly, along a strike through the ground south of the Unit 2 containment. The bedding dips 48 to 75° north within this zone. Individual claystone beds range from 1/2 inch to about 6 inches in thickness, and they occur as interbeds in the sandstone-mudstone rock section.

The relationship of the claystone layers to the foundation excavation is such that they crop out in several narrow bands across the floor and walls (refer to Figures 2.5-15 and 2.5-16). Thus, the claystone bed remains confined within the rock section, except in a narrow strip at the face of the excavation. Because of the small amount of claystone mass and the geometric relationship of the steeply dipping claystone interbeds to the foundation structures, it was determined that the finished structure would not be affected by any tendency of the claystone to undergo further changes in volume.

The only area in which claystone swelling was monitored was along the north wall of the lower part of the large slot cut for the cooling water discharge structure. There are several thin (6 inches or less) claystone interbeds in the sandstone-mudstone section. Because the orientation of the bedding and the plane of the cut face differ by only about 30°, and the bedding dips steeply into the face, opening of the cut served both to remove lateral support from the rock behind the face, and also to expose the clay beds to rainfall and runoff. This apparently resulted in both load relief and hydration swelling of the newly exposed claystone, which in turn caused some outward movement of the cut face. The movement then continued as gravity creep of the locally destabilized mass of rock between the claystone beds and the free face. The movement was finally controlled by installation of drilled-in lateral tie-backs, prior to placement of the reinforced concrete wall of the discharge structure.

No evidence of unrelieved residual stresses in the bedrock was noted during the excavation or subsequent construction of the plant foundation. Isolated occurrences of temporary slope instability clearly were related to locally weathered and fractured rock, hydration swelling of claystone interbeds, and local saturation by surface runoff. The Units 1 and 2 power plant facilities are founded on physically and chemically stable bedrock.

2.5.5.2 Properties of Underlying Materials

Static and dynamic engineering properties of materials in the subsurface at the site are presented in Section 2.5.2.2.6, Site Engineering Properties.

2.5.5.3 Plot Plan

Plan views of the site indicating exploratory boring and trenching locations are presented in Figures 2.5-8 and 2.5-11 through 2.5-15. Profiles illustrating the subsurface conditions relative to the PG&E Design Class I structures are furnished in Figures 2.5-12 through 2.5-16. Discussions of engineering properties of materials and groundwater conditions are included in Section 2.5.2.2.6, Site Engineering Properties.

2.5.5.4 Soil and Rock Characteristics

Information on compressional and shear wave velocity surveys performed at the site are included in Appendices 2.5A and 2.5B of Reference 27 of Section 2.3. Values of soil modulus of elasticity and Poisson's ratio calculated from seismic measurements are presented in Table 1 of Appendix 2.5A of Reference 27 of Section 2.3, and in Figure 2.5-19. Boring and trench logs are presented in Figures 2.5-23 through 2.5-28.

2.5.5.5 Excavations and Backfill

Plan and profile drawings of excavations and backfill at the site are presented in Figures 2.5-17 and 2.5-18. The engineered backfill placement operations are discussed in Section 2.5.2.2.6.4, Engineered Backfill.

2.5.5.6 Groundwater Conditions

Groundwater conditions at the site are discussed in Section 2.4.13. The effect on foundations of PG&E Design Class I structures is discussed in Section 2.5.2.2.6, Site Engineering Properties.

2.5.5.7 Response of Soil and Rock to Dynamic Loading

Details of dynamic testing on site materials are contained in Appendices 2.5A and 2.5B of Reference 27 in Section 2.3.

2.5.5.8 Liquefaction Potential

As stated in Section 2.5.2.2.6.5, adverse hydrologic effects on foundations of PG&E Design Class I structures can be neglected due to the structures being founded on bedrock and the groundwater level lying well below final grade.

There is a small local zone of medium dense sand located northeast of the intake structure and beneath a portion of buried ASW piping that is not attached to the circulating water tunnels. This zone is susceptible to liquefaction during design basis seismic events (References 45 and 46). The associated liquefaction-induced settlements from seismic events are considered in the design of the buried ASW piping. (References 48 and 49)

2.5.5.9 Earthquake Design Basis

The earthquake design bases for the DCPP site are discussed in Section 2.5.3.9, a discussion of the design response spectra is provided in Section 2.5.3.10, and the application of the earthquake ground motions to the seismic analysis of structures, systems, and components is provided in Section 3.7. Response acceleration curves for the site resulting from Earthquake B and Earthquake D-modified are shown in Figures 2.5-20 and 2.5-21, respectively. Response spectrum curves for the Hosgri earthquake are shown in Figures 2.5-29 through 2.5-32.

2.5.5.10 Static Analysis

A discussion of the analyses performed on materials at the site is presented in Section 2.5.2.2.6, Site Engineering Properties.

2.5.5.11 Criteria and Design Methods

The criteria and methods used in evaluating subsurface material stability are presented in Section 2.5.2.2.6, Site Engineering Properties.

2.5.5.12 Techniques to Improve Subsurface Conditions

Due to the bearing of in situ rock being well in excess of the foundation pressure, no treatment of the in situ rock is necessary. Compaction specifications for backfill are presented in Section 2.5.2.2.6.4, Engineered Backfill.

2.5.6 SLOPE STABILITY

2.5.6.1 Slope Characteristics

The only slope whose failure during a DDE could adversely affect the nuclear power plant is the slope east of the building complex (refer to Figures 2.5-17, 2.5-18, and 2.5-22). To evaluate the stability of this slope, the soil and rock conditions were investigated by exploratory borings, test pits, and a thorough geological reconnaissance by the soil consultant, Harding-Lawson Associates, and was in addition to the overall geologic investigation performed by other consultants.

The slope configuration and representative locations of the subsurface conditions determined from the exploration are shown on Plates 2, 3, and 4 of Appendix 2.5C of Reference 27 of Section 2.3. Reference 44 provides further information compiled in 1997 in response to NRC questions on landslide potential.

Bedrock is exposed along the lower portions of the cut slope up to about the lower bench at elevation 115 feet. It consists of tuffaceous siltstone and fine-grained sandstone of the Monterey Formation. Terrace gravel overlies bedrock and extends to an approximate elevation of 145 feet. Stiff clays and silty soils with gravel and rock fragments constitute the upper material on the site. The upper few feet of fine-grained soils are dark brown and expansive.

No free groundwater was observed in any of the borings which were drilled in April 1971, nor was any evidence of groundwater observed in this slope during the previous years of investigation and construction of the project.

In response to an NRC request in early 1997, PG&E conducted further investigations of slope stability at the site (Reference 44). The results of the investigations showed that earthquake loading, as a result of an earthquake on the Hosgri fault zone, following periods of prolonged precipitation will not produce any significant slope failure that can impact Design Class I structures and equipment. In addition, potential slope failures under such conditions will not adversely impact other important facilities, including the raw water reservoirs, the 230 kV and 500 kV switchyards, and the intake and discharge structures. Potential landslides may temporarily block the access road at several locations. However, there is considerable room adjacent to and north of the road to reroute emergency traffic.The investigation of the cut slope included geologic mapping of the soil and rock conditions exposed on the surface of slope and existing benches. Subsurface conditions were investigated by drilling test borings and by excavating test pits in the natural slope above the plant site (refer to Figure 2.5-22). The test borings were drilled with a truck mounted, 24 inch flight auger drill rig, and the test pits were excavated with a track-mounted backhoe. Boring and Log of Test Pits 1, 2, and 3 were logged by the soil consultant; borings 2 and 3 were logged by PG&E engineering personnel. The logs of all borings were verified by the soil consultant, who examined all samples obtained from each boring. Undisturbed samples were obtained from boring 2 and each of the test pits. Because of the stiffness of the soil, hardness of the rock, and type of drilling equipment used, the undisturbed samples were obtained by pushing an 18-inch steel tube that measured 2.5 inches in outside diameter. A Sprague & Henwood split-barrel sampler containing brass liners was used to obtain undisturbed soil samples from the test pits. The brass liners measured 2.5 inches in outside diameter and 6 inches in height. Logs of the borings and pits are shown in Figures 2.5-23 through 2.5-27. The soils were classified in accordance with the Unified Soil Classification System presented in Figure 2.5-28.

2.5.6.2 Design Criteria and Analyses

Undisturbed samples of the materials encountered in pits and borings were examined by the soil consultant in the laboratory and were subsequently tested to determine the shear strength, moisture content, and dry density. Strain controlled, unconsolidated, undrained triaxial tests at field moisture were performed on the clay to evaluate the shear strength of the materials penetrated. (The samples were maintained at field moisture since adverse moisture or seepage conditions were not encountered during this investigation nor previous investigations.) The confining stress was varied in relation to depth at which the undisturbed sample was taken. The test results are presented on the boring logs and are explained by the Key to Test Data, Figure 2.5-28.

The results of strength tests were correlated with the results developed during earlier investigations of DCPP site. Mohr circles of stresses at failure (6 to 7 percent strain) were drawn for each strength test result, and failure lines were developed through points representing one-half the deviator stresses. An average C-θ strength equal to a cohesion (C) value of 1000 psf and an angle of internal friction (θ) of 29° was selected for the slope stability analysis. The analysis was checked by maintaining the angle of internal friction (θ) constant at 19° and varying the cohesion (C) from 950 psf (weakest layer) to 3400 psf (deepest and strongest layer).

Because of the presence of large gravel sizes, it was not possible to accurately determine the strength of the sand and gravel lense. However, based on tests on sand samples from other parts of the site, an angle of internal friction of 35° was selected as being the minimum available. An assumed rock strength of 5000 psf was used. This value is consistent with strength tests performed on remold rock samples from other areas of the site.

The stability of the slope was analyzed for the forces of gravity using a static method that is, the conventional method of slices. This analysis was checked using Bishop's modified method. The static method of analysis was chosen because, for the soil conditions at the site, it was judged to be more conservative than a dynamic analysis.

Because the overall strength of the rock would preclude a stability failure except along a plane of weakness which was not encountered in the borings or during the many geologic mappings of the slope, only the stability of the soil over the rock was analyzed. The strength parameters were varied as previously discussed to determine the minimum factor of safety under the most critical strength condition. For the static analysis excluding horizontal forces, the factor of safety was computed to be 3. When the additional unbalanced horizontal force of 0.4 times the weight of the soil within the critical surface combined with a vertical force of 0.26 times the weight was included, the minimum computed factor of safety was 1.1.

On the basis of the investigation and analysis, it was concluded that the slope adjacent to DCPP site would not experience instability of sufficient magnitude to damage adjacent safety-related structures.

The above conclusion is substantiated by additional field exploration, laboratory tests, and dynamic analyses using finite element techniques. Refer to Appendix 2.5C of Reference 27 in Section 2.3, Harding-Lawson Associates' report on this work.

2.5.6.3 Slope Stability for Buried Auxiliary Saltwater System Piping

A portion of the buried ASW piping for Unit 1 ascends an approximate 2:1 (horizontal/vertical) slope to the parking area near the meteorology tower (Plates 1 and 2 of Reference 47). To ensure the stability of this slope in which the ASW piping is buried, a geotechnical evaluation, considering various design basis seismic events, was performed by Harding Lawson Associates. This evaluation is described in Reference 47. Based on this evaluation, it was concluded that this slope will be stable during seismic events and that additional loads resulting from permanent deformation of the slope will not impact the buried ASW piping.

2.5.7 LONG TERM SEISMIC PROGRAM

On November 2, 1984, the NRC issued the Diablo Canyon Unit 1 Facility Operating License DPR-80. In DPR-80, License Condition Item 2.C.(7), the NRC stated, in part:

> "PG&E shall develop and implement a program to reevaluate the seismic design bases used for the Diablo Canyon Power Plant."

PG&E's reevaluation effort in response to the license condition was titled the "Long Term Seismic Program" (LTSP). PG&E prepared and submitted to the NRC the "Final Report of the Diablo Canyon Long Term Seismic Program" in July 1988 (Reference 40). Between 1988 and 1991, the NRC performed an extensive review of the Final Report, and PG&E prepared and submitted written responses to formal NRC questions. In February 1991, PG&E issued the "Addendum to the 1988 Final Report of the Diablo Canyon Long Term Seismic Program" (Reference 41). In June 1991, the NRC issued Supplement Number 34 to the Diablo Canyon Safety Evaluation Report (SSER) (Reference 42) in which the NRC concluded that PG&E had satisfied License Condition 2.C.(7) of Facility Operating License DPR-80. In the SSER the NRC requested certain confirmatory analyses from PG&E, and PG&E subsequently submitted the requested analyses. The NRC's final acceptance of the LTSP is documented in a letter to PG&E dated April 17, 1992 (Reference 43).

The LTSP contains extensive data bases and analyses that update the basic geologic and seismic information in this section of the FSAR Update. However, the LTSP material does not address or alter the current design licensing basis for the plant. In SSER 34 (Reference 42), the NRC stated, "The Staff notes that the seismic qualification basis for Diablo Canyon will continue to be the original design basis plus the Hosgri Evaluation basis, along with associated analytical methods, initial conditions, etc."

As a condition of the NRC's close out of License Condition 2.C.(7), PG&E committed to several ongoing activities in support of the LTSP, as discussed in a public meeting between PG&E and the NRC on March 15, 1991 (Reference 53), described as the "Framework for the Future," in a letter to the NRC, dated April 17, 1991 (Reference 50), and affirmed by the NRC in SSER 34 (Reference 43). These ongoing activities include the following that are related to geology and seismology (Reference 42, Section 2.5.2.4):

(1) To continue to maintain a strong geosciences and engineering staff to keep abreast of new geological, seismic, and seismic engineering information and evaluate it with respect to its significance to Diablo Canyon.

(2) To continue to operate the strong-motion accelerometer array and the coastal seismic network.

A complete listing of bibliographic references to the LTSP reports and other documents may be found in References 40, 41 and 42.

2.5.7.1 Shoreline Fault Zone

In November 2008, as a result of the ongoing activities described in Section 2.5.7, the USGS, working in collaboration with the PG&E Geosciences Department, identified an alignment of microseismicity subparallel to the coastline adjacent to DCPP indicating the possible presence of a previously unidentified fault located approximately 1 km offshore of DCPP. The offshore region associated with this fault was subsequently named the Shoreline fault zone.

PG&E developed estimates of the 84th percentile deterministic ground motion response spectrum for earthquakes associated with the Shoreline fault zone. The results of the study of the Shoreline fault zone are documented in Reference 52. A map showing the location of the Shoreline Fault Zone is provided in Figure 2.5-36. This report includes a comparison of the updated 84th percentile deterministic response spectra with the 1991 LTSP and 1977 Hosgri earthquake response spectra. This comparison indicates that the updated deterministic response spectra are enveloped by both the 1977 Hosgri earthquake spectrum and the 1991 LTSP earthquake spectrum.

The NRC developed an independent assessment of the seismic source characteristics of the Shoreline fault and performed an independent deterministic seismic hazard assessment (References 54 and 55). The NRC concluded that their conservative estimates for the potential ground motions from the Shoreline fault are at or below the ground motions for which the DCPP has been evaluated previously and demonstrated to have a reasonable assurance of safety (i.e., the 1977 Hosgri earthquake and 1991 LTSP earthquake ground motion response spectra). The NRC stated that the "Shoreline scenario should be considered as a lesser included case under the Hosgri evaluation."

2.5.7.2 Evaluation of Updated Estimates of Ground Motion

As an outcome of the Shoreline fault zone evaluation described in Section 2.5.7.1, the process to be used for the evaluation of new/updated geological/seismological information has been developed (References 55 and 56). The new/updated geological/seismological information, resulting from the activities described in Section 2.5.7, will be evaluated using a process that is consistent with the evaluation process defined by the NRC in Reference 57.

2.5.8 Safety Evaluation

2.5.8.1 General Design Criterion 2, 1967 Performance Standards

The determination of the appropriate earthquake parameters for design of plant SSCs is addressed throughout Section 2.5, and the maximum earthquakes for the plant site are presented in Sections 2.5.3.9.1, 2.5.3.9.2, and 2.5.3.9.3. The associated design basis site free field accelerations and response spectra are presented in Sections 2.5.3.10.1, 2.5.3.10.2, and 2.5.3.10.3. The seismic design of these SSC is addressed in Section 3.7.

2.5.8.2 License Condition 2.C(7) of DCPP Facility Operating License DPR-80 Rev 44 (LTSP), Elements (1), (2) and (3)

PG&E’s reevaluation effort in response to the license condition was titled the "Long Term Seismic Program" (LTSP). PG&E prepared and submitted to the NRC the "Final Report of the Diablo Canyon Long Term Seismic Program" in July 1988. Between 1988 and 1991, the NRC performed an extensive review of the Final Report, and PG&E prepared and submitted written responses to formal NRC questions. In February 1991, PG&E issued the "Addendum to the 1988 Final Report of the Diablo Canyon Long Term Seismic Program". In June 1991, the NRC issued Supplement Number 34 to the Diablo Canyon Safety Evaluation Report (SSER) in which the NRC concluded that PG&E had satisfied License Condition 2.C(7) of Facility Operating License DPR-80. In the SSER the NRC requested certain confirmatory analyses from PG&E, and PG&E subsequently submitted the requested analyses. The NRC’s final acceptance of the LTSP is documented in a letter to PG&E dated April 17, 1992

The commitments made as a part of the Diablo Canyon Long Term Seismic Program are detailed in Section 2.5.3.9.4 and Section 2.5.7.

2.5.8.3 10 CFR Part 100, March 1966 - Reactor Site Criteria

As described in Sections 2.5.2 through 2.5.6 above, the physical characteristics of the site, including seismology and geology have been considered.

2.5.9 REFERENCES

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TABLE 2.1-1

HISTORICAL INFORMATION BELOW IS SHOWN IN ITALICS

POPULATION TRENDS OF THE STATE OF CALIFORNIA
AND OF SAN LUIS OBISPO AND SANTA BARBARA COUNTIES

Year	*State of California*	*San Luis Obispo County*	*Santa Barbara County*	*Notes*
1940	*6,907,387*	*33,246*	*70,555*	*(a)*
1950	*10,586,233*	*51,417*	*98,220*	*(a)*
1960	*15,717,204*	*81,044*	*168,962*	*(a)*
1970	*19,953,134*	*105,690*	*264,324*	*(a)*
1980	*23,668,562*	*155,345*	*298,660*	*(a)*
1990	*29,760,021*	*217,162*	*369,608*	*(a)*
2000	*33,871,648*	*246,681*	*399,347*	*(a)*
2010	*40,262,400*	*323,100*	*467,700*	*(b)*
2025	*48,626,052*	*426,812*	*603,966*	*(c)*

Notes: *(a) U.S. Bureau of the Census*
(b) State of California Department of Finance (June 2001)
(c) State of California Department of Finance Data Files (March 16, 2000)

TABLE 2.1-2

HISTORICAL INFORMATION BELOW IS SHOWN IN ITALICS

GROWTH OF PRINCIPAL COMMUNITIES WITHIN
50 MILES OF DCPP SITE

Community	*Population (1960 Census)*	*Population (1970 Census)*	*Population (1980 Census)*	*Population (1990 Census)*	*Population (2000 Census)*
Arroyo Grande	*3,291*	*7,454*	*10,350*	*14,378*	*15,851*
Atascadero	*5,983*	*10,290*	*15,930*	*23,138*	*26,411*
Grover City	*5,210*	*5,939*	*8,827*	*11,656*	*13,067*
Guadalupe	*2,614*	*3,145*	*3,629*	*5,479*	*5,659*
Lompoc	*14,415*	*25,284*	*26,267*	*37,649*	*41,103*
Morro Bay	*3,692*	*7,109*	*9,064*	*9,664*	*10,350*
Paso Robles	*6,617*	*7,168*	*9,163*	*18,583*	*24,297*
Pismo Beach	*1,762*	*4,043*	*5,364*	*7,669*	*8,551*
San Luis Obispo	*20,437*	*28,036*	*34,253*	*41,958*	*44,174*
Santa Maria	*20,027*	*32,749*	*39,685*	*61,284*	*77,423*

TABLE 2.1-3

HISTORICAL INFORMATION BELOW IS SHOWN IN ITALICS

POPULATION CENTERS OF 1,000 OR MORE
WITHIN 50 MILES OF DCPP SITE

Community	*County*	*Distance and Direction From the Site*	*Population (1970 Census)*	*Population (1980 Census)*	*Population (1990 Census)*	*Population (2000 Census)*
Baywood-Los Osos	*San Luis Obispo*	*8 miles N*	*3,487*	*10,933*	*15,290*	*14,351*
Morro Bay	*San Luis Obispo*	*10 miles N*	*7,109*	*9,064*	*12,949*	*10,350*
San Luis Obispo	*San Luis Obispo*	*12 miles ENE*	*28,036*	*34,253*	*51,173*	*44,174*
Pismo Beach	*San Luis Obispo*	*13 miles ESE*	*4,043*	*5,364*	*7,699*	*8,551*
Grover City	*San Luis Obispo*	*14 miles ESE*	*5,939*	*8,827*	*11,656*	*13,067*
Oceano	*San Luis Obispo*	*15 miles ESE*	*2,564*	*4,478*	*6,169*	*7,260*
Arroyo Grande	*San Luis Obispo*	*17 miles ESE*	*7,454*	*10,350*	*14,378*	*15,851*
Cayucos	*San Luis Obispo*	*17 miles N*	*1,772*	*2,301*	*2,960*	*2,943*
Atascadero	*San Luis Obispo*	*21 miles NNE*	*10,290*	*15,930*	*23,138*	*26,411*
Guadalupe	*Santa Barbara*	*23 miles SE*	*3,145*	*3,629*	*5,479*	*5,659*
Nipomo	*San Luis Obispo*	*24 miles ESE*	*3,642*	*5,247*	*7,109*	*12,626*
Cambria	*San Luis Obispo*	*28 miles NNW*	*1,716*	*3,061*	*5,382*	*6,232*
Santa Maria	*Santa Barbara*	*29 miles SE*	*39,878*	*39,685*	*61,284*	*77,423*
Paso Robles	*San Luis Obispo*	*30 miles NNE*	*7,168*	*9,163*	*18,583*	*24,297*
Orcutt	*Santa Barbara*	*33 miles SE*	*8,500*	*1,469*	*----*	*28,830*
Vandenberg	*Santa Barbara*	*35 miles SSE*	*13,193*	*13,975*	*----*	*11,953*
Lompoc	*Santa Barbara*	*45 miles SSE*	*25,284*	*26,267*	*37,649*	*41,103*
		Total	*180,793*	*203,996*	*280,898*	*351,081*

TABLE 2.1-4
HISTORICAL INFORMATION BELOW IS SHOWN IN ITALICS
TRANSIENT POPULATION AT RECREATION AREAS
WITHIN 50 MILES OF DCPP SITE

Names	*Visitor - Days*	*Name*	*Visitor - Days*
State Parks (a)		*Los Padres National Forest [c]*	
Cayucos State Beach	*698,000*	*Agua Escondido*	*700*
Hearst San Simeon State Historical Monument	*795,000*	*American Canyon*	*800*
Montana de Oro State Park	*683,000*	*Balm of Gilead*	*200*
Morro Bay State Park	*1,129,000*	*Brookshire Springs*	*1,600*
Morro Strand State Beach	*129,000*	*Buckeye*	*200*
Pismo State Beach	*1,297,000*	*Cerro Alto*	*15,600*
San Simeon State Beach	*696,000*	*French*	*200*
W. R. Hearst Memorial State Beach	*213,000*	*Frus*	*700*
		Hi Mountain	*4,800*
County and Local Parks (b)		*Horseshoe Springs*	*1,400*
		Indians	*600*
Atascadero Lake	*300,000*	*Kerry Canyon*	*300*
Avila Beach	*800,000*	*La Panza*	*4,400*
Cambria	*15,000*	*Lazy Camp*	*500*
Cayucos Beach	*918,000*	*Miranda Pine*	*2,300*
Cuesta	*67,000*	*Navajo*	*2,800*
Lake Nacimiento	*345,000*	*Pine Flat*	*300*
Lopez Recreation Area	*379,000*	*Pine Springs*	*400*
Los Alamos Park	*45,000*	*Plowshare Springs*	*300*
Miquelito Park	*36,000*	*Queen Bee*	*2,200*
Nipomo	*168,000*	*Stony Creek*	*1,100*
Ocean Park	*105,000*	*Sulphur Pot*	*1,000*
Oceano	*95,000*	*Upper Lopez*	*600*
Rancho Guadalupe Dunes Park	*48,000*	*Wagon Flat*	*2,200*
San Antonio Reservoir	*361,000*		
San Miguel	*54,000*		
Santa Margarita Lake	*169,000*		
Shamel	*130,000*		
Templeton	*99,000*		
Waller	*450,000*		

(a) California Department of Parks and Recreation (July 1998 through June 1999).
(b) County Park Departments.
Monterey County (July 1, 1998 through June 30, 1999).
San Luis Obispo and Santa Barbara Counties (July 1998 through June 1999).
(c) Los Padres National Forest (July 1, 1971 through June 30, 1972. Current data is no longer compiled.).

TABLE 2.1-5

HISTORICAL INFORMATION BELOW IS SHOWN IN ITALICS

1985 LAND USE CENSUS
DISTANCES IN MILES FROM THE UNIT 1 CENTERLINE
TO THE NEAREST MILK ANIMAL, RESIDENCE, VEGETABLE GARDEN

22-1/2 Degree[(a)] Radial Sector	*Nearest Milk Animal*	*Nearest Residence km (mi)*	*Residence Azimuth degree*	*Nearest Vegetable Garden*
NW	*None*	*5.95 (3.7)*	*326*	*None*
NNW	*None*	*2.50 (1.55)*	*333*	*None*
N	*None*	*7.15 (4.44)*	*008*	*None*
NNE	*None*	*5.30 (3.3)*	*018.5*	*None*
NE	*None*	*8.15 (5.06)*	*037*	*None*
ENE	*None*	*7.15 (4.44)*	*062.5*	*None*
E	*None*	*7.25 (4.5)*	*096.5*	*None*
ESE	*None*	*None*	*--*	*2*
SE	*None*	*None*	*--*	*None*

(a) *Sectors not shown contain no land beyond the site boundary, other than islets not used for the purposes indicated in this table.*

TABLE 2.3-1

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERSISTENCE OF CALM AT DIABLO CANYON EXPRESSED AS PERCENTAGE OF TOTAL HOURLY OBSERVATIONS FOR WHICH THE MEAN HOURLY WIND SPEED WAS LESS THAN 1 MILE PER HOUR FOR MORE THAN 1 TO 10 HOURS

	Station E	
Consecutive Hours	*25-foot level*	*250-foot level*
1	*5.9*	*4.9*
2	*3.8*	*3.1*
3	*2.5*	*2.0*
4	*1.8*	*1.2*
5	*1.0*	*0.7*
6	*0.7*	*0.4*
7	*0.5*	*0.3*
8	*0.3*	*0.2*
9	*0.2*	*0.2*
10	*0.1*	*0.1*

TABLE 2.3-2

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

NORMALIZED ANNUAL GROUND LEVEL CONCENTRATIONS DOWNWIND FROM DCPP SITE GROUND RELEASE

Ground Level Release 10-meter wind data and Temperature Gradient (76-10 meters). For calculations with wind speeds below 1.5 meters per second stability is based on Temperature Gradient only and either building wake or wind meander is considered - with wind speed above 1.5 meters per second stability is based on measured Sigma A and Temperature Gradient with building wake only considered. Data Period May 1973 through April 1975.

Midpoint of Directions from Plant for each 22.5 degree Sector

Dilution Factors $\chi/Q \times 10^{-8}$ sec m^{-3}

Downwind Distance (km)	*NW*	*NNW*	*N*	*NNE*	*NE*	*ENE*	*E*	*ESE*	*SE*
0.8	*387.15*	*220.81*	*95.726*	*57.503*	*61.687*	*49.292*	*89.447*	*355.48*	*978.67*
5.0	*24.738*	*12.860*	*5.6009*	*3.2347*	*3.8566*	*2.9593*	*5.0400*	*21.388*	*68.029*
10.0	*9.2115*	*4.6658*	*2.0693*	*1.1535*	*1.4426*	*1.0949*	*1.8138*	*7.6144*	*25.269*
15.0	*5.3897*	*2.6719*	*1.2018*	*0.65477*	*0.84233*	*0.63167*	*1.0391*	*4.3081*	*14.651*
30.0	*2.3889*	*1.1375*	*0.52497*	*0.27935*	*0.36768*	*0.27011*	*0.45145*	*1.8261*	*6.3086*
40.0	*1.7484*	*0.82010*	*0.38341*	*0.20223*	*0.26689*	*0.19464*	*0.33046*	*1.3223*	*4.5669*
50.0	*1.3803*	*0.64135*	*0.30252*	*0.15868*	*0.20947*	*0.15208*	*0.26155*	*1.0377*	*3.5778*
80.0	*0.84914*	*0.38822*	*0.18632*	*0.09654*	*0.12747*	*0.09173*	*0.16222*	*0.63113*	*2.1699*

TABLE 2.3-3

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

MONTHLY MIXING HEIGHTS[a] AT DCPP SITE

Month	*Morning*	*Hours of Day[b]*	*Afternoon*	*Hours of Day[b]*	*Evening*	*Hours of Day[b]*	*Night*	*Hours of Day[b]*
January	*500*	*9-11*	*600*	*12-16*	*700*	*17-19*	*500*	*20-8*
February	*600*	*9-11*	*600*	*12-17*	*800*	*18-20*	*600*	*21-8*
March	*700*	*8-10*	*800*	*11-17*	*1,000*	*18-20*	*800*	*21-7*
April	*600*	*7-10*	*700*	*11-18*	*800*	*19-21*	*700*	*22-6*
May	*500*	*7-11*	*600*	*12-20*	*700*	*21-23*	*600*	*24-6*
June	*500*	*7-10*	*500*	*11-20*	*600*	*21-23*	*500*	*24-6*
July	*500*	*7-9*	*500*	*10-20*	*700*	*21-23*	*500*	*24-6*
August	*500*	*7-9*	*600*	*10-20*	*700*	*21-23*	*600*	*24-6*
September	*500*	*8-10*	*600*	*11-19*	*800*	*20-22*	*600*	*23-7*
October	*500*	*8-10*	*600*	*11-19*	*800*	*20-22*	*500*	*23-7*
November	*500*	*8-10*	*600*	*11-17*	*700*	*18-20*	*500*	*21-7*
December	*500*	*9-11*	*600*	*12-17*	*700*	*18-20*	*500*	*21-8*

(a) Mixing heights (in meters) derived from seasonal estimates given by Holzworth[6]

(b) Definition of morning, afternoon, evening, and nighttime hours. Hours are inclusive in local time.

TABLE 2.3-4
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
ESTIMATES OF RELATIVE CONCENTRATIONS (χ/Q sec m^{-3}) AT SPECIFIED LOCATIONS DOWNWIND OF DCPP SITE[(a, b)]

Direction From Site	*Distance, mi*	*χ/Q (σ_r - ΔT)*
NW	*0.5*	*3.87×10^{-6}*
326	*3.6*	*1.71×10^{-7}*
NW	*5.0*	*1.25×10^{-7}*
NNW	*0.5*	*2.21×10^{-6}*
330	*1.75*	*4.28×10^{-7}*
NNW	*5.0*	*6.37×10^{-8}*
N	*0.5*	*9.57×10^{-7}*
N	*5.0*	*2.81×10^{-8}*
NNE	*0.5*	*5.75×10^{-7}*
NNE	*3.3*	*2.93×10^{-8}*
NNE	*5.0*	*1.58×10^{-8}*
NE	*0.5*	*6.17×10^{-7}*
035	*4.9*	*1.64×10^{-8}*
NE	*5.0*	*1.95×10^{-8}*
ENE	*0.7*	*2.83×10^{-7}*
ENE	*4.7*	*1.62×10^{-8}*
ENE	*5.0*	*1.49×10^{-8}*
E	*1.0*	*2.86×10^{-7}*
E	*3.8*	*3.70×10^{-8}*
E	*5.0*	*2.48×10^{-8}*
ESE	*1.0*	*1.21×10^{-6}*
ESE	*5.0*	*1.05×10^{-7}*
SE	*1.1*	*3.10×10^{-6}*
124	*2.0*	*9.42×10^{-7}*
SE	*5.0*	*3.43×10^{-7}*

(a) Based on the models described in Reference 21 and used for Table 2.3-2 (January 1978, Amendment 57) of the DCPP FSAR.

(b) Estimates Involve Wind Data From the 10 Meter Level and Temperature Gradient From the 76m - 10m Levels.

TABLE 2.3-6
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE PRECIPITATION DATA

Mean Monthly and Annual Precipitation for Indicated Period of Record
Precipitation in Inches -- Record in Years

STATIONS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Annual MEAN	Annual MAX	Annual MIN	Mean No. Days [a] Precipitation Greater Than 0.09 and 0.49	
Morro Bay	2.94	2.72	1.86	1.46	0.22	0.05	0.06	0.01	0.21	0.72	2.65	2.50	15.40	24.12	6.60	31	(10)
Years	14	14	14	14	14	14	13	13	13	12	13	13					
Pismo Beach	3.79	3.05	2.10	1.92	0.34	0.04	0.06	0.01	0.20	0.46	1.82	2.65	16.44	27.45	6.75	28	(11)
Years	11	11	11	11	11	11	12	12	12	12	12	12					
San Luis Obispo	4.72	4.12	3.34	1.60	0.51	0.11	0.01	0.02	0.20	0.82	1.72	3.94	21.11	48.76	6.93	30	(14)
Years	91	91	91	91	91	91	92	92	92	92	92	92					
Santa Maria	2.81	2.50	2.60	1.05	0.39	0.08	0.02	0.02	0.20	0.73	1.18	2.32	13.90	28.46	4.40	25	(7)
Years	69	69	69	68	68	68	68	69	69	69	69	69					
Santa Margarita	6.04	5.81	5.27	3.25	0.73	0.05	0.06	0.01	0.22	1.03	3.11	6.47	32.05	49.55	7.67	34	(21)
Years	20	20	20	20	21	21	21	21	20	21	21	21					
Camp San Luis	3.91	3.48	3.29	1.95	0.45	0.05	0.03	0.01	0.13	0.59	2.02	3.62	19.53	29.89	10.29	32	(13)
Years	18	18	18	18	18	18	17	18	18	18	18	19					

(a) Values shown in parentheses are mean number of days with precipitation amounts greater than 0.49.

TABLE 2.3-7

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE TEMPERATURE DATA

Coastal Stations Morro Bay and Pismo Beach. Values Shown in Parentheses are Pismo Beach. Period of Record: Morro Bay 14 years; Pismo Beach 12 years

Temperature in °F

Months	*Mean Temperature*		*Mean Maximum*		*Mean Minimum*		*Extreme Maximum*		*Extreme Minimum*		*Mean No. of Days Above 90°F*		*Mean No. of Days Below 32°F*	
January	*52.6*	*(51.7)*	*62.0*	*(61.3)*	*43.2*	*(42.0)*	*82*	*(80)*	*30*	*(24)*	*0*	*(0)*	*1*	*(2)*
February	*53.8*	*(53.7)*	*63.0*	*(64.0)*	*44.6*	*(43.4)*	*82*	*(82)*	*30*	*(29)*	*0*	*(0)*	*0*	*(1)*
March	*53.1*	*(54.8)*	*62.5*	*(65.5)*	*43.6*	*(44.0)*	*85*	*(88)*	*32*	*(30)*	*0*	*(0)*	*0*	*(1)*
April	*54.1*	*(56.1)*	*63.5*	*(66.1)*	*44.7*	*(46.1)*	*93*	*(90)*	*33*	*(32)*	*0*	*(0)*	*0*	*(0)*
May	*55.1*	*(57.3)*	*62.9*	*(67.5)*	*47.3*	*(47.1)*	*98*	*(89)*	*33*	*(36)*	*0*	*(1)*	*0*	*(0)*
June	*57.5*	*(59.8)*	*64.4*	*(69.8)*	*50.5*	*(49.7)*	*98*	*(96)*	*40*	*(40)*	*0*	*(0)*	*0*	*(0)*
July	*58.2*	*(60.5)*	*65.1*	*(68.7)*	*51.3*	*(52.3)*	*89*	*(104)*	*34*	*(38)*	*0*	*(0)*	*0*	*(0)*
August	*55.5*	*(60.6)*	*66.7*	*(68.5)*	*52.7*	*(52.7)*	*94*	*(102)*	*45*	*(43)*	*0*	*(0)*	*0*	*(0)*
September	*60.7*	*(62.1)*	*68.8*	*(71.8)*	*52.5*	*(52.3)*	*101*	*(99)*	*43*	*(41)*	*1*	*(1)*	*0*	*(0)*
October	*60.8*	*(60.6)*	*70.5*	*(71.3)*	*51.0*	*(49.8)*	*99*	*(95)*	*38*	*(32)*	*1*	*(1)*	*0*	*(0)*
November	*57.0*	*(58.3)*	*66.0*	*(69.4)*	*47.8*	*(47.1)*	*92*	*(91)*	*32*	*(29)*	*0*	*(0)*	*0*	*(0)*
December	*52.4*	*(54.6)*	*61.6*	*(65.3)*	*43.2*	*(43.9)*	*79*	*(92)*	*29*	*(28)*	*0*	*(0)*	*1*	*(1)*
Annual	*55.9*	*(57.5)*	*64.8*	*(67.4)*	*47.7*	*(47.5)*	*101*	*(104)*	*29*	*(24)*	*2*	*(3)*	*2*	*(5)*

Inland Stations San Luis Obispo and Santa Maria. Values Shown in Parenthesis are Santa Maria. Period of Record: San Luis Obispo 66 years; Santa Maria 17 years.

Months	*Mean Temperature*		*Mean Maximum*		*Mean Minimum*		*Extreme Maximum*		*Extreme Minimum*		*Mean No. of Days Above 90°F*		*Mean No. of Days Below 32°F*	
January	*51.8*	*(50.2)*	*62.1*	*(62.3)*	*41.5*	*(38.2)*	*84*	*(82)*	*20*	*(21)*	*0*	*(0)*	*1*	*(4)*
February	*53.6*	*(51.6)*	*63.5*	*(63.3)*	*43.5*	*(39.9)*	*89*	*(87)*	*25*	*(24)*	*0*	*(0)*	*1*	*(4)*
March	*54.9*	*(53.0)*	*65.2*	*(64.3)*	*44.8*	*(41.6)*	*93*	*(88)*	*28*	*(29)*	*0*	*(0)*	*0*	*(1)*
April	*56.7*	*(55.3)*	*67.6*	*(66.3)*	*46.0*	*(44.3)*	*97*	*(97)*	*30*	*(31)*	*0*	*(0)*	*0*	*(0)*
May	*58.6*	*(57.2)*	*69.3*	*(67.7)*	*47.8*	*(46.8)*	*100*	*(93)*	*34*	*(34)*	*0*	*(0)*	*0*	*(0)*
June	*62.0*	*(59.8)*	*73.6*	*(70.2)*	*50.2*	*(49.4)*	*110*	*(95)*	*37*	*(36)*	*1*	*(0)*	*0*	*(0)*
July	*64.6*	*(62.0)*	*76.9*	*(71.6)*	*52.0*	*(52.4)*	*106*	*(104)*	*42*	*(43)*	*2*	*(0)*	*0*	*(0)*
August	*64.7*	*(61.9)*	*77.0*	*(71.5)*	*52.4*	*(52.2)*	*107*	*(93)*	*40*	*(43)*	*1*	*(0)*	*0*	*(0)*
September	*64.9*	*(62.7)*	*77.8*	*(74.1)*	*52.0*	*(51.3)*	*110*	*(102)*	*38*	*(36)*	*4*	*(1)*	*0*	*(0)*
October	*62.5*	*(60.0)*	*75.3*	*(72.6)*	*49.8*	*(47.4)*	*103*	*(103)*	*35*	*(30)*	*2*	*(1)*	*0*	*(0)*
November	*58.3*	*(55.8)*	*70.7*	*(69.7)*	*45.9*	*(42.0)*	*96*	*(93)*	*24*	*(25)*	*0*	*(0)*	*0*	*(1)*
December	*53.5*	*(52.2)*	*64.4*	*(64.8)*	*42.8*	*(39.6)*	*92*	*(90)*	*24*	*(26)*	*0*	*(0)*	*0*	*(3)*
Annual	*58.8*	*(56.8)*	*70.3*	*(68.2)*	*47.4*	*(45.4)*	*110*	*(104)*	*20*	*(21)*	*10*	*(2)*	*2*	*(13)*

Air Weather Service - Directorate of Climatology
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Surface Winds

TABLE 2.3-8

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273 — *Station*
Santa Maria, California WBAS — *Station Name*
All — *Month*
Class
Jan 1948 - Jun 1958 — *Years*

Speed Dir.	1-3 Knots	4-10 Knots	11-21 Knots	22-27 Knots	28-40 Knots	41 Knots and Over	Total 4 Knots and Over	Total No. of Observations %	Total No. of Observations Obs	Sum of Speed	Mean Wind Speed, Knots
N	0.5	1.1	0.7				1.8	2.2	2095	17809	8.5
NNE	0.3	1.0	0.9	0.1			2.0	2.3	2160	21637	10.0
NE	0.8	1.2	0.5	0.1			1.8	2.6	2412	18236	7.6
ENE	0.5	1.2	0.1				1.3	1.8	1637	8937	5.5
E	1.2	5.2	0.2				5.5	6.7	6230	37649	6.0
ESE	0.8	2.9	0.3				3.3	4.1	3814	24253	6.4
SE	0.8	2.9	0.8	0.1			3.8	4.6	4295	33136	7.7
SSE	0.4	0.9	0.5				1.4	1.8	1644	13935	8.5
S	0.5	0.8	0.2				1.0	1.6	1455	9343	6.4
SSW	0.4	0.7	0.2				0.9	1.3	1205	7848	6.5
SW	0.9	2.0	0.3				2.4	3.3	3119	18690	6.0
WSW	0.9	3.3	0.9				4.2	5.1	4737	34900	7.4
W	1.6	9.4	4.4	0.1			13.8	15.5	14446	127257	8.8
WNW	1.2	9.8	5.4	0.1			15.3	16.5	15458	142383	9.2
NW	0.9	4.5	1.2				5.8	6.7	6221	46750	7.5
NNW	0.3	1.0	0.2				1.1	1.5	1375	9091	6.6
CALM								21.8	20397		
TOTALS	12.0	47.9	16.8	0.6	0.1		65.3	100.0	92700	571854	6.1

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Surface Winds

TABLE 2.3-9

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273 — *Station*

Santa Maria, California WBAS — *Station Name*

Jan — *Month*

Class

48 49 50 51 52 53 54 55 56 57 58

Years

Speed Dir.	1-3 Knots	4-10 Knots	11-21 Knots	22-27 Knots	28-40 Knots	41 Knots and Over	Total 4 Knots and Over	Total No. of Observations %	Total No. of Observations Obs.	Sum of Speed	Mean Wind Speed, Knots
N	0.4	1.9	1.4	0.1			3.3	3.7	300	2892	9.6
NNE	0.3	2.2	1.6	0.2			4.0	4.3	350	3671	10.5
NE	0.9	2.3	1.2	0.2			3.8	4.7	383	3408	8.9
ENE	0.7	2.3	0.1				2.4	3.1	254	1476	5.8
E	1.7	10.5	0.5				11.0	12.7	1042	6790	6.5
ESE	1.4	5.6	0.9				6.5	7.9	644	4443	6.9
SE	1.0	6.1	1.8	0.1			8.1	9.1	743	6108	8.2
SSE	0.4	1.3	1.0	0.1			2.4	2.8	229	2209	9.6
S	0.4	1.0	0.4				1.4	1.8	148	1070	7.2
SSW	0.3	0.7	0.4				1.1	1.4	115	964	8.4
SW	0.7	1.4	0.3				1.8	2.5	201	1308	6.5
WSW	0.5	1.6	0.4				1.9	2.4	196	1327	6.8
W	1.1	6.0	1.6				7.6	8.7	712	5493	7.7
WNW	0.7	6.7	1.8				8.5	9.3	757	6090	8.0
NW	0.6	4.0	0.7				4.7	5.4	439	3165	7.2
NNW	0.2	1.5	0.3				1.8	2.0	164	1207	7.4
CALM								18.3	1501		
TOTALS	11.4	54.9	14.5	0.8			70.2	100.0	8178	51621	6.3

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Surface Winds

TABLE 2.3-10

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273 — *Station*

Santa Maria, California WBAS — *Station Name*

Feb — *Month*

Class

48 49 50 51 52 53 54 55 56 57 58

Years

Speed Dir.	*1-3 Knots*	*4-10 Knots*	*11-21 Knots*	*22-27 Knots*	*28-40 Knots*	*41 Knots and Over*	*Total 4 Knots and Over*	*Total No. of Observations %*	*Total No. of Observations Obs.*	*Sum of Speed*	*Mean Wind Speed, Knots*
N	*0.6*	*2.1*	*1.6*	*0.1*			*3.8*	*4.4*	*325*	*3152*	*9.7*
NNE	*0.4*	*1.4*	*1.5*	*0.2*			*3.1*	*3.5*	*259*	*2822*	*10.9*
NE	*0.9*	*2.2*	*1.0*				*3.3*	*4.2*	*312*	*2458*	*7.9*
ENE	*0.7*	*2.4*					*2.5*	*3.2*	*240*	*1419*	*5.9*
E	*1.3*	*9.6*	*0.5*				*10.2*	*11.5*	*857*	*5626*	*6.6*
ESE	*1.2*	*4.6*	*0.4*	*0.1*			*5.1*	*6.3*	*472*	*3078*	*6.5*
SE	*1.0*	*4.5*	*1.5*	*0.1*			*6.0*	*7.0*	*524*	*4300*	*8.2*
SSE	*0.3*	*1.1*	*1.0*	*0.1*			*2.1*	*2.5*	*183*	*1758*	*9.6*
S	*0.5*	*1.0*	*0.5*				*1.6*	*2.0*	*152*	*1140*	*7.5*
SSW	*0.3*	*0.9*	*0.4*				*1.2*	*1.5*	*112*	*0841*	*7.5*
SW	*0.5*	*1.7*	*0.4*				*2.2*	*2.7*	*201*	*1393*	*6.9*
WSW	*0.4*	*2.5*	*0.6*				*3.1*	*3.5*	*260*	*1951*	*7.5*
W	*0.7*	*6.9*	*2.9*	*0.1*			*9.8*	*10.5*	*787*	*6984*	*8.9*
WNW	*0.7*	*9.8*					*10.5*	*11.3*	*841*	*7341*	*8.7*
NW	*0.8*	*4.5*	*1.0*				*5.5*	*6.3*	*470*	*3511*	*7.5*
NWW	*0.3*	*5.5*	*6.3*				*2.0*	*2.3*	*170*	*1332*	*7.8*
CALM								*17.4*	*1297*		
TOTALS	*10.6*	*54.4*	*17.2*	*0.5*			*72.0*	*100.0*	*7462*	*49106*	*6.6*

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Surface Winds

TABLE 2.3-11

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273 — *Station*
Santa Maria, California WBAS — *Station Name*
Mar — *Month*
Class

48 49 50 51 52 53 54 55 56 57 58
Years

Speed Dir.	1-3 Knots	4-10 Knots	11-21 Knots	22-27 Knots	28-40 Knots	41 Knots and Over	Total 4 Knots and Over	Total No. of Observations %	Total No. of Observations Obs.	Sum of Speed	Mean Wind Speed, Knots
N	0.5	1.5	0.9				2.4	2.9	239	2069	8.7
NNE	0.4	1.4	1.6	0.1			3.0	3.4	281	2894	10.3
NE	0.8	1.4	0.7	0.1			2.2	3.0	249	2015	8.1
ENE	0.5	1.4	0.0				1.4	1.9	153	807	5.3
E	1.0	6.2	0.2				6.4	7.4	605	3667	6.1
ESE	0.8	4.2	0.5				4.7	5.5	448	3059	6.8
SE	0.9	3.8	1.5	0.2	0.1		5.5	6.4	524	4696	9.0
SSE	0.6	1.2	0.9	0.1	0.1		2.4	3.0	242	2502	10.3
S	0.4	0.8	0.4				1.3	1.7	140	1188	8.5
SSW	0.4	0.8	0.3	0.1			1.2	1.6	129	1029	8.0
SW	0.8	1.9	0.6				2.5	3.3	266	1898	7.1
WSW	0.4	2.9	1.0				4.0	4.4	359	2917	8.1
W	1.1	6.1	4.8	0.2			11.2	12.2	999	10067	10.1
WNW	0.9	8.6	7.4	0.1			16.1	17.0	1391	14436	10.3
NW	0.8	5.3	1.8				7.1	7.9	645	5282	8.2
NNW	0.3	1.3	0.3				1.6	1.8	148	1078	7.3
CALM								16.7	1365		
TOTALS	10.4	48.7	22.9	1.0	0.3		73.0	100.0	8183	59504	7.3

Air Weather Service - Directorate of Climatology | Surface Winds
Data Control Division

TABLE 2.3-12

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273 — *Station*; *Santa Maria, California WBAS* — *Station Name*; *Apr* — *Month*; *Class*

48 49 50 51 52 53 54 55 56 57 58
Years

Speed Dir.	1-3 Knots	4-10 Knots	11-21 Knots	22-27 Knots	28-40 Knots	41 Knots and Over	Total 4 Knots and Over	Total No. of Observations %	Total No. of Observations Obs.	Sum of Speed	Mean Wind Speed, Knots
N	0.4	0.9	0.4				1.4	1.7	138	1061	7.7
NNE	0.2	0.7	0.7				1.5	1.7	133	1322	9.9
NE	0.9	0.9	0.2				1.1	2.0	156	822	5.3
ENE	0.4	0.4	0.0				0.4	0.9	68	282	4.1
E	1.1	3.3	0.1				3.4	4.5	356	1814	5.1
ESE	0.6	2.5	0.2				2.7	3.4	266	1564	5.9
SE	0.8	3.2	1.1	0.1			4.4	5.2	409	3269	8.0
SSE	0.5	1.2	0.6				1.9	2.4	188	1543	8.2
S	0.5	1.1	0.4				1.5	1.9	154	1118	7.3
SSW	0.5	0.8	0.3				1.1	1.6	123	870	7.1
SW	0.8	2.8	0.9				3.7	4.4	352	2651	7.5
WSW	0.7	3.2	1.3				4.5	5.1	408	3280	8.0
W	1.7	9.0	5.5	0.2			14.7	16.3	1294	12182	9.4
WNW	1.3	10.5	7.9	0.2			18.7	20.0	1583	15873	10.0
NW	1.0	5.1	1.3				6.4	7.4	587	4502	7.7
NNW	0.3	1.1	0.1				1.2	1.5	117	731	6.2
CALM								20.0	1588		
TOTALS	11.4	46.6	21.0	0.7			68.6	100.0	7920	52884	6.7

Air Weather Service - Directorate of Climatology
Data Control Division

Surface Winds

TABLE 2.3-13

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273	*Santa Maria, California WBAS*	*May*	
Station	*Station Name*	*Month*	*Class*

48 49 50 51 52 53 54 55 56 57 58
Years

Speed Dir.	*1-3 Knots*	*4-10 Knots*	*11-21 Knots*	*22-27 Knots*	*28-40 Knots*	*41 Knots and Over*	*Total 4 Knots and Over*	*Total No. of Observations %*	*Total No. of Observations Obs.*	*Sum of Speed*	*Mean Wind Speed, Knots*
N	*0.4*	*0.5*	*0.3*				*0.8*	*1.2*	*102*	*763*	*7.5*
NNE	*0.2*	*0.5*	*0.2*				*0.7*	*0.9*	*75*	*509*	*6.8*
NE	*0.5*	*0.7*	*0.2*				*0.9*	*1.3*	*107*	*682*	*6.4*
ENE	*0.4*	*0.6*					*0.7*	*1.1*	*87*	*421*	*4.8*
E	*0.7*	*2.2*					*2.2*	*3.0*	*244*	*1298*	*5.3*
ESE	*0.7*	*1.4*	*0.1*				*1.4*	*2.1*	*173*	*898*	*5.2*
SE	*0.7*	*1.6*	*0.1*				*1.7*	*2.5*	*201*	*1128*	*5.6*
SSE	*0.3*	*0.6*	*0.1*				*0.7*	*1.0*	*83*	*508*	*6.1*
S	*0.7*	*0.9*	*0.2*				*1.1*	*1.8*	*146*	*850*	*5.8*
SSW	*0.5*	*1.1*	*0.2*				*1.2*	*1.7*	*139*	*820*	*5.9*
SW	*1.0*	*2.8*	*0.4*				*3.3*	*4.3*	*352*	*2071*	*5.9*
WSW	*1.1*	*4.4*	*2.0*				*6.5*	*7.5*	*615*	*5056*	*8.2*
W	*1.6*	*10.7*	*7.7*	*0.3*			*18.7*	*20.3*	*1664*	*16546*	*9.9*
WNW	*1.3*	*11.7*	*7.9*	*0.2*			*19.9*	*21.1*	*1730*	*16949*	*9.8*
NW	*1.0*	*4.3*	*1.8*				*6.1*	*7.1*	*581*	*4684*	*8.1*
NNW	*0.4*	*0.7*	*0.1*				*0.8*	*1.2*	*95*	*511*	*5.4*
CALM								*21.9*	*1789*		
TOTALS	*11.5*	*44.6*	*21.5*	*0.6*			*66.7*	*100.0*	*8183*	*53694*	*6.6*

Air Weather Service - Directorate of Climatology
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Surface Winds

TABLE 2.3-14

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273 — *Station*

Santa Maria, California WBAS — *Station Name*

June — *Month*

Class

48 49 50 51 52 53 54 55 56 57 58

Years

Speed Dir.	1-3 Knots	4-10 Knots	11-21 Knots	22-27 Knots	28-40 Knots	41 Knots and Over	Total 4 Knots and Over	Total No. of Observations %	Total No. of Observations Obs.	Sum of Speed	Mean Wind Speed, Knots
N	0.4	0.5	0.1				0.5	0.9	73	361	4.9
NNE	0.2	0.2	0.2				0.4	0.6	49	378	7.7
NE	0.5	0.4	0.1				0.5	1.0	83	455	5.5
ENE	0.3	0.2					0.2	0.5	43	160	3.7
E	1.0	1.3					1.4	2.3	185	780	4.2
ESE	0.4	0.6					0.6	1.0	78	326	4.2
SE	0.6	1.1					1.1	1.7	133	610	4.6
SSE	0.3	0.4					0.4	0.6	51	241	4.7
S	0.5	0.6					0.7	1.1	89	414	4.7
SSW	0.4	0.7	0.1				0.9	1.2	97	596	6.1
SW	1.4	2.7	0.4				3.1	4.5	357	2029	5.7
WSW	0.9	3.9	1.8				5.8	6.7	528	4395	8.3
W	2.1	12.3	8.0	0.1			20.4	22.5	1782	16856	9.5
WNW	1.7	13.5	10.0	0.2			23.6	25.3	2004	19743	9.9
NW	0.9	4.9	1.8				6.7	7.6	605	4861	8.0
NNW	0.3	0.3	0.1				0.4	0.7	52	290	5.6
CALM								21.6	1710		
TOTALS	11.8	43.9	22.6	0.3			66.6	100.0	7919	52495	6.6

Air Weather Service - Directorate of Climatology
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Surface Winds

TABLE 2.3-15

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273 — *Station*
Santa Maria, California WBAS — *Station Name*
July — *Month*
Class

48 49 50 51 52 53 54 55 56 57 58
Years

Speed Dir.	1-3 Knots	4-10 Knots	11-21 Knots	22-27 Knots	28-40 Knots	41 Knots and Over	Total 4 Knots and Over	Total No. of Observations %	Total No. of Observations Obs.	Sum of Speed	Mean Wind Speed, Knots
N	0.5	0.6					0.6	1.1	89	352	4.0
NNE	0.3	0.4					0.4	0.7	58	246	4.2
NE	0.4	0.5					0.5	0.9	74	277	3.7
ENE	0.3	0.2					0.2	0.5	40	146	3.7
E	0.4	0.7					0.7	1.2	96	403	4.2
ESE	0.3	0.3					0.3	0.6	52	196	3.8
SE	0.3	0.7					0.7	1.0	84	370	4.4
SSE	0.1	0.3					0.3	0.5	38	175	4.6
S	0.5	0.5					0.5	1.0	83	314	3.8
SSW	0.5	0.5					0.5	1.0	82	334	4.1
SW	1.3	2.1	0.1				2.2	3.5	285	1410	4.9
WSW	1.6	4.3	0.6				4.9	6.5	533	3422	6.4
W	2.7	14.5	5.6				20.1	22.8	1863	15557	8.4
WNW	2.2	14.4	6.7				21.1	23.3	1906	16377	8.6
NW	1.2	5.2	1.5				6.7	7.9	646	4697	7.3
NNW	0.4	0.5					0.5	0.9	76	313	4.1
CALM								26.6	2177		
TOTALS	12.9	45.8	14.6				60.4	100.0	8182	44589	5.4

Air Weather Service - Directorate of Climatology
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Surface Winds

TABLE 2.3-16

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273 — *Station*
Santa Maria, California WBAS — *Station Name*
Aug — *Month*
Class

48 49 50 51 52 53 54 55 56 57
Years

Speed Dir.	1-3 Knots	4-10 Knots	11-21 Knots	22-27 Knots	28-40 Knots	41 Knots and Over	Total 4 Knots and Over	Total No. of Observations %	Total No. of Observations Obs.	Sum of Speed	Mean Wind Speed, Knots
N	0.5	0.6					0.6	1.1	79	311	3.9
NNE	0.2	0.2					0.2	0.5	36	140	3.9
NE	0.5	0.4					0.4	0.9	64	228	3.6
ENE	0.3	0.5					0.5	0.7	55	235	4.3
E	0.4	0.7					0.7	1.1	83	354	4.3
ESE	0.3	0.6					0.6	0.9	69	287	4.2
SE	0.5	1.2					1.2	1.7	128	578	4.5
SSE	0.4	0.5					0.5	0.9	68	286	4.2
S	0.7	0.6					0.6	1.2	91	348	3.8
SSW	0.4	0.5					0.5	0.9	67	274	4.1
SW	1.5	3.2	0.1				3.3	4.8	356	1755	4.9
WSW	1.5	5.2	1.1				6.3	7.8	579	4012	6.9
W	2.1	14.9	5.5				20.4	22.5	1676	14120	8.4
WNW	1.1	12.5	4.8				17.3	18.4	1369	11893	8.7
NW	1.1	4.7	1.2				5.9	7.0	522	3765	7.2
NNW	0.4	0.4					0.4	0.8	60	251	4.2
CALM								28.7	2132		
TOTALS	11.8	46.7	12.8				59.5	100.0	7434	38837	5.2

Air Weather Service - Directorate of Climatology
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Surface Winds

TABLE 2.3-17

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273 — *Station*; *Santa Maria, California WBAS* — *Station Name*; *Sept* — *Month*; Class

48 49 50 51 52 53 54 55 56 57
Years

Speed Dir.	*1-3 Knots*	*4-10 Knots*	*11-21 Knots*	*22-27 Knots*	*28-40 Knots*	*41 Knots and Over*	*Total 4 Knots and Over*	*Total No. of Observations %*	*Obs.*	*Sum of Speed*	*Mean Wind Speed, Knots*
N	*0.6*	*0.5*	*0.1*				*0.7*	*1.2*	*89*	*474*	*5.3*
NNE	*0.5*	*0.5*	*0.1*				*0.7*	*1.2*	*85*	*461*	*5.4*
NE	*0.7*	*0.8*	*0.1*				*0.9*	*1.6*	*118*	*574*	*4.9*
ENE	*0.3*	*0.7*					*0.8*	*1.1*	*77*	*379*	*4.9*
E	*0.9*	*2.3*					*2.4*	*3.3*	*239*	*1191*	*5.0*
ESE	*0.7*	*1.4*					*1.4*	*2.1*	*154*	*716*	*4.6*
SE	*1.0*	*1.6*					*1.7*	*2.6*	*189*	*874*	*4.6*
SSE	*0.3*	*0.6*					*0.6*	*1.0*	*69*	*320*	*4.6*
S	*0.6*	*0.8*					*0.8*	*1.4*	*101*	*436*	*4.3*
SSW	*0.5*	*0.5*					*0.5*	*1.0*	*71*	*309*	*4.4*
SW	*1.2*	*2.1*	*0.1*				*2.2*	*3.4*	*244*	*1240*	*5.1*
WSW	*1.3*	*4.4*	*0.9*				*5.3*	*6.6*	*473*	*3287*	*6.9*
W	*2.1*	*12.7*	*4.6*	*0.1*			*17.3*	*19.4*	*1394*	*11723*	*8.4*
WNW	*1.3*	*10.3*	*5.0*	*0.1*			*15.4*	*16.7*	*1202*	*10714*	*8.9*
NW	*1.1*	*4.3*	*0.9*				*5.2*	*6.3*	*452*	*3068*	*6.8*
NNW	*0.5*	*0.5*					*0.5*	*1.0*	*74*	*342*	*4.6*
CALM								*30.1*	*2166*		
TOTALS	*13.6*	*44.0*	*12.1*	*0.2*			*56.3*	*100.0*	*7197*	*36108*	*5.0*

Air Weather Service - Directorate of Climatology
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Surface Winds

TABLE 2.3-18

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273 — *Station*
Santa Maria, California WBAS — *Station Name*
Oct — *Month*
Class
48 49 50 51 52 53 54 55 56 57 — *Years*

Speed Dir.	*1-3 Knots*	*4-10 Knots*	*11-21 Knots*	*22-27 Knots*	*28-40 Knots*	*41 Knots and Over*	*Total 4 Knots and Over*	*Total No. of Observations %*	*Total No. of Observations Obs.*	*Sum of Speed*	*Mean Wind Speed, Knots*
N	*0.4*	*0.7*	*0.5*				*1.2*	*1.6*	*119*	*1046*	*8.8*
NNE	*0.4*	*0.9*	*0.6*	*0.1*			*1.7*	*2.1*	*157*	*1466*	*9.3*
NE	*1.1*	*1.4*	*0.7*	*0.1*			*2.2*	*3.3*	*247*	*1775*	*7.2*
ENE	*0.5*	*1.3*					*1.3*	*1.8*	*132*	*736*	*5.6*
E	*1.7*	*6.0*	*0.2*				*6.3*	*8.0*	*592*	*3348*	*5.7*
ESE	*1.2*	*3.4*	*0.1*				*3.5*	*4.7*	*347*	*1921*	*5.5*
SE	*0.9*	*3.3*	*0.3*				*3.6*	*4.5*	*335*	*2030*	*6.1*
SSE	*0.4*	*0.8*	*0.2*				*1.0*	*1.4*	*102*	*679*	*6.7*
S	*0.5*	*0.7*	*0.3*				*1.0*	*1.5*	*112*	*726*	*6.5*
SSW	*0.5*	*0.5*	*0.2*				*0.7*	*1.1*	*84*	*496*	*5.9*
SW	*1.1*	*1.8*	*0.2*				*1.9*	*3.0*	*223*	*1173*	*5.3*
WSW	*1.0*	*3.2*	*0.7*				*3.9*	*4.9*	*363*	*2503*	*6.9*
W	*2.1*	*9.1*	*3.7*				*12.8*	*15.0*	*1112*	*9125*	*8.2*
WNW	*1.3*	*8.6*	*4.8*	*0.2*			*13.6*	*14.9*	*1109*	*10269*	*9.3*
NW	*0.9*	*4.2*	*1.0*				*5.2*	*6.1*	*454*	*3204*	*7.1*
NNW	*0.3*	*0.6*	*0.1*				*0.7*	*1.1*	*79*	*470*	*5.9*
CALM								*25.0*	*1856*		
TOTALS	*14.3*	*46.5*	*13.7*	*0.5*	*0.0*		*60.7*	*100.0*	*7423*	*40967*	*5.5*

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Surface Winds

TABLE 2.3-19

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273 — *Station*; *Santa Maria, California WBAS* — *Station Name*; *Nov* — *Month*; *Class*

48 49 50 51 52 53 54 55 56 57 — *Years*

Speed Dir.	*1-3 Knots*	*4-10 Knots*	*11-21 Knots*	*22-27 Knots*	*28-40 Knots*	*41 Knots and Over*	*Total 4 Knots and Over*	*Total No. of Observations %*	*Total No. of Observations Obs.*	*Sum of Speed*	*Mean Wind Speed, Knots*
N	*0.5*	*1.4*	*1.2*	*0.1*			*2.6*	*3.1*	*224*	*2109*	*9.4*
NNE	*0.5*	*1.7*	*1.7*	*0.3*			*3.7*	*4.2*	*302*	*3374*	*11.2*
NE	*0.8*	*1.8*	*0.8*	*0.4*	*0.2*		*3.2*	*4.0*	*288*	*2840*	*9.9*
ENE	*0.7*	*2.1*					*2.1*	*2.8*	*204*	*1125*	*5.5*
E	*2.1*	*10.2*	*0.6*				*10.8*	*13.0*	*933*	*6008*	*6.4*
ESE	*1.1*	*5.3*	*0.8*				*6.1*	*7.2*	*516*	*3491*	*6.8*
SE	*1.0*	*3.9*	*0.9*	*0.2*			*5.0*	*6.0*	*433*	*3400*	*7.9*
SSE	*0.5*	*1.0*	*0.5*				*1.5*	*2.1*	*148*	*1190*	*8.0*
S	*0.5*	*0.9*	*0.3*				*1.2*	*1.7*	*120*	*795*	*6.6*
SSW	*0.4*	*0.5*	*0.3*				*0.9*	*1.3*	*96*	*733*	*.6*
SW	*0.6*	*1.0*	*0.3*				*1.3*	*2.0*	*141*	*947*	*.7*
WSW	*0.6*	*2.1*	*0.3*				*2.4*	*3.0*	*219*	*1418*	*6.5*
W	*1.4*	*6.6*	*1.4*				*8.1*	*9.4*	*678*	*5104*	*7.5*
WNW	*1.1*	*7.6*	*3.3*				*10.9*	*12.1*	*868*	*7440*	*8.6*
NW	*0.7*	*3.9*	*0.7*				*4.6*	*5.3*	*379*	*2732*	*7.2*
NNW	*0.3*	*1.4*	*0.4*				*1.8*	*2.1*	*148*	*1127*	*7.6*
CALM								*20.7*	*1490*		
TOTALS	*13.0*	*51.4*	*13.7*	*0.9*	*0.2*		*66.3*	*100.0*	*7187*	*43833*	*6.1*

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Surface Winds

TABLE 2.3-20

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PERCENTAGE FREQUENCY OF OCCURRENCE
DIRECTIONS BY SPEED GROUPS

23273	*Santa Maria, California WBAS*	*Dec*	
Station	*Station Name*	*Month*	*Class*

48 49 50 51 52 53 54 55 56 57
Years

Speed Dir.	*1-3 Knots*	*4-10 Knots*	*11-21 Knots*	*22-27 Knots*	*28-40 Knots*	*41 Knots and Over*	*Total 4 Knots and Over*	*Total No. of Observations %*	*Total No. of Observations Obs.*	*Sum of Speed*	*Mean Wind Speed, Knots*
N	*0.5*	*2.0*	*1.6*	*0.1*			*3.8*	*4.3*	*318*	*3219*	*10.1*
NNE	*0.4*	*1.9*	*2.4*	*0.3*	*0.1*		*4.6*	*5.0*	*375*	*4354*	*11.6*
NE	*1.1*	*2.1*	*1.1*	*0.2*			*3.3*	*4.5*	*331*	*2702*	*8.2*
ENE	*1.0*	*2.6*	*0.2*				*2.8*	*3.8*	*284*	*1751*	*6.2*
E	*2.4*	*10.5*	*0.6*				*11.1*	*13.4*	*998*	*6370*	*6.4*
ESE	*1.3*	*5.6*	*1.1*				*6.7*	*8.0*	*595*	*4274*	*7.2*
SE	*0.9*	*4.5*	*2.1*	*0.3*	*0.1*		*7.1*	*8.0*	*592*	*5773*	*9.8*
SSE	*0.5*	*1.4*	*1.1*	*0.2*	*0.1*		*2.8*	*3.3*	*243*	*2524*	*10.4*
S	*0.5*	*0.8*	*0.3*				*1.1*	*1.6*	*119*	*944*	*7.9*
SSW	*0.4*	*0.6*	*0.2*				*0.8*	*1.2*	*90*	*582*	*6.5*
SW	*0.6*	*1.0*	*0.2*				*1.3*	*1.9*	*141*	*815*	*5.8*
WSW	*0.7*	*1.7*	*0.4*				*2.1*	*2.7*	*204*	*1332*	*6.5*
W	*1.1*	*4.4*	*1.0*				*5.4*	*6.5*	*485*	*3500*	*7.2*
WNW	*1.0*	*6.8*	*1.6*				*8.4*	*9.4*	*698*	*5358*	*7.7*
NW	*0.7*	*4.4*	*0.8*				*5.2*	*5.9*	*441*	*3279*	*7.4*
NNW	*0.4*	*1.8*	*0.4*				*2.2*	*2.6*	*192*	*1439*	*7.5*
CALM								*17.8*	*1326*		
TOTALS	*13.5*	*52.2*	*15.0*	*1.2*	*0.3*		*68.7*	*100.0*	*7432*	*48216*	*6.5*

TABLE 2.3-21
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
EXTREMELY UNSTABLE (ΔT less than -1.9°C/100M)
DIABLO CANYON PERIOD OF RECORD JULY 1967-OCTOBER 1969

FREQUENCY TABLE

			Wind Speed, mph						
Direction	*Calm*	*2.0*	*5.1*	*9.6*	*15.1*	*21.1*	*39.6*	*Row Sums*	*Row Avg*
CALM	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*3*	*0.0*
22.50	*0*	*1*	*7*	*6*	*0*	*0*	*0*	*14*	*7.4*
45.00	*0*	*0*	*1*	*3*	*1*	*0*	*0*	*5*	*9.6*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*1*	*2*	*0*	*0*	*0*	*0*	*3*	*3.7*
112.50	*0*	*0*	*1*	*3*	*11*	*12*	*9*	*36*	*19.9*
135.00	*0*	*2*	*3*	*12*	*24*	*12*	*14*	*67*	*17.6*
157.50	*0*	*2*	*5*	*7*	*6*	*10*	*4*	*34*	*15.7*
180.00	*0*	*3*	*5*	*5*	*4*	*7*	*3*	*27*	*13.2*
202.50	*0*	*0*	*2*	*4*	*1*	*0*	*0*	*7*	*9.3*
225.00	*0*	*1*	*1*	*3*	*3*	*0*	*0*	*8*	*10.4*
247.50	*0*	*13*	*1*	*1*	*3*	*0*	*0*	*18*	*4.8*
270.00	*0*	*15*	*7*	*1*	*3*	*0*	*0*	*26*	*4.7*
292.50	*0*	*3*	*12*	*6*	*12*	*2*	*0*	*35*	*10.2*
315.00	*0*	*2*	*4*	*24*	*39*	*24*	*7*	*100*	*16.0*
337.50	*0*	*0*	*1*	*6*	*6*	*5*	*3*	*21*	*16.3*
360.00	*0*	*0*	*1*	*1*	*2*	*0*	*0*	*4*	*11.0*
Column Sums	*3*	*43*	*53*	*82*	*15*	*72*	*40*	*408*	*13.9*

TABLE 2.3-22

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

MODERATELY UNSTABLE (ΔT -1.9 to -1.7°C/100M)

DIABLO CANYON PERIOD OF RECORD JULY 1967-OCTOBER 1969

FREQUENCY TABLE

		Wind Speed, mph							
Direction	*Calm*	*2.0*	*5.1*	*9.6*	*15.1*	*21.1*	*39.6*	*Row Sums*	*Row Avg*
CALM	*5*	*0*	*0*	*0*	*0*	*0*	*0*	*5*	*0.0*
22.50	*0*	*0*	*1*	*1*	*0*	*0*	*0*	*2*	*8.0*
45.00	*0*	*0*	*0*	*2*	*0*	*0*	*0*	*2*	*10.0*
67.50	*0*	*0*	*0*	*1*	*0*	*0*	*0*	*1*	*12.0*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*0*	*0*	*0*	*1*	*4*	*1*	*0*	*6*	*15.5*
135.00	*0*	*1*	*1*	*3*	*2*	*0*	*1*	*8*	*13.0*
157.50	*0*	*0*	*3*	*4*	*0*	*0*	*8*	*15*	*21.3*
180.00	*0*	*2*	*0*	*2*	*1*	*1*	*2*	*8*	*14.2*
202.50	*0*	*1*	*1*	*0*	*0*	*0*	*0*	*2*	*4.5*
225.00	*0*	*7*	*0*	*2*	*1*	*0*	*0*	*10*	*4.5*
247.50	*0*	*2*	*0*	*0*	*0*	*0*	*0*	*2*	*2.5*
270.00	*0*	*3*	*5*	*0*	*0*	*0*	*0*	*8*	*3.7*
292.50	*0*	*0*	*2*	*5*	*6*	*0*	*0*	*13*	*11.8*
315.00	*0*	*2*	*3*	*5*	*12*	*4*	*1*	*27*	*13.9*
337.50	*0*	*0*	*2*	*0*	*2*	*1*	*0*	*5*	*12.8*
360.00	*0*	*0*	*1*	*1*	*0*	*0*	*0*	*2*	*9.0*
Column Sums	*5*	*18*	*19*	*27*	*28*	*7*	*12*	*116*	*11.9*

TABLE 2.3-23
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
SLIGHTLY UNSTABLE (ΔT -1.7 to -1.5°C/100M)
DIABLO CANYON PERIOD OF RECORD JULY 1967-OCTOBER 1969

FREQUENCY TABLE

			Wind Speed, mph					*Row*	*Row*
Direction	*Calm*	*2.0*	*5.1*	*9.6*	*15.1*	*21.1*	*39.6*	*Sums*	*Avg*
CALM	*6*	*0*	*0*	*0*	*0*	*0*	*0*	*6*	*0.0*
22.50	*0*	*0*	*0*	*1*	*1*	*0*	*0*	*2*	*13.0*
45.00	*0*	*1*	*0*	*1*	*0*	*0*	*0*	*2*	*6.5*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
135.00	*0*	*1*	*2*	*1*	*5*	*5*	*1*	*15*	*15.5*
157.50	*0*	*2*	*10*	*2*	*1*	*1*	*4*	*20*	*13.1*
180.00	*0*	*1*	*0*	*0*	*1*	*0*	*1*	*3*	*18.0*
202.50	*0*	*0*	*1*	*1*	*0*	*0*	*0*	*2*	*6.5*
225.00	*0*	*3*	*0*	*0*	*0*	*0*	*0*	*3*	*1.7*
247.50	*0*	*2*	*1*	*0*	*0*	*0*	*0*	*3*	*3.0*
270.00	*0*	*2*	*5*	*0*	*1*	*2*	*0*	*10*	*8.9*
292.50	*0*	*1*	*2*	*11*	*0*	*1*	*2*	*17*	*11.4*
315.00	*0*	*0*	*1*	*5*	*8*	*9*	*2*	*25*	*17.4*
337.50	*0*	*0*	*0*	*2*	*0*	*0*	*0*	*2*	*12.0*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*6*	*13*	*22*	*24*	*17*	*18*	*10*	*110*	*12.3*

TABLE 2.3-24
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
NEUTRAL (ΔT -1.5 to -0.5°C/100M)
DIABLO CANYON PERIOD OF RECORD JULY 1967-OCTOBER 1969

FREQUENCY TABLE

			Wind Speed, mph					Row	Row
Direction	*Calm*	*2.0*	*5.1*	*9.6*	*15.1*	*21.1*	*39.6*	*Sums*	*Avg*
CALM	*290*	*2*	*0*	*0*	*0*	*0*	*0*	*292*	*0.0*
22.50	*0*	*24*	*36*	*40*	*17*	*4*	*0*	*121*	*8.1*
45.00	*0*	*20*	*35*	*39*	*17*	*1*	*0*	*112*	*8.0*
67.50	*0*	*23*	*20*	*33*	*6*	*0*	*0*	*82*	*6.8*
90.00	*0*	*25*	*18*	*6*	*3*	*0*	*0*	*52*	*4.6*
112.50	*0*	*32*	*51*	*60*	*53*	*9*	*1*	*206*	*9.4*
135.00	*0*	*171*	*284*	*203*	*157*	*54*	*17*	*886*	*8.9*
157.50	*0*	*182*	*155*	*61*	*29*	*23*	*13*	*463*	*6.5*
180.00	*0*	*126*	*46*	*21*	*22*	*17*	*9*	*241*	*6.9*
202.50	*0*	*79*	*16*	*11*	*6*	*6*	*0*	*120*	*4.9*
225.00	*0*	*87*	*12*	*5*	*8*	*2*	*0*	*114*	*3.5*
247.50	*0*	*95*	*20*	*1*	*2*	*3*	*0*	*121*	*3.0*
270.00	*0*	*126*	*96*	*17*	*1*	*4*	*0*	*244*	*4.1*
292.50	*0*	*110*	*223*	*187*	*104*	*28*	*4*	*656*	*8.5*
315.00	*0*	*67*	*242*	*530*	*652*	*308*	*143*	*1942*	*14.2*
337.50	*0*	*42*	*97*	*210*	*160*	*98*	*80*	*687*	*13.9*
360.00	*0*	*41*	*5*	*63*	*53*	*6*	*0*	*218*	*8.7*
Column Sums	*290*	*1252*	*1406*	*1487*	*1290*	*563*	*269*	*6557*	*9.8*

TABLE 2.3-25

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

SLIGHTLY STABLE (ΔT -0.5 to 1.5°C/100M)

DIABLO CANYON PERIOD OF RECORD JULY 1967-OCTOBER 1969

FREQUENCY TABLE

		Wind Speed, mph						Row	Row
Direction	*Calm*	*2.0*	*5.1*	*9.6*	*15.1*	*21.1*	*39.6*	*Sums*	*Avg*
CALM	*405*	*12*	*0*	*0*	*0*	*0*	*0*	*417*	*0.0*
22.50	*0*	*66*	*92*	*96*	*58*	*14*	*2*	*328*	*8.7*
45.00	*0*	*53*	*94*	*66*	*29*	*1*	*0*	*243*	*6.9*
67.50	*0*	*42*	*58*	*35*	*21*	*2*	*0*	*158*	*6.7*
90.00	*0*	*84*	*40*	*13*	*4*	*0*	*0*	*141*	*3.8*
112.50	*0*	*128*	*57*	*25*	*9*	*5*	*0*	*224*	*4.5*
135.00	*0*	*296*	*279*	*164*	*47*	*11*	*5*	*802*	*5.9*
157.50	*0*	*330*	*129*	*16*	*1*	*0*	*4*	*480*	*3.0*
180.00	*0*	*188*	*16*	*2*	*1*	*3*	*1*	*211*	*2.2*
202.50	*0*	*94*	*13*	*2*	*0*	*0*	*0*	*109*	*1.9*
225.00	*0*	*91*	*12*	*4*	*3*	*0*	*0*	*110*	*2.7*
247.50	*0*	*83*	*16*	*1*	*0*	*0*	*0*	*100*	*2.2*
270.00	*0*	*158*	*33*	*5*	*3*	*0*	*0*	*199*	*2.6*
292.50	*0*	*166*	*154*	*132*	*99*	*44*	*12*	*607*	*8.5*
315.00	*0*	*161*	*344*	*454*	*497*	*479*	*304*	*2239*	*14.9*
337.50	*0*	*97*	*136*	*159*	*97*	*62*	*35*	*586*	*10.7*
360.00	*0*	*99*	*123*	*130*	*78*	*20*	*4*	*454*	*8.5*
Column Sums	*405*	*2148*	*1596*	*1304*	*947*	*641*	*367*	*7408*	*8.6*

TABLE 2.3-26

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

MODERATELY STABLE (ΔT 1.5 to 4.0°C/100M)

DIABLO CANYON PERIOD OF RECORD JULY 1967-OCTOBER 1969

FREQUENCY TABLE

		Wind Speed, mph						Row	Row
Direction	*Calm*	*2.0*	*5.1*	*9.6*	*15.1*	*21.1*	*39.6*	*Sums*	*Avg*
CALM	*117*	*1*	*0*	*0*	*0*	*0*	*0*	*118*	*0.0*
22.50	*0*	*9*	*9*	*5*	*6*	*2*	*0*	*31*	*8.0*
45.00	*0*	*12*	*10*	*3*	*3*	*0*	*0*	*28*	*5.1*
67.50	*0*	*12*	*6*	*4*	*1*	*1*	*0*	*24*	*5.6*
90.00	*0*	*20*	*12*	*2*	*0*	*0*	*0*	*34*	*3.0*
112.50	*0*	*33*	*16*	*5*	*0*	*0*	*0*	*54*	*3.4*
135.00	*0*	*54*	*52*	*25*	*2*	*0*	*0*	*133*	*4.6*
157.50	*0*	*68*	*17*	*1*	*0*	*0*	*0*	*86*	*2.3*
180.00	*0*	*35*	*6*	*0*	*0*	*0*	*0*	*41*	*1.8*
202.50	*0*	*20*	*0*	*1*	*0*	*0*	*0*	*21*	*1.8*
225.00	*0*	*18*	*3*	*0*	*0*	*0*	*0*	*21*	*2.0*
247.50	*0*	*30*	*2*	*0*	*0*	*0*	*0*	*32*	*1.7*
270.00	*0*	*34*	*4*	*1*	*0*	*0*	*0*	*39*	*2.5*
292.50	*0*	*38*	*28*	*28*	*8*	*4*	*3*	*109*	*7.2*
315.00	*0*	*43*	*65*	*114*	*167*	*179*	*170*	*738*	*17.3*
337.50	*0*	*20*	*39*	*25*	*15*	*13*	*0*	*112*	*8.7*
360.00	*0*	*20*	*14*	*11*	*7*	*3*	*0*	*55*	*6.9*
Column Sums	*117*	*467*	*283*	*225*	*209*	*202*	*173*	*1676*	*10.1*

TABLE 2.3-27

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

EXTREMELY STABLE (ΔT greater than 4.0°C/100M)

DIABLO CANYON PERIOD OF RECORD JULY 1967-OCTOBER 1969

FREQUENCY TABLE

			Wind Speed, mph					Row	Row
Direction	Calm	2.0	5.1	9.6	15.1	21.1	39.6	Sums	Avg
CALM	46	0	0	0	0	0	0	46	0.0
22.50	0	9	8	6	0	0	0	23	5.2
45.00	0	8	13	3	0	0	0	24	4.6
67.50	0	11	7	1	0	0	0	19	3.5
90.00	0	13	10	1	0	0	0	24	3.7
112.50	0	14	6	1	0	0	0	21	3.3
135.00	0	36	11	2	0	0	0	49	2.9
157.50	0	23	7	1	0	0	0	31	2.8
180.00	0	29	2	0	0	0	0	31	1.5
202.50	0	13	1	0	0	0	0	14	1.7
225.00	0	12	1	0	0	0	0	13	1.6
247.50	0	13	1	0	0	0	0	14	2.2
270.00	0	22	6	2	0	0	0	30	3.0
292.50	0	12	19	14	4	3	0	52	7.4
315.00	0	19	32	73	87	94	95	400	17.7
337.50	0	16	16	12	9	6	0	59	8.5
360.00	0	9	12	5	2	0	0	28	5.7
Column Sums	46	259	152	121	102	103	95	878	10.3

TABLE 2.3-28
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DISTRIBUTION OF WIND SPEED OBSERVATIONS BY STABILITY CLASS

Stability Class	*T, °C/100M*	*Number of Observations*
Extremely unstable	*Less than -1.9*	*3*
Moderately unstable	*-1.9 to -1.7*	*5*
Slightly unstable	*-1.7 to -1.5*	*6*
Neutral	*-1.5 to -0.5*	*290*
Slightly stable	*-0.5 to 1.5*	*405*
Moderately stable	*1.5 to 4.0*	*117*
Extremely stable	*Greater than 4.0*	*46*

(a) Observations for which the mean hourly wind speed was less than one mile per hour when stability is defined by vertical temperature gradient between the 25-foot levels at Station E period of record July 1, 1967 through October 31, 1969.

(b) Total hourly observations for period of record: 17,153.

TABLE 2.3-29
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
VERTICAL ANGLE STABILITY CLASS A

Direction, deg.	*Wind Speed, mph 1.5*	*5.5*	*10.0*	*15.5*	*21.5*	*37.5*	*Row Sum*	*Row Avg.*
Calm	*2*	*0*	*0*	*0*	*0*	*0*	*2*	*0.0*
22.5	*106*	*185*	*63*	*14*	*1*	*0*	*369*	*5.6*
45.0	*127*	*152*	*71*	*12*	*1*	*0*	*363*	*5.3*
67.5	*77*	*69*	*44*	*9*	*0*	*0*	*199*	*5.3*
90.0	*101*	*47*	*16*	*7*	*2*	*0*	*173*	*4.1*
112.5	*97*	*25*	*17*	*11*	*4*	*0*	*144*	*3.9*
135.0	*178*	*111*	*27*	*10*	*3*	*0*	*329*	*4.2*
157.5	*185*	*168*	*22*	*1*	*0*	*0*	*376*	*3.9*
180.0	*209*	*64*	*5*	*1*	*0*	*0*	*279*	*3.0*
202.5	*117*	*19*	*1*	*0*	*0*	*0*	*137*	*2.2*
225.0	*83*	*10*	*1*	*1*	*0*	*0*	*95*	*2.0*
247.5	*90*	*15*	*2*	*1*	*0*	*0*	*108*	*2.2*
270.0	*126*	*23*	*9*	*1*	*0*	*0*	*159*	*2.7*
292.5	*164*	*98*	*60*	*18*	*5*	*3*	*348*	*5.6*
315.0	*108*	*166*	*126*	*64*	*13*	*1*	*478*	*7.7*
337.5	*79*	*126*	*119*	*66*	*15*	*3*	*408*	*8.2*
360.0	*91*	*215*	*146*	*32*	*4*	*0*	*488*	*6.8*
Column Sums	*1,940*	*1,493*	*729*	*238*	*48*	*7*	*4,455*	*5.7*

TABLE 2.3-30

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
VERTICAL ANGLE STABILITY CLASS B

Direction, deg.	*Wind Speed, mph*						*Row Sum*	*Row Avg.*
	1.5	*5.5*	*10.0*	*15.5*	*21.5*	*37.5*		
Calm	*2*	*0*	*0*	*0*	*0*	*0*	*2*	*0.0*
22.5	*32*	*43*	*27*	*12*	*4*	*1*	*119*	*7.1*
45.0	*44*	*55*	*28*	*3*	*0*	*0*	*130*	*5.5*
67.5	*33*	*20*	*18*	*9*	*0*	*0*	*80*	*5.9*
90.0	*46*	*18*	*8*	*2*	*1*	*1*	*76*	*4.4*
112.5	*52*	*19*	*32*	*27*	*6*	*0*	*136*	*7.8*
135.0	*107*	*152*	*104*	*57*	*11*	*1*	*432*	*7.4*
157.5	*94*	*127*	*52*	*10*	*2*	*3*	*288*	*5.6*
180.0	*59*	*47*	*6*	*0*	*0*	*0*	*112*	*3.6*
202.5	*24*	*7*	*0*	*0*	*0*	*0*	*31*	*2.4*
225.0	*19*	*8*	*1*	*0*	*0*	*0*	*28*	*2.5*
247.5	*23*	*6*	*1*	*0*	*0*	*0*	*30*	*2.4*
270.0	*48*	*7*	*2*	*0*	*0*	*0*	*57*	*2.5*
292.5	*74*	*90*	*47*	*33*	*16*	*3*	*263*	*7.6*
315.0	*52*	*143*	*156*	*110*	*65*	*19*	*545*	*11.1*
337.5	*43*	*81*	*102*	*98*	*58*	*8*	*390*	*11.5*
360.0	*32*	*92*	*64*	*21*	*7*	*0*	*216*	*7.6*
Column Sums	*784*	*915*	*648*	*382*	*170*	*36*	*2,935*	*7.9*

TABLE 2.3-31
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
VERTICAL ANGLE STABILITY CLASS C

Direction, deg.	*Wind Speed, mph 1.5*	*5.5*	*10.0*	*15.5*	*21.5*	*37.5*	*Row Sum*	*Row Avg.*
Calm	*2*	*0*	*0*	*0*	*0*	*0*	*2*	*0.0*
22.5	*7*	*12*	*8*	*2*	*1*	*0*	*30*	*7.1*
45.0	*24*	*24*	*6*	*4*	*0*	*0*	*58*	*5.3*
67.5	*19*	*17*	*10*	*5*	*0*	*0*	*51*	*5.6*
90.0	*18*	*6*	*3*	*6*	*0*	*1*	*34*	*6.2*
112.5	*34*	*4*	*19*	*16*	*6*	*3*	*82*	*8.8*
135.0	*76*	*102*	*134*	*63*	*29*	*9*	*413*	*9.3*
157.5	*55*	*96*	*56*	*20*	*6*	*0*	*233*	*6.7*
180.0	*21*	*18*	*2*	*3*	*1*	*1*	*46*	*5.4*
202.5	*10*	*4*	*4*	*0*	*0*	*0*	*17*	*3.5*
225.0	*8*	*6*	*0*	*0*	*0*	*0*	*14*	*3.5*
247.5	*15*	*4*	*0*	*0*	*0*	*0*	*19*	*2.5*
270.0	*32*	*23*	*4*	*0*	*1*	*1*	*61*	*4.3*
292.5	*29*	*94*	*76*	*73*	*43*	*2*	*317*	*10.8*
315.0	*49*	*222*	*388*	*445*	*390*	*148*	*1,642*	*15.0*
337.5	*35*	*65*	*114*	*123*	*93*	*28*	*458*	*13.6*
360.0	*14*	*27*	*12*	*7*	*3*	*0*	*63*	*7.3*
Column Sums	*448*	*724*	*836*	*767*	*573*	*192*	*3,540*	*12.0*

TABLE 2.3-32

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
VERTICAL ANGLE STABILITY CLASS D

Direction, deg.	*Wind Speed, mph 1.5*	*5.5*	*10.0*	*15.5*	*21.5*	*37.5*	*Row Sum*	*Row Avg.*
Calm	*2*	*0*	*0*	*0*	*0*	*0*	*2*	*0.0*
22.5	*1*	*5*	*0*	*0*	*0*	*0*	*6*	*4.5*
45.0	*16*	*4*	*1*	*0*	*0*	*0*	*21*	*3.1*
67.5	*9*	*5*	*4*	*5*	*1*	*0*	*24*	*9.7*
90.0	*15*	*4*	*3*	*0*	*1*	*0*	*23*	*5.4*
112.5	*31*	*5*	*2*	*2*	*0*	*0*	*40*	*4.5*
135.0	*63*	*40*	*15*	*8*	*4*	*5*	*135*	*5.9*
157.5	*30*	*17*	*12*	*5*	*2*	*0*	*66*	*5.7*
180.0	*8*	*4*	*1*	*2*	*1*	*0*	*16*	*6.1*
202.5	*7*	*1*	*0*	*0*	*0*	*0*	*8*	*1.6*
225.0	*4*	*4*	*0*	*1*	*0*	*0*	*9*	*5.2*
247.5	*6*	*5*	*1*	*0*	*0*	*0*	*12*	*3.7*
270.0	*22*	*6*	*4*	*2*	*3*	*0*	*37*	*5.5*
292.5	*14*	*43*	*55*	*55*	*40*	*12*	*219*	*12.7*
315.0	*31*	*181*	*369*	*556*	*463*	*271*	*1,871*	*16.5*
337.5	*16*	*33*	*69*	*85*	*63*	*50*	*316*	*15.6*
360.0	*3*	*11*	*9*	*0*	*0*	*0*	*23*	*6.5*
Column Sums	*278*	*368*	*545*	*721*	*578*	*338*	*2,828*	*14.5*

TABLE 2.3-33

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
VERTICAL ANGLE STABILITY CLASS E

Direction, deg.	*Wind Speed, mph*						*Row Sum*	*Row Avg.*
	1.5	*5.5*	*10.0*	*15.5*	*21.5*	*37.5*		
Calm	*1*	*0*	*0*	*0*	*0*	*0*	*1*	*0.0*
22.5	*0*	*1*	*0*	*0*	*0*	*0*	*1*	*4.0*
45.0	*2*	*1*	*1*	*0*	*0*	*0*	*4*	*3.8*
67.5	*0*	*2*	*3*	*0*	*0*	*0*	*5*	*7.6*
90.0	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.5	*10*	*1*	*0*	*0*	*0*	*0*	*11*	*1.9*
135.0	*15*	*3*	*0*	*0*	*0*	*0*	*18*	*2.3*
157.5	*7*	*2*	*1*	*0*	*2*	*0*	*12*	*2.8*
180.0	*4*	*1*	*0*	*0*	*0*	*0*	*5*	*2.4*
202.5	*2*	*0*	*0*	*1*	*0*	*0*	*3*	*5.3*
225.0	*2*	*2*	*0*	*0*	*0*	*0*	*4*	*3.3*
247.5	*2*	*3*	*1*	*0*	*0*	*0*	*6*	*4.6*
270.0	*1*	*0*	*1*	*1*	*0*	*0*	*3*	*8.3*
292.5	*2*	*8*	*8*	*4*	*11*	*8*	*41*	*15.8*
315.0	*8*	*30*	*42*	*105*	*111*	*47*	*343*	*17.3*
337.5	*3*	*3*	*5*	*4*	*2*	*3*	*20*	*13.2*
360.0	*0*	*0*	*1*	*0*	*0*	*0*	*1*	*8.0*
Column Sums	*59*	*57*	*63*	*115*	*126*	*58*	*478*	*14.8*

TABLE 2.3-34
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
VERTICAL ANGLE STABILITY CLASS F AND G

Direction, deg.	*Wind Speed, mph 1.5*	*5.5*	*10.0*	*15.5*	*21.5*	*37.5*	*Row Avg.*	*Row Sum*
Calm	*516*	*0*	*0*	*0*	*0*	*0*	*516*	*0.0*
22.5	*5*	*0*	*0*	*0*	*0*	*0*	*5*	*1.2*
45.0	*5*	*1*	*0*	*0*	*0*	*0*	*6*	*2.5*
67.5	*11*	*0*	*0*	*0*	*0*	*0*	*11*	*1.7*
90.0	*8*	*1*	*0*	*0*	*0*	*0*	*9*	*1.4*
112.5	*15*	*0*	*0*	*0*	*0*	*0*	*15*	*1.6*
135.0	*55*	*3*	*0*	*0*	*0*	*0*	*58*	*1.7*
157.5	*32*	*2*	*1*	*0*	*0*	*0*	*35*	*1.9*
180.0	*19*	*0*	*1*	*0*	*0*	*0*	*20*	*1.9*
202.5	*11*	*0*	*0*	*0*	*0*	*0*	*11*	*1.4*
225.0	*8*	*0*	*0*	*0*	*0*	*0*	*8*	*1.3*
247.5	*11*	*0*	*0*	*0*	*0*	*0*	*11*	*1.0*
270.0	*17*	*0*	*0*	*0*	*0*	*0*	*17*	*1.3*
292.5	*9*	*5*	*5*	*0*	*2*	*0*	*22*	*6.5*
315.0	*21*	*18*	*25*	*32*	*27*	*15*	*138*	*13.4*
337.5	*15*	*3*	*4*	*4*	*2*	*0*	*28*	*4.8*
360.0	*11*	*4*	*0*	*0*	*0*	*0*	*15*	*2.7*
Column Sums	*769*	*37*	*36*	*36*	*31*	*15*	*925*	*2.7*

TABLE 2.3-35
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
AZIMUTH ANGLE STABILITY CLASS A

Direction, deg.	*Wind Speed, mph*						*Row Sum*	*Row Avg.*
	1.5	*5.5*	*10.0*	*15.5*	*21.5*	*37.5*		
Calm	*1*	*0*		*0*	*0*	*0*	*1*	*0.0*
22.5	*44*	*87*	*26*	*4*	*0*	*0*	*161*	*5.4*
45.0	*42*	*88*	*46*	*8*	*0*	*0*	*184*	*6.0*
67.5	*35*	*43*	*40*	*4*	*0*	*0*	*122*	*6.0*
90.0	*63*	*34*	*12*	*1*	*0*	*0*	*110*	*3.7*
112.5	*61*	*11*	*4*	*0*	*0*	*0*	*76*	*2.8*
135.0	*84*	*32*	*4*	*2*	*0*	*0*	*122*	*3.1*
157.5	*54*	*26*	*4*	*0*	*0*	*0*	*84*	*3.2*
180.0	*55*	*17*	*2*	*0*	*0*	*0*	*74*	*2.7*
202.5	*39*	*6*	*1*	*0*	*0*	*0*	*46*	*2.6*
225.0	*25*	*3*	*2*	*1*	*0*	*0*	*31*	*3.1*
247.5	*41*	*5*	*1*	*0*	*0*	*0*	*47*	*2.0*
270.0	*46*	*12*	*6*	*0*	*0*	*0*	*64*	*3.2*
292.5	*32*	*29*	*16*	*6*	*1*	*0*	*84*	*5.7*
315.0	*28*	*55*	*53*	*23*	*6*	*2*	*167*	*8.6*
337.5	*32*	*71*	*53*	*13*	*3*	*1*	*173*	*7.1*
360.0	*41*	*96*	*40*	*11*	*1*	*0*	*189*	*6.4*
Column Sums	*723*	*615*	*310*	*73*	*11*	*3*	*1,735*	*5.2*

TABLE 2.3-36

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
AZIMUTH ANGLE STABILITY CLASS B

Direction, deg.	*Wind Speed, mph 1.5*	*5.5*	*10.0*	*15.5*	*21.5*	*37.5*	*Row Sum*	*Row Avg.*
Calm	*2*	*0*	*0*	*0*	*0*	*0*	*2*	*0.0*
22.5	*31*	*43*	*18*	*7*	*0*	*0*	*99*	*5.9*
45.0	*30*	*38*	*22*	*2*	*0*	*0*	*92*	*5.4*
67.5	*24*	*19*	*13*	*3*	*0*	*0*	*59*	*5.1*
90.0	*26*	*12*	*3*	*5*	*1*	*0*	*47*	*5.6*
112.5	*22*	*10*	*4*	*0*	*1*	*0*	*37*	*4.7*
135.0	*40*	*14*	*4*	*1*	*0*	*0*	*59*	*3.2*
157.5	*25*	*19*	*1*	*0*	*0*	*0*	*45*	*3.7*
180.0	*20*	*5*	*0*	*0*	*0*	*0*	*25*	*2.4*
202.5	*20*	*3*	*0*	*0*	*0*	*0*	*23*	*2.5*
225.0	*17*	*2*	*0*	*0*	*0*	*0*	*19*	*2.4*
247.5	*21*	*4*	*2*	*0*	*0*	*0*	*27*	*2.8*
270.0	*25*	*9*	*4*	*0*	*0*	*0*	*38*	*3.6*
292.5	*22*	*22*	*9*	*1*	*0*	*1*	*55*	*5.7*
315.0	*13*	*23*	*27*	*20*	*12*	*3*	*98*	*10.8*
337.5	*19*	*24*	*31*	*20*	*4*	*1*	*99*	*9.1*
360.0	*20*	*64*	*61*	*16*	*3*	*0*	*164*	*8.0*
Column Sums	*377*	*311*	*199*	*75*	*21*	*5*	*988*	*6.2*

TABLE 2.3-37

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
AZIMUTH ANGLE STABILITY CLASS C

Direction, deg.	*Wind Speed, mph 1.5*	*5.5*	*10.0*	*15.5*	*21.5*	*37.5*	*Row Sum*	*Row Avg.*
Calm	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
22.5	*34*	*58*	*35*	*8*	*3*	*0*	*138*	*6.5*
45.0	*44*	*53*	*27*	*5*	*1*	*0*	*130*	*5.6*
67.5	*24*	*24*	*11*	*6*	*1*	*0*	*66*	*5.6*
90.0	*21*	*12*	*7*	*3*	*2*	*0*	*45*	*5.4*
112.5	*43*	*12*	*6*	*5*	*1*	*0*	*67*	*4.2*
135.0	*79*	*43*	*19*	*8*	*1*	*0*	*150*	*4.8*
157.5	*54*	*43*	*11*	*2*	*0*	*0*	*110*	*4.3*
180.0	*39*	*9*	*1*	*0*	*0*	*0*	*49*	*2.6*
202.5	*28*	*5*	*0*	*0*	*0*	*0*	*33*	*1.9*
225.0	*19*	*6*	*1*	*0*	*0*	*0*	*26*	*2.7*
247.5	*29*	*3*	*1*	*0*	*0*	*0*	*33*	*2.3*
270.0	*34*	*6*	*2*	*1*	*0*	*0*	*44*	*2.9*
292.5	*49*	*36*	*23*	*11*	*5*	*0*	*124*	*6.4*
315.0	*36*	*55*	*78*	*56*	*36*	*3*	*270*	*11.2*
337.5	*26*	*50*	*65*	*59*	*24*	*7*	*229*	*11.1*
360.0	*30*	*78*	*75*	*18*	*2*	*4*	*203*	*8.9*
Column Sums	*589*	*93*	*363*	*182*	*76*	*14*	*1,717*	*7.2*

TABLE 2.3-38

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
AZIMUTH ANGLE STABILITY CLASS D

Direction, deg.	Wind Speed, mph 1.5	5.5	10.0	15.5	21.5	37.5	Row Sum	Row Avg.
Calm	1	0	0	0	0	0	1	0.0
22.5	34	44	17	9	3	1	108	6.4
45.0	55	54	22	5	0	0	136	5.1
67.5	34	16	16	10	0	0	76	6.0
90.0	46	23	9	6	1	1	86	5.6
112.5	56	17	35	24	7	1	140	7.9
135.0	126	178	122	65	15	6	512	7.5
157.5	106	148	45	9	3	1	312	5.2
180.0	70	36	6	2	1	0	115	3.7
202.5	27	7	0	0	0	0	34	2.3
225.0	30	8	2	0	0	0	40	2.6
247.5	23	8	0	0	0	0	31	2.3
270.0	53	9	4	0	1	1	68	3.4
292.5	73	81	62	43	32	10	301	9.2
315.0	69	171	222	209	138	47	856	12.6
337.5	35	83	116	139	109	25	507	13.4
360.0	39	62	53	15	8	0	177	7.4
Column Sums	877	945	731	536	318	93	3,500	9.0

TABLE 2.3-39
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
AZIMUTH ANGLE STABILITY CLASS E

Direction, deg.	*Wind Speed, mph*						*Row Sum*	*Row Avg.*
	1.5	*5.5*	*10.0*	*15.5*	*21.5*	*37.5*		
Calm	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
22.5	*11*	*13*	*4*	*1*	*0*	*0*	*29*	*5.2*
45.0	*44*	*32*	*4*	*2*	*0*	*0*	*82*	*4.0*
67.5	*28*	*19*	*17*	*7*	*0*	*0*	*71*	*5.8*
90.0	*30*	*8*	*2*	*0*	*0*	*1*	*41*	*3.9*
112.5	*47*	*10*	*11*	*8*	*2*	*1*	*79*	*5.6*
135.0	*120*	*116*	*96*	*56*	*25*	*7*	*420*	*8.0*
157.5	*105*	*136*	*69*	*18*	*6*	*2*	*336*	*6.1*
180.0	*64*	*41*	*4*	*3*	*1*	*1*	*114*	*4.0*
202.5	*20*	*5*	*2*	*1*	*0*	*0*	*28*	*3.3*
225.0	*24*	*10*	*1*	*0*	*0*	*0*	*35*	*3.0*
247.5	*22*	*8*	*2*	*0*	*0*	*0*	*32*	*2.9*
270.0	*47*	*23*	*5*	*2*	*2*	*0*	*79*	*4.2*
292.5	*72*	*129*	*106*	*90*	*54*	*9*	*460*	*10.2*
315.0	*83*	*319*	*549*	*696*	*608*	*292*	*2,547*	*15.5*
337.5	*46*	*63*	*126*	*120*	*101*	*61*	*517*	*14.3*
360.0	*20*	*29*	*13*	*5*	*0*	*0*	*67*	*6.0*
Column Sums	*783*	*961*	*1,011*	*1,009*	*799*	*374*	*4,937*	*12.1*

TABLE 2.3-40

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STATION E 25-FOOT LEVEL
OCTOBER 1969 THROUGH MARCH 1971 AND
APRIL 1972 THROUGH SEPTEMBER 1972
AZIMUTH ANGLE STABILITY CLASS F AND G

Direction, deg.	*Wind Speed, mph*						*Row Sum*	*Row Avg.*
	1.5	*5.5*	*10.0*	*15.5*	*21.5*	*37.5*		
Calm	*564*	*0*	*0*	*0*	*0*	*0*	*564*	*0.0*
22.5	*7*	*2*	*0*	*0*	*0*	*0*	*9*	*2.3*
45.0	*17*	*4*	*0*	*0*	*0*	*0*	*21*	*2.5*
67.5	*17*	*2*	*2*	*1*	*0*	*0*	*22*	*3.6*
90.0	*15*	*3*	*1*	*0*	*0*	*0*	*19*	*2.5*
112.5	*27*	*1*	*0*	*1*	*0*	*0*	*29*	*2.0*
135.0	*75*	*19*	*6*	*2*	*2*	*1*	*105*	*3.4*
157.5	*65*	*31*	*5*	*3*	*2*	*0*	*106*	*3.8*
180.0	*52*	*7*	*2*	*1*	*0*	*0*	*62*	*2.5*
202.5	*29*	*4*	*1*	*0*	*0*	*0*	*34*	*2.0*
225.0	*16*	*1*	*0*	*1*	*0*	*0*	*18*	*2.2*
247.5	*17*	*4*	*1*	*0*	*0*	*0*	*22*	*2.3*
270.0	*55*	*9*	*1*	*0*	*0*	*0*	*65*	*2.2*
292.5	*50*	*55*	*53*	*36*	*23*	*7*	*224*	*9.4*
315.0	*56*	*151*	*222*	*314*	*286*	*172*	*1,201*	*15.8*
337.5	*32*	*15*	*21*	*37*	*9*	*5*	*118*	*10.4*
360.0	*9*	*7*	*2*	*0*	*0*	*0*	*18*	*3.7*
Column Sums	*1,103*	*315*	*317*	*396*	*322*	*185*	*2,637*	*9.4*

TABLE 2.3-41

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

CUMULATIVE PERCENTAGE DISTRIBUTIONS OF χ/Q ESTIMATES BASED ON DISTANCE AND WIND SECTOR CENTERLINE FOR GROUND LEVEL RELEASES

10-meter wind data and stability categories based on measured Sigma A and Temperature Gradient (76M - 10M) values. For calculations with wind speed below 1.5 meters per second stability is based on Temperature Gradient only and building wake or a meander factor is considered - with wind speeds above 1.5 meters per second stability is based on measured Sigma A and Temperature Gradient with building wake only considered. X is downwind distance in meters, Y is sector centerline from north in degrees, and Z is terrain height defined as zero for Ground Level Releases. Data Period May 1973 through April 1975. In the following Tables Y=0.0 is equivalent to Y=360°=North.

CUMULATIVE FREQUENCY DISTRIBUTION AT X=800.0 Y=315.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	*0.0*	*0.0*	*0.0*	*0.0*	*0.60140297E-06*	*0.17928305E-05*
50	*0.0*	*0.0*	*0.34090243E-06*	*0.11967086E-05*	*0.22880003E-05*	*0.36180809E-05*
75	*0.0*	*0.35322555E-05*	*0.47339599E-05*	*0.52474825E-05*	*0.51153211E-05*	*0.56741301E-05*
90	*0.46975747E-05*	*0.13407243E-04*	*0.12096141E-04*	*0.11476736E-04*	*0.96943622E-05*	*0.75202488E-05*
95	*0.26914247E-04*	*0.21392218E-04*	*0.18613035E-04*	*0.16908802E-04*	*0.13674124E-04*	*0.82745992E-05*
99	*0.79830948E-04*	*0.41694584E-04*	*0.31726566E-04*	*0.28974857E-04*	*0.22724547E-04*	*0.93198487E-05*
99.5	*0.10060299E-03*	*0.48705522E-04*	*0.38378115E-04*	*0.35206263E-04*	*0.25349524E-04*	*0.97140346E-05*
99.9	*0.17863358E-03*	*0.69454283E-04*	*0.52891977E-04*	*0.55085635E-04*	*0.29252842E-04*	*0.98666351E-05*
100	*0.42693969E-03*	*0.16204809E-03*	*0.91344118E-04*	*0.63421554E-04*	*0.31318254E-04*	*0.10198001E-04*

CUMULATIVE FREQUENCY DISTRIBUTION AT X=800.0 Y=337.5 Z=0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	*0.0*	*0.0*	*0.0*	*0.0*	*0.27897920E-06*	*0.14245843E-05*
50	*0.0*	*0.0*	*0.39965435E-07*	*0.47623541E-06*	*0.11941902E-05*	*0.21688302E-05*
75	*0.0*	*0.11479096E-05*	*0.25813661E-05*	*0.28035602E-05*	*0.31818436E-05*	*0.30996152E-05*
90	*0.16196464E-06*	*0.73396404E-05*	*0.71391196E-05*	*0.69225625E-05*	*0.60822440E-05*	*0.39170363E-05*
95	*0.10839826E-04*	*0.13190673E-04*	*0.11428615E-04*	*0.10343385E-04*	*0.76751812E-05*	*0.42903375E-05*
99	*0.57332712E-04*	*0.28498354E-04*	*0.21073996E-04*	*0.16707505E-04*	*0.10494983E-04*	*0.51341722E-05*
99.5	*0.77042845E-04*	*0.34250028E-04*	*0.23407832E-04*	*0.18469131E-04*	*0.11948351E-04*	*0.52089890E-05*
99.9	*0.11422510E-03*	*0.46669331E-04*	*0.30434865E-04*	*0.26693204E-04*	*0.17508937E-04*	*0.54098209E-05*
100	*0.45017432E-03*	*0.59372076E-04*	*0.29615683E-04*	*0.30263240E-04*	*0.18352424E-04*	*0.55004302E-05*

CUMULATIVE FREQUENCY DISTRIBUTION AT X=800.0 Y=0.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.20178135E-07	0.61033268E-06
50	0.0	0.0	0.0	0.84964356E-08	0.52331109E-06	0.96415260E-06
75	0.0	0.25302236E-08	0.70840883E-06	0.11574984E-05	0.13509989E-05	0.12979572E-05
90	0.0	0.30571773E-05	0.34360637E-05	0.30373176E-05	0.24284118E-05	0.16593158E-05
95	0.11744612E-06	0.66978700E-05	0.51172337E-05	0.42316142E-05	0.33221295E-05	0.19114732E-05
99	0.33281089E-04	0.14604380E-04	0.10118109E-04	0.86893342E-05	0.67918800E-05	0.23723878E-05
99.5	0.49149618E-04	0.18833016E-04	0.13381233E-04	0.12046017E-04	0.82666420E-05	0.24336141E-05
99.9	0.88619912E-04	0.34606783E-04	0.24519803E-04	0.19771018E-04	0.94038032E-05	0.25613817E-05
100	0.31923875E-03	0.49039605E-04	0.29722723E-04	0.24404493E-04	0.10373947E-04	0.25717727E-05

CUMULATIVE FREQUENCY DISTRIBUTION AT X=800.0 Y=22.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.31590432E-08	0.33214155E-06
50	0.0	0.0	0.0	0.0	0.31713915E-06	0.54789928E-06
75	0.0	0.0	0.12785688E-06	0.61485019E-06	0.78384534E-06	0.75719402E-06
90	0.0	0.14533725E-05	0.21641408E-05	0.18840965E-05	0.15855258E-05	0.10827689E-05
95	0.0	0.41104977E-05	0.33112265E-05	0.28097411E-05	0.22921749E-05	9.12460659E-05
99	0.20790569E-04	0.10313162E-04	0.76756214E-05	0.57558864E-05	0.31460195E-05	0.15083806E-05
99.5	0.36712212E-04	0.13969571E-04	0.84455642E-05	0.69937705E-05	0.38429389E-05	0.15696178E-05
99.9	0.64066669E-04	0.20981301E-04	0.12264602E-04	0.12232087E-04	0.47792591E-05	0.16129306E-05
100	0.29356778E-03	0.36696263E-04	0.18348132E-04	0.14337682E-04	0.54107604E-05	0.16385302E-05

CUMULATIVE FREQUENCY DISTRIBUTION AT X=800.0 Y=45.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.55598703E-09	0.35731915E-06
50	0.0	0.0	0.0	0.0	0.20440370E-06	0.49795892E-06
75	0.0	0.0	0.86261821E-07	0.39869042E-06	0.85617688E-06	0.81063536E-06
90	0.0	0.11350148E-05	0.24830606E-05	0.22180611E-05	0.16361364E-05	0.12692826E-05
95	0.0	0.49661339E-05	0.38527442E-05	0.31920190E-05	0.23277044E-05	0.14467960E-05
99	0.19482410E-04	0.11177684E-04	0.85418196E-05	0.70383176E-05	0.53034983E-05	0.17087514E-05
99.5	0.42459200E-04	0.15879719E-04	0.10553575E-04	0.93715735E-05	0.57840080E-05	0.20348043E-05
99.9	0.77170160E-04	0.28114708E-04	0.16107486E-04	0.15625614E-04	0.76384376E-05	0.21556871E-05
100	0.37501496E-03	0.46876870E-04	0.23438435E-04	0.15689511E-04	0.81559456E-05	0.21701553E-05

CUMULATIVE FREQUENCY DISTRIBUTION AT X=800.0 Y=67.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.19910218E-08	0.24889209E-06
50	0.0	0.0	0.0	0.0	0.14546737E-06	0.42911802E-06
75	0.0	0.0	0.10832360E-06	0.32727348E-06	0.79993595E-06	0.62177327E-06
90	0.0	0.88918227E-06	0.17428902E-05	0.17519760E-05	0.13282652E-05	0.91909624E-06
95	0.0	0.34773839E-05	0.31748004E-05	0.28340100E-05	0.18950996E-05	0.11182974E-05
99	0.16080114E-04	0.89836503E-05	0.65093709E-05	0.54437296E-05	0.31568488E-05	0.19424133E-05
99.5	0.32439624E-04	0.12748404E-04	0.83791401E-05	0.62988538E-05	0.35999619E-05	0.19563859E-05
99.9	0.63343803E-04	0.18939914E-04	0.11131005E-04	0.77977775E-05	0.47309759E-05	0.19899007E-05
100	0.16785040E-03	0.30303869E-04	0.11655334E-04	0.92206838E-05	0.59454760E-05	0.20108682E-05

CUMULATIVE FREQUENCY DISTRIBUTION AT X=800.0 Y=90.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.98381292E-07	0.42774644E-06
50	0.0	0.0	0.67347941E-08	0.10075223E-06	0.47905326E-06	0.81535012E-06
75	0.0	0.11733863E-06	0.85482128E-06	0.11202137E-05	0.12998180E-05	0.12833762E-05
90	0.0	0.30206447E-05	0.28861887E-05	0.27604883E-05	0.22959002E-05	0.16845443E-05
95	0.47983724E-06	0.56365625E-05	0.45940978E-05	0.41290305E-05	0.30253241E-05	0.18356177E-05
99	0.30167124E-04	0.12510503E-04	0.89678560E-05	0.74601758E-05	0.54740649E-05	0.24251367E-05
99.5	0.43825232E-04	0.15991667E-04	0.11190264E-04	0.10325079E-04	0.61040009E-05	0.25050431E-05
99.9	0.80253856E-04	0.25524816E-04	0.16437087E-04	0.13011633E-04	0.64430151E-05	0.26079852E-05
100	0.26299339E-03	0.32874173E-04	0.25596077E-04	0.17605096E-04	0.77006334E-05	0.26970001E-05

CUMULATIVE FREQUENCY DISTRIBUTION AT X=800.0 Y=112.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.21881338E-07	0.17739683E-06	0.80441509E-06	0.15274791E-05
50	0.0	0.15332762E-06	0.99273075E-06	0.15290261E-05	0.21965543E-05	0.28209042E-05
75	0.36544221E-08	0.36351485E-05	0.45084162E-05	0.49050886E-05	0.52850155E-05	0.55923738E-05
90	0.58355099E-05	0.11542677E-04	0.10774902E-04	0.99791041E-05	0.84557751E-05	0.71882914E-05
95	0.24372421E-04	0.19057174E-04	0.15440848E-04	0.13660998E-04	0.11346160E-04	0.84373014E-05
99	0.73329080E-04	0.35874895E-04	0.26128837E-04	0.22772845E-04	0.17065628E-04	0.94562383E-05
99.5	0.92018949E-04	0.41281746E-04	0.31428004E-04	0.29057730E-04	0.19443658E-04	0.98133150E-05
99.9	0.13031083E-03	0.60571503E-04	0.44662738E-04	0.35700417E-04	0.23957633E-04	0.10166443E-04
100	0.25177584E-03	0.87236898E-04	0.52296658E-04	0.39889244E-04	0.25460802E-04	0.10185076E-04

CUMULATIVE FREQUENCY DISTRIBUTION AT X=800.0 Y=135.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.15051785E-06	0.19566469E-05	0.29644816E-05	0.44178805E-05	0.58212872E-05
50	0.81038642E-08	0.53202129E-05	0.68629924E-05	0.74960581E-05	0.86957034E-05	0.92801483E-05
75	0.10795834E-04	0.14239811E-04	0.13835153E-04	0.13944897E-04	0.13847120E-04	0.13906842E-04
90	0.31399934E-04	0.25514790E-04	0.22496853E-04	0.20765699E-04	0.18985549E-04	0.15998783E-04
95	0.47333873E-04	0.34454075E-04	0.29741001E-04	0.27354195E-04	0.21924367E-04	0.17443468E-04
99	0.98935401E-04	0.62552077E-04	0.51852519E-04	0.41265914E-04	0.32405180E-04	0.20288542E-04
99.5	0.13996252E-03	0.73905539E-04	0.57608209E-04	0.50959148E-04	0.36743804E-04	0.21132306E-04
99.9	0.21938581E-03	0.92197675E-04	0.76897748E-04	0.72839248E-04	0.58351958E-04	0.22017281E-04
100	0.43604663E-03	0.12359285E-03	0.10618559E-03	0.90063026E-04	0.63509433E-04	0.23351851E-04

CUMULATIVE FREQUENCY DISTRIBUTION AT X=5000.0 Y=315.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.36168682E-07	0.10453664E-06
50	0.0	0.0	0.85387946E-08	0.58115610E-07	0.13548572E-06	0.22112152E-06
75	0.0	0.17811465E-06	0.27010481E-06	0.30452969E-06	0.31845224E-06	0.36362576E-06
90	0.13189924E-06	0.80447398E-06	0.75813131E-06	0.73916146E-06	0.64316288E-06	0.50780699E-06
95	0.14717771E-05	0.13776043E-05	0.12477758E-05	0.11084267E-05	0.88879460E-06	0.56026227E-06
99	0.54080638E-05	0.29498933E-05	0.22118938E-05	0.19417621E-05	0.16314389E-05	0.61426806E-06
99.5	0.72580106E-05	0.35909725E-05	0.27032129E-05	0.25552408E-05	0.19728277E-05	0.63870863E-06
99.9	0.15196728E-04	0.59458198E-05	0.44717808E-05	0.66192533E-05	0.26420630E-05	0.64619587E-06
100	0.84131505E-04	0.20947293E-04	0.10490432E-04	0.71668255E-05	0.29852199E-05	0.66191802E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=5000.0 Y=337.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.13080108E-07	0.80222492E-07
50	0.0	0.0	0.34313286E-09	0.19187748E-07	0.69095165E-07	0.12573850E-06
75	0.0	0.35439491E-07	0.13064300E-06	0.15438911E-06	0.18920201E-06	0.17924884E-06
90	0.10963741E-08	0.41085059E-06	0.41715919E-06	0.41353013E-06	0.36168058E-06	0.22895489E-06
95	0.43192472E-06	0.78051630E-06	0.69862722E-06	0.63962761E-06	0.45303062E-06	0.26817463E-06
99	0.36181909E-05	0.18028550E-05	0.12881756E-05	0.10581916E-05	0.65855136E-06	0.29816505E-06
99.5	0.51098368E-05	0.22534186E-05	0.15325004E-05	0.11887769E-05	0.76233380E-06	0.30169258E-06
99.9	0.90557323E-05	0.33098568E-05	0.18670871E-05	0.14873640E-05	0.91363347E-06	0.31337936E-06
100	0.21146378E-04	0.37607297E-05	0.22657759E-05	0.17264520E-05	0.96313761E-06	0.31662830E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=5000.0 Y=0.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.14577914E-09	0.35206888E-07
50	0.0	0.0	0.0	0.45826593E-10	0.29770831E-07	0.52756821E-07
75	0.0	0.58812052E-11	0.27133446E-07	0.58794626E-07	0.75742776E-07	0.73219780E-07
50	0.0	0.15007060E-06	0.19557928E-06	0.17316466E-06	0.12850018E-06	0.11007444E-06
95	0.59110117E-09	0.38010899E-06	0.29189255E-06	0.24359178E-06	0.19158728E-06	0.12307993E-06
99	0.17581433E-05	0.83792327E-06	0.62629374E-06	0.58788004E-06	0.49176458E-06	0.14301321E-06
99.5	0.29224684E-05	0.11764350E-05	0.91553710E-06	0.82102144E-06	0.56503490E-06	0.14409750E-06
99.9	0.56128920E-05	0.24570221E-05	0.21853366E-05	0.17620714E-05	0.10170143E-05	0.14915304E-06
100	0.42233936E-04	0.52862160E-05	0.26431080E-05	0.28504437E-05	0.10222102E-05	0.15006066E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=5000.0 Y=22.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.20389176E-10	0.19992385E-07
50	0.0	0.0	0.0	0.0	0.13813594E-07	0.28445811E-07
75	0.0	0.0	0.23833167E-08	0.24813552E-07	0.42410218E-07	0.40333958E-07
90	0.0	0.54455477E-07	0.11946088E-06	0.10623080E-06	0.94955624E-07	0.62961988E-07
95	0.0	0.22869460E-06	0.19328428E-06	0.16950753E-06	0.13235518E-06	0.75315427E-07
99	0.11487864E-05	0.60345269E-06	0.46176353E-06	0.36859063E-06	0.18517102E-06	0.90167873E-07
99.5	0.21258884E-05	0.90078481E-06	0.56635531E-06	0.42334409E-06	0.24530493E-06	0.90845560E-07
99.9	0.40991818E-05	0.12700320E-05	0.76666845E-06	0.98082091E-06	0.37735197E-06	0.92208381E-07
100	0.23539702E-04	0.29424627E-05	0.14712314E-05	0.11320553E-05	0.37735197E-06	0.93904873E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=5000.0 Y=45.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.14582606E-11	0.20480179E-07
50	0.0	0.0	0.0	0.0	0.10482022E-07	0.29226378E-07
75	0.0	0.0	0.10083996E-08	0.14940628E-07	0.50052737E-07	0.50985456E-07
90	0.0	0.43275435E-07	0.13510913E-06	0.12896214E-06	0.10230991E-06	0.85150248E-07
95	0.0	0.27654005E-06	0.24085580E-06	0.20941150E-06	0.14324081E-06	0.93926701E-07
99	0.10752683E-05	0.72607270E-06	0.58212339E-06	0.46406512E-06	0.39380984E-06	0.12303661E-06
99.5	0.25677864E-05	0.10686435E-05	0.78176242E-06	0.65926480E-06	0.44854397E-06	0.15118201E-06
99.9	0.53723543E-05	0.19507843E-05	0.17475313E-05	0.12004366E-05	0.52173317E-06	0.15981925E-06
100	0.28810478E-04	0.36013098E-05	0.18006549E-05	0.12010714E-05	0.56730175E-06	0.16089183E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=5000.0 Y=67.5 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.71000514E-11	0.14641117E-07
50	0.0	0.0	0.0	0.0	0.70953732E-08	0.25460412E-07
75	0.0	0.0	0.14370967E-08	0.15539950E-07	0.47672188E-07	0.39219451E-07
90	0.0	0.31356308E-07	0.95708401E-07	0.10693691E-06	0.83028453E-07	0.52725785E-07
95	0.0	0.19428228E-06	0.20763008E-06	0.16664444E-06	0.12739076E-06	0.63567370E-07
99	0.87763675E-06	0.55491716E-06	0.43887115E-06	0.35722360E-06	0.18691651E-06	0.12359015E-06
99.5	0.20100751E-05	0.87702165E-06	0.58959779E-06	0.42241618E-06	0.20926058E-06	0.12442842E-06
99.9	0.40991790E-05	0.14034076E-05	0.70586043E-06	0.50623231E-06	0.30232917E-06	0.12563163E-06
100	0.12149576E-04	0.24299152E-05	0.93458272E-06	0.58561466E-06	0.37802903E-06	0.12742345E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=5000.0 Y=90.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.40193058E-08	0.23817272E-07
50	0.0	0.0	0.25931188E-10	0.20316566E-08	0.25079416E-07	0.43462933E-07
75	0.0	0.12126509E-08	0.35333194E-07	0.55407174E-07	0.67701819E-07	0.66386406E-07
90	0.0	0.14632678E-06	0.15654877E-06	0.15234110E-06	0.12525743E-06	0.98119585E-07
95	0.42502215E-08	0.31009449E-06	0.26077959E-06	0.23628263E-06	0.18326324E-06	0.11449896E-06
99	0.16467056E-05	0.79867789E-06	0.58171133E-06	0.52813391E-06	0.34962306E-06	0.17958166E-06
99.5	0.26262996E-05	0.10492295E-05	0.89531187E-06	0.76633961E-06	0.43603438E-06	0.18571274E-06
99.9	0.58168962E-05	0.21671476E-05	0.17486946E-05	0.11915372E-05	0.50521476E-06	0.19400682E-06
100	0.28155948E-04	0.35194935E-05	0.19037416E-05	0.12691607E-05	0.66004691E-06	0.20071275E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=5000.0 Y=112.5 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.21045696E-09	0.42891095E-08	0.41060400E-07	0.84093926E-07
50	0.0	0.19965642E-08	0.40799645E-07	0.77697678E-07	0.12948186E-06	0.16824447E-06
75	0.95705752E-11	0.18900982E-06	0.26854855E-06	0.29308774E-06	0.31357575E-06	0.32824516E-06
90	0.18181407E-06	0.71684804E-06	0.64501506E-06	0.61001066E-06	0.51318074E-06	0.45536137E-06
95	0.13917124E-05	0.11540496E-05	0.95202239E-06	0.85374086E-06	0.71276997E-06	0.51387065E-06
99	0.49232312E-05	0.22464483E-05	0.19052277E-05	0.15245078E-05	0.10616695E-05	0.57821558E-06
99.5	0.62745294E-05	0.29965740E-05	0.22118547E-05	0.19582285E-05	0.12047130E-05	0.59775459E-06
99.9	0.10065412E-04	0.42593965E-05	0.29897665E-05	0.24965957E-05	0.15395262E-05	0.60724216E-06
100	0.18854073E-04	0.69515136E-05	0.40005962E-05	0.26940934E-05	0.16514177E-05	0.60842700E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=5000.0 Y=135.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.23127003E-08	0.10091179E-06	0.16987104E-06	0.27334886E-06	0.38332479E-06
50	0.21753807E-10	0.29918658E-06	0.41405730E-06	0.45959354E-06	0.55291048E-06	0.61919405E-06
75	0.54507149E-06	0.90911567E-06	0.91407458E-06	0.91384572E-06	0.93934290E-06	0.97202701E-06
90	0.20855923E-05	0.17987522E-05	0.15966352E-05	0.15276491E-05	0.14050647E-05	0.11763577E-05
95	0.23725582E-05	0.26130037E-05	0.23192615E-05	0.21618853E-05	0.16822378E-05	0.12873825E-05
99	0.82008863E-05	0.53806925E-05	0.41197482E-05	0.35819121E-05	0.27855021E-05	0.15215419E-05
99.5	0.12291127E-04	0.63633797E-05	0.50722783E-05	0.40628656E-05	0.34123441E-05	0.16112281E-05
99.9	0.21577056E-04	0.95259120E-05	0.79465844E-05	0.73275551E-05	0.51353773E-05	0.17033917E-05
100	0.48696151E-04	0.12974342E-04	0.11042428E-04	0.87179824E-05	0.56531680E-05	0.17843304E-05

CUMULATIVE FREQUENCY DISTRIBUTION AT X=10000.0 Y=315.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.11898855E-07	0.39420250E-07
50	0.0	0.0	0.16825294E-08	0.17398602E-07	0.48724825E-07	0.81325879E-07
75	0.0	0.57269261E-07	0.95579821E-07	0.10819360E-06	0.11580113E-06	0.13432509E-06
90	0.26942164E-07	0.29764158E-06	0.28247553E-06	0.26897686E-06	0.24290699E-06	0.19045433E-06
95	0.50944533E-06	0.51394477E-06	0.46507063E-06	0.40625190E-06	0.33735540E-06	0.21974654E-06
99	0.20018133E-05	0.11472730E-05	0.87053417E-06	0.76213655E-06	0.63080751E-06	0.24315244E-06
99.5	0.28199247E-05	0.14036650E-05	0.10350741E-05	0.95666292E-06	0.74433427E-06	0.24778973E-06
99.9	0.60016846E-05	0.23125722E-05	0.17085695E-05	0.31087411E-05	0.12042174E-05	0.25372725E-06
100	0.44052867E-04	0.98463388E-05	0.49264872E-05	0.33393026E-05	0.13460522E-05	0.25695738E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=10000.0 Y=337.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.38649119E-08	0.29699688E-07
50	0.0	0.0	0.30288896E-10	0.48866617E-08	0.23919647E-07	0.44517531E-07
75	0.0	0.79878504E-08	0.48146685E-07	0.55628789E-07	0.67815961E-07	0.64992946E-07
90	0.87539087E-10	0.14584339E-06	0.15432619E-06	0.15032344E-06	0.13241515E-06	0.83584041E-07
95	0.12040977E-06	0.28320778E-06	0.25935333E-06	0.23495539E-06	0.16971364E-06	0.97883003E-07
99	0.13245890E-05	0.67357894E-06	0.48515130E-06	0.40291360E-06	0.24880012E-06	0.10802484E-06
99.5	0.19078780E-05	0.87823923E-06	0.57994760E-06	0.44074397E-06	0.27561646E-06	0.11238512E-06
99.9	0.35605253E-05	0.13149829E-05	0.72390708E-06	0.52097437E-06	0.30930397E-06	0.11448327E-06
100	0.81129356E-05	0.14976467E-05	0.78242454E-06	0.62768413E-06	0.32477283E-06	0.11568238E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=10000.0 Y=0.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.16834784E-10	0.12309329E-07
50	0.0	0.0	0.0	0.31556008E-11	0.10926414E-07	0.19151265E-07
75	0.0	0.27285314E-12	0.70115966E-08	0.20003114E-07	0.27724038E-07	0.26369385E-07
90	0.0	0.49454080E-07	0.68042254E-07	0.63437994E-07	0.45896854E-07	0.41167368E-07
95	0.48697504E-10	0.13497800E-06	0.10816240E-06	0.87966214E-07	0.67900999E-07	0.47427534E-07
99	0.66222321E-06	0.31061472E-06	0.23853221E-06	0.20138287E-06	0.19189895E-06	0.63146729E-07
99.5	0.10830563E-05	0.42724957E-06	0.35048373E-06	0.30563172E-06	0.21208240E-06	0.63680375E-07
99.9	0.21694095E-05	0.14475672E-05	0.93568065E-06	0.86835882E-06	0.46961736E-06	0.64146434E-07
100	0.20840351E-04	0.26050766E-05	0.13025383E-05	0.13460503E-05	0.47182988E-06	0.64146434E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=10000.0 Y=22.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.14361038E-11	0.71856725E-08
50	0.0	0.0	0.0	0.0	0.40441996E-11	0.96102717E-08
75	0.0	0.0	0.40697401E-09	0.68615691E-08	0.14953013E-07	0.14444218E-07
90	0.0	0.14685845E-07	0.40945434E-07	0.39981616E-07	0.25217113E-07	0.23196083E-07
95	0.0	0.81890562E-07	0.67836766E-07	0.60708089E-07	0.48405600E-07	0.28668552E-07
99	0.38537263E-06	0.21330425E-06	0.17496620E-06	0.13972345E-06	0.71506918E-07	0.32824058E-07
99.5	0.75938635E-06	0.32788751E-06	0.22722116E-06	0.16397127E-06	0.89498371E-07	0.33046216E-07
99.9	0.15570795E-05	0.49191385E-06	0.29049704E-06	0.40790667E-06	0.15543850E-06	0.33252277E-07
100	0.97897600E-05	0.12237197E-05	0.61186000E-06	0.46631561E-06	0.15543850E-06	0.33908496E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=10000.0 Y=45.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.15478715E-12	0.75894029E-08
50	0.0	0.0	0.0	0.0	0.30787499E-08	0.10594668E-07
75	0.0	0.0	0.17251980E-09	0.45243382E-08	0.18642215E-07	0.18259620E-07
90	0.0	0.10589190E-07	0.49752124E-07	0.47352728E-07	0.39567627E-07	0.32490437E-07
95	0.0	0.10258418E-06	0.87888225E-07	0.79934239E-07	0.54511247E-07	0.35676820E-07
99	0.41110115E-06	0.28796683E-06	0.22915549E-06	0.18589975E-06	0.16457756E-06	0.48509975E-07
99.5	0.93040444E-06	0.42667341E-06	0.29673453E-06	0.25408576E-06	0.17955091E-06	0.59185670E-07
99.9	0.20388570E-05	0.75091657E-06	0.73016986E-06	0.49865224E-06	0.19861722E-06	0.62496042E-07
100	0.11967654E-04	0.14967463E-05	0.74837357E-06	0.49891571E-06	0.21501700E-06	0.62915490E-07

TABLE 2.3-41

CUMULATIVE FREQUENCY DISTRIBUTION AT X=10000.0 Y=67.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.48820513E-12	0.53997553E-08
50	0.0	0.0	0.0	0.0	0.21488484E-08	0.90656442E-08
75	0.0	0.0	0.19064900E-09	0.42332360E-08	0.16774479E-07	0.14740483E-07
90	0.0	0.88868433E-08	0.35620776E-07	0.38310301E-07	0.30575201E-07	0.19366762E-07
95	0.0	0.71241516E-07	0.74582317E-07	0.61896685E-07	0.48626674E-07	0.24230758E-07
99	0.29504389E-06	0.22101483E-06	0.16693326E-06	0.13171086E-06	0.73985177E-07	0.47917510E-07
99.5	0.72123976E-06	0.32533364E-06	0.22253283E-06	0.17043118E-06	0.80711118E-07	0.48377270E-07
99.9	0.17442262E-05	0.54720454E-06	0.29351366E-06	0.19567574E-06	0.10984843E-06	0.48791776E-07
100	0.50527960E-05	0.10105587E-05	0.38867660E-06	0.24060932E-06	0.13656671E-06	0.49360189E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=10000.0 Y=90.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.91895269E-09	0.89162064E-08
50	0.0	0.0	0.15489381E-11	0.33227221E-09	0.85072607E-08	0.14988771E-07
75	0.0	0.14541743E-09	0.10274171E-07	0.19407693E-07	0.24531005E-07	0.23121000E-07
90	0.0	0.49379487E-07	0.55943179E-07	0.53257811E-07	0.46220329E-07	0.35295201E-07
95	0.44333115E-09	0.11061496E-06	0.93854453E-07	0.86122611E-07	0.65709855E-07	0.42358931E-07
99	0.60299112E-06	0.31099250E-06	0.22322087E-06	0.20123827E-06	0.13368032E-06	0.69289285E-07
99.5	0.95648102E-06	0.41436692E-06	0.34033985E-06	0.30379016E-06	0.17566379E-06	0.71729801E-07
99.9	0.23944494E-05	0.80091729E-06	0.71727061E-06	0.51419346E-06	0.21365605E-06	0.74848742E-07
100	0.12227629E-04	0.15284531E-05	0.77249723E-06	0.51499813E-06	0.28325258E-06	0.77379980E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=10000.0 Y=112.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.20839219E-10	0.84166718E-09	0.14151624E-07	0.29900782E-07
50	0.0	0.27501934E-09	0.11402463E-07	0.25803622E-07	0.47297128E-07	0.60711443E-07
75	0.16949690E-12	0.60673813E-07	0.94820109E-07	0.10325118E-06	0.10866609E-06	0.11630510E-06
90	0.40460890E-07	0.26155215E-06	0.23049108E-06	0.21451825E-06	0.18499907E-06	0.16472660E-06
95	0.45990720E-06	0.41155283E-06	0.34330810E-06	0.30469738E-06	0.25186006E-06	0.18124632E-06
99	0.18238316E-05	0.83206919E-06	0.70830458E-06	0.56533167E-06	0.40792537E-06	0.19859812E-06
99.5	0.23300199E-05	0.10834447E-05	0.81554646E-06	0.70112458E-06	0.43298786E-06	0.20610111E-06
99.9	0.29353081E-05	0.15642336E-05	0.10322783E-05	0.92796711E-06	0.53611660E-06	0.20792345E-06
100	0.73103029E-05	0.27324531E-05	0.15486767E-05	0.10390831E-05	0.58725243E-06	0.20836592E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=10000.0 Y=135.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.30513836E-09	0.33975152E-07	0.60142440E-07	0.97421378E-07	0.14158559E-06
50	0.81213882E-12	0.10084892E-06	0.14449859E-06	0.15982766E-06	0.19696233E-06	0.22543361E-06
75	0.16013809E-06	0.32541402E-06	0.32830889E-06	0.32482870E-06	0.34691823E-06	0.35846648E-06
90	0.76552914E-06	0.66848929E-06	0.60408695E-06	0.57567422E-06	0.53463327E-06	0.45199931E-06
95	0.12743885E-05	0.99710542E-06	0.89382365E-06	0.83963278E-06	0.65709690E-06	0.48823017E-06
99	0.32882863E-05	0.21611031E-05	0.16481881E-05	0.14180096E-05	0.11066504E-05	0.58373649E-06
99.5	0.49012660E-05	0.25623904E-05	0.20536236E-05	0.16799531E-05	0.13855224E-05	0.61466212E-06
99.9	0.90633584E-05	0.39299375E-05	0.32929020E-05	0.30402252E-05	0.20607076E-05	0.64537051E-06
100	0.21114320E-04	0.53340282E-05	0.45244979E-05	0.35760759E-05	0.22838203E-05	0.69248154E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=15000.0 Y=315.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.65581034E-08	0.22620974E-07
50	0.0	0.0	0.55711591E-09	0.87253866E-08	0.28744893E-07	0.47982880E-07
75	0.0	0.28775133E-07	0.54719095E-07	0.61016749E-07	0.68279519E-07	0.77431821E-07
90	0.98195940E-08	0.16965191E-06	0.16625148E-06	0.15875236E-06	0.14214334E-06	0.11057045E-06
95	0.28281733E-06	0.30325634E-06	0.27776838E-06	0.24559421E-06	0.19655960E-06	0.12945674E-06
99	0.11925194E-05	0.69576959E-06	0.51861451E-06	0.47041817E-06	0.37303488E-06	0.15222821E-06
99.5	0.17336597E-05	0.83605255E-06	0.63974949E-06	0.57241328E-06	0.43054570E-06	0.15427327E-06
99.9	0.41135654E-05	0.13727631E-05	0.10024742E-05	0.20443877E-05	0.78021060E-06	0.15839174E-06
100	0.30345429E-04	0.64752967E-05	0.32388989E-05	0.21875321E-05	0.86791459E-06	0.16016929E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=15000.0 Y=337.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
250.0	0.0	0.0	0.0	0.0	.19011699E-08	0.16883281E-07
50	0.0	0.0	0.66232393E-11	0.20247073E-08	0.13712853E-07	0.25553373E-07
75	0.0	0.31021297E-08	0.26537951E-07	0.30617500E-07	0.38804675E-07	0.37484096E-07
90	0.17074051E-10	0.84400710E-07	0.87690921E-07	0.88357922E-07	0.76232027E-07	0.48131973E-07
95	0.55542273E-07	0.16197566E-06	0.15089341E-06	0.13481110E-06	0.10000292E-06	0.55359607E-07
99	0.77743528E-06	0.38396513E-06	0.27873079E-06	0.23427765E-06	0.14466826E-06	0.65524091E-07
99.5	0.11094689E-05	0.51570566E-06	0.34316957E-06	0.26858902E-06	0.16511478E-06	0.69627163E-07
99.9	0.21586957E-05	0.78843152E-06	0.43353373E-06	0.30150534E-06	0.17615287E-06	0.71047509E-07
100	0.50085073E-05	0.90445928E-06	0.45224823E-06	0.37201784E-06	0.18749381E-06	0.71783518E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=15000.0 Y=0.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.42491158E-11	0.68898416E-08
50	0.0	0.0	0.0	0.50700438E-12	0.60802101E-08	0.10994714E-07
75	0.0	0.0	0.29931513E-08	0.10471879E-07	0.15913940E-07	0.15212056E-07
90	0.0	0.25995199E-07	0.39465355E-07	0.37539614E-07	0.26602049E-07	0.23917156E-07
95	0.84520620E-11	0.78655887E-07	0.64125516E-07	0.50901349E-07	0.38762082E-07	0.27592829E-07
99	0.39688416E-06	0.18071910E-06	0.13130398E-06	0.12209216E-06	0.10809686E-06	0.39930477E-07
99.5	0.63648679E-06	0.24657982E-06	0.21058162E-06	0.17981279E-06	0.13211240E-06	0.40202647E-07
99.9	0.13129211E-05	0.89727888E-06	0.59065110E-06	0.57528712E-06	0.30167593E-06	0.40480511E-07
100	0.13806886E-04	0.17258608E-05	0.86293073E-06	0.87396558E-06	0.30306848E-06	0.40480511E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=15000.0 Y=22.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.33338339E-12	0.39631480E-08
50	0.0	0.0	0.0	0.0	0.18983932E-08	0.53384994E-08
75	0.0	0.0	0.13741124E-09	0.32824041E-08	0.85651628E-08	0.81658236E-08
90	0.0	0.67215353E-08	0.22862757E-07	0.21644198E-07	0.19387659E-07	0.13477994E-07
95	0.0	0.47744486E-07	0.40322814E-07	0.35388123E-07	0.26921331E-07	0.16593361E-07
99	0.20850109E-06	0.12234443E-06	0.10114360E-06	0.81633004E-07	0.41390166E-07	0.19006450E-07
99.5	0.43771365E-06	0.19947140E-06	0.13024169E-06	0.96822475E-07	0.51571217E-07	0.19195689E-07
99.9	0.91841656E-06	0.30954106E-06	0.16814755E-06	0.25093811E-06	0.95084317E-07	0.19331665E-07
100	0.60225157E-05	0.75281446E-06	0.37640723E-06	0.28525307E-06	0.95084317E-07	0.19398097E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=15000.0 Y=45.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.16775125E-13	0.43620254E-08
50	0.0	0.0	0.0	0.0	0.15074273E-08	0.62900263E-08
75	0.0	0.0	0.50292701E-10	0.18487309E-08	0.10394373E-07	0.10547602E-07
90	0.0	0.46893938E-08	0.29167751E-07	0.26986175E-07	0.22845800E-07	0.19048986E-07
95	0.0	0.59042485E-07	0.54270807E-07	0.46654009E-07	0.32093070E-07	0.21249946E-07
99	0.23026769E-06	0.16328897E-06	0.13279731E-06	0.13307749E-06	0.98863723E-07	0.29389479E-07
99.5	0.55310579E-06	0.24180156E-06	0.19961624E-06	0.15243188E-06	0.10709704E-06	0.35758887E-07
99.9	0.11425655E-05	0.44322462E-06	0.43867260E-06	0.31491277E-06	0.12175013E-06	0.37773642E-07
100	0.75579073E-05	0.94493657E-06	0.47246829E-06	0.31497882E-06	0.13047810E-06	0.38027157E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=15000.0 Y=67.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.84983778E-13	0.28847238E-08
50	0.0	0.0	0.0	0.0	0.10945815E-08	0.51356537E-08
75	0.0	0.0	0.51957716E-10	0.18548296E-08	0.95388017E-08	0.86745793E-08
90	0.0	0.39902375E-08	0.20557884E-07	0.22688319E-07	0.18057346E-07	0.11139001E-07
95	0.0	0.41215444E-07	0.40739160E-07	0.35761879E-07	0.28582093E-07	0.14339826E-07
99	0.15917016E-06	0.13336694E-06	0.99734848E-07	0.77640095E-07	0.45095966E-07	0.28247744E-07
99.5	0.42300820E-06	0.19946970E-06	0.12478461E-06	0.10174961E-06	0.48676057E-07	0.28512972E-07
99.9	0.10532685E-05	0.31382297E-06	0.18011775E-06	0.12007848E-06	0.62027539E-07	0.28752943E-07
100	0.31094064E-05	0.62168124E-06	0.23910815E-06	0.14801935E-06	0.76844231E-07	0.29126156E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=15000.0 Y=90.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.35814640E-09	0.50313389E-08
50	0.0	0.0	0.26058447E-12	0.10601167E-09	0.45774478E-08	0.82628588E-08
75	0.0	0.35607323E-10	0.48928221E-08	0.10372965E-07	0.13553148E-07	0.13117738E-07
90	0.0	0.27021031E-07	0.31015087E-07	0.31375979E-07	0.26601228E-07	0.20073045E-07
95	0.98749744E-10	0.61981723E-07	0.54284722E-07	0.49494322E-07	0.39624183E-07	0.24721672E-07
99	0.34081376E-06	0.17759987E-06	0.13384806E-06	0.12186308E-06	0.83738769E-07	0.40617124E-07
99.5	0.56139402E-06	0.26591306E-06	0.19706977E-06	0.18206487E-06	0.10600883E-06	0.42088498E-07
99.9	0.13590184E-05	0.48480365E-06	0.43750231E-06	0.31885628E-06	0.13196990E-06	0.43901814E-07
100	0.76525512E-05	0.95656833E-06	0.48143238E-06	0.32095488E-06	0.17561098E-06	0.45360050E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=15000.0 Y=112.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.43523128E-11	0.30470293E-09	0.75992297E-08	0.16716541E-07
50	0.0	0.77833850E-10	0.54824341E-08	0.13232260E-07	0.27178555E-07	0.34113487E-07
75	0.0	0.32367581E-07	0.53171512E-07	0.58579150E-07	0.60095658E-07	0.64531093E-07
90	0.15921973E-07	0.14831653E-06	0.12985015E-06	0.12489033E-06	0.10569761E-06	0.94182724E-07
95	0.24518067E-06	0.23630218E-06	0.19737212E-06	0.16936474E-06	0.14695985E-06	0.10216166E-06
99	0.10542435E-05	0.48661377E-06	0.40624525E-06	0.32735522E-06	0.23301607E-06	0.11407087E-06
99.5	0.13866784E-05	0.62256959E-06	0.47548781E-06	0.39148244E-06	0.25524099E-06	0.11504034E-06
99.9	0.23214416E-05	0.91088100E-06	0.58397745E-06	0.54756902E-06	0.30177517E-06	0.11607568E-06
100	0.44572580E-05	0.16203585E-05	0.91777423E-06	0.61470507E-06	0.32979307E-06	0.11660666E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=15000.0 Y=135.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.87701235E-10	0.17875784E-07	0.32972068E-07	0.54499544E-07	0.83042266E-07
50	0.0	0.54354430E-07	0.80337145E-07	0.89459093E-07	0.11152611E-06	0.13161014E-06
75	0.76664151E-07	0.18519063E-06	0.18656362E-06	0.18567346E-06	0.19885351E-06	0.20508207E-06
90	0.43765573E-06	0.38521569E-06	0.35380822E-06	0.33945889E-06	0.31262346E-06	0.26365655E-06
95	0.74108164E-06	0.58376895E-06	0.53680361E-06	0.49867924E-06	0.38944358E-06	0.28592308E-06
99	0.20399293E-05	0.12813935E-05	0.98928103E-06	0.86699043E-06	0.64731728E-06	0.34321846E-06
99.5	0.28955837E-05	0.15893529E-05	0.12666242E-05	0.10356380E-05	0.83330207E-06	0.35980611E-06
99.9	0.57216357E-05	0.24207111E-05	0.20257066E-05	0.18715227E-05	0.12421297E-05	0.37859621E-06
100	0.13350488E-04	0.32579665E-05	0.27657179E-05	0.21882661E-05	0.13833542E-05	0.40881116E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=30000.0 Y=315.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.9	0.0	0.25466331E-08	0.95368655E-08
50	0.0	0.0	0.78097778E-10	0.29290086E-08	0.12006758E-07	0.21992719E-07
75	0.0	0.96733572E-08	0.23271738E-07	0.26817059E-07	0.29294508E-07	0.33200383E-07
90	0.15804347E-08	0.75784897E-07	0.72687101E-07	0.69492899E-07	0.62654692E-07	0.50567294E-07
95	0.11228110E-06	0.13580825E-06	0.12496923E-06	0.11077958E-06	0.87188369E-07	0.57074697E-07
99	0.54333287E-06	0.31370473E-06	0.23337026E-06	0.20854372E-06	0.16181275E-06	0.73323690E-07
99.5	0.78534606E-06	0.38627832E-06	0.29357693E-06	0.25960179E-06	0.18710909E-06	0.74192769E-07
99.9	0.18420833E-05	0.59907313E-06	0.44757218E-06	0.10596832E-05	0.39342717E-06	0.77013965E-07
100	0.16618098E-04	0.33369779E-05	0.16687000E-05	0.11220336E-05	0.43589006E-06	0.77496850E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=30000.0 Y=337.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.57549632E-09	0.73884365E-08
50	0.0	0.0	0.36937684E-12	0.42537796E-09	0.53889941E-08	0.10797148E-07
75	0.0	0.60633898E-09	0.98081117E-08	0.13246350E-07	0.16726691E-07	0.15961593E-07
90	0.53020040E-12	0.36850878E-07	0.36645520E-07	0.38873999E-07	0.32473420E-07	0.19701353E-07
95	0.14082605E-07	0.69344082E-07	0.68281850E-07	0.57674050E-07	0.43507271E-07	0.22800123E-07
99	0.35425074E-06	0.16567549E-06	0.12153515E-06	0.10311425E-06	0.63890070E-07	0.31202003E-07
99.5	0.49062709E-06	0.21010817E-06	0.15162880E-06	0.11906582E-06	0.68105351E-07	0.32523928E-07
99.9	0.93845620E-06	0.35708922E-06	0.20588476E-06	0.15175669E-06	0.90669801E-07	0.33244039E-07
100	0.23853527E-05	0.45305342E-06	0.22763510E-06	0.16396825E-06	0.99481440E-07	0.33542680E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=30000.0 Y=0.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.29609935E-12	0.28636971E-08
50	0.0	0.0	0.0	0.11113838E-13	0.22228508E-08	0.47122803E-08
75	0.0	0.0	0.73223183E-09	0.38157530E-08	0.69859922E-08	0.66229759E-07
90	0.0	0.97641788E-08	0.17463666E-07	0.17415971E-07	0.11571441E-07	0.10530172E-07
95	0.15624204E-12	0.34692391E-07	0.29171744E-07	0.23136387E-07	0.16517987E-07	0.12725344E-07
99	0.16071755E-06	0.76794834E-07	0.58218461E-07	0.51091696E-07	0.47019551E-07	0.18705244E-07
99.5	0.29896370E-06	0.10925226E-06	0.93230540E-07	0.79488757E-07	0.63624952E-07	0.18805757E-07
99.9	0.58652580E-06	0.42542558E-06	0.28551841E-06	0.28332755E-06	0.14418623E-06	0.18922940E-07
100	0.67998617E-05	0.84998271E-06	0.42499136E-06	0.42281243E-06	0.14469254E-06	0.18922940E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=30000.0 Y=22.5 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.10799511E-13	0.15164612E-08
50	0.0	0.0	0.0	0.0	0.61131900E-09	0.21830511E-08
75	0.0	0.0	0.16979002E-10	0.87244656E-09	0.35564787E-08	0.36691099E-08
90	0.0	0.17925390E-08	0.95135775E-08	0.92867367E-08	0.84610576E-08	0.63178405E-08
95	0.0	0.19747674E-07	0.18311582E-07	0.16318879E-07	0.12306064E-07	0.74280209E-08
99	0.73414014E-07	0.58875209E-07	0.44931362E-07	0.37825529E-07	0.19342966E-07	0.87324601E-08
99.5	0.19305151E-06	0.85239321E-07	0.57500131E-07	0.45481329E-07	0.23543205E-07	0.88065910E-08
99.9	0.40763098E-06	0.14512125E-07	0.78058292E-07	0.11679180E-06	0.43894588E-07	0.88819903E-08
100	0.28030036E-05	0.35037544E-06	0.17518772E-06	0.13168375E-06	0.43894588E-07	0.89204555E-08

CUMULATIVE FREQUENCY DISTRIBUTION AT X=30000.0 Y=45.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.0	0.18662873E-08
50	0.0	0.0	0.0	0.0	0.50042726E-09	0.26730709E-08
75	0.0	0.0	0.39705583E-11	0.52659033E-09	0.46017661E-08	0.47644235E-08
90	0.0	0.10701231E-08	0.11990824E-07	0.11826288E-07	0.10011917E-07	0.81641041E-08
95	0.0	0.24349113E-07	0.24542345E-07	0.20046127E-07	0.13839816E-07	0.92468966E-08
99	0.92524488E-07	0.69798716E-07	0.60351681E-07	0.59177616E-07	0.43500155E-07	0.13539605E-07
99.5	0.23834008E-06	0.11319975E-06	0.89329035E-07	0.70118176E-07	0.51050378E-07	0.16306434E-07
99.9	0.53826830E-06	0.28328691E-06	0.19303008E-06	0.15226681E-06	0.54461015E-07	0.17222273E-07
100	0.36544034E-05	0.45681497E-06	0.22840749E-06	0.15227164E-06	0.61999003E-07	0.17337861E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=30000.0 Y=67.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.22644992E-14	0.11720056E-08
50	0.0	0.0	0.0	0.0	0.31417380E-09	0.22692874E-08
75	0.0	0.0	0.52800845E-11	0.48673421E-09	0.40366999E-08	0.38441001E-08
90	0.0	0.88995789E-09	0.85814769E-08	0.10402204E-07	0.83399527E-08	0.47788618E-08
95	0.0	0.17683718E-07	0.17903890E-07	0.14717511E-07	0.11889053E-07	0.60918310E-08
99	0.54966797E-07	0.61008564E-07	0.43119574E-07	0.35202913E-07	0.19701574E-07	0.12019374E-07
99.5	0.18337704E-06	0.85349200E-07	0.53488929E-07	0.43651355E-07	0.22257193E-07	0.12152224E-07
99.9	0.49097372E-06	0.14134537E-06	0.80181849E-07	0.53454563E-07	0.25143208E-07	0.12246886E-07
100	0.14467178E-05	0.28934352E-06	0.11128594E-06	0.68891268E-07	0.31165513E-07	0.12445035E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=30000.0 Y=90.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.65864397E-10	0.20825124E-08
50	0.0	0.0	0.92840334E-14	0.11592745E-10	0.16247081E-08	0.35264047E-08
75	0.0	0.25062209E-11	0.12690611E-08	0.36838277E-08	0.57753837E-08	0.57722005E-08
90	0.0	0.98424167E-08	0.14377200E-07	0.13429144E-07	0.12304604E-07	0.88840082E-08
95	0.59097033E-11	0.28230950E-07	0.25187923E-07	0.21970269E-07	0.19115102E-07	0.10852848E-07
99	0.13526767E-06	0.80749828E-07	0.62610525E-07	0.57959785E-07	0.40544993E-07	0.17481465E-07
99.5	0.25345207E-06	0.11674047E-06	0.94287941E-07	0.81522671E-07	0.47361077E-07	0.17962108E-07
99.9	0.62218726E-06	0.22123038E-06	0.19813035E-06	0.14556815E-06	0.59094461E-07	0.18709208E-07
100	0.34936356E-05	0.43670445E-06	0.21909176E-06	0.14606115E-06	0.78980747E-07	0.19330866E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=30000.0 Y=112.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.23345361E-12	0.47088083E-10	0.28057887E-08	0.67437043E-08
50	0.0	0.62331269E-11	0.15318589E-08	0.47521667E-08	0.10813505E-07	0.14239223E-07
75	0.0	0.11210062E-07	0.22545485E-07	0.23727203E-07	0.25251463E-07	0.28560905E-07
90	0.30338845E-08	0.61439380E-07	0.58503737E-07	0.54185911E-07	0.46454740E-07	0.40374253E-07
95	0.90963283E-07	0.10561797E-06	0.86726743E-07	0.76200081E-07	0.62735467E-07	0.44272010E-07
99	0.47112485E-06	0.22302675E-06	0.16909689E-06	0.14090898E-06	0.99676811E-07	0.50122829E-07
99.5	0.65351367E-06	0.27533167E-06	0.20922118E-06	0.17070170E-06	0.11543676E-06	0.51748586E-07
99.9	0.10784406E-05	0.41844237E-06	0.29186162E-06	0.24557761E-06	0.13325729E-06	0.52903065E-07
100	0.20587149E-05	0.70244937E-06	0.40353405E-06	0.26973731E-06	0.14793858E-06	0.53056681E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=30000.0 Y=135.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.69041387E-11	0.64947194E-08	0.12591528E-07	0.23224793E-07	0.36272571E-07
50	0.0	0.20490724E-07	0.33088554E-07	0.36662065E-07	0.46559546E-07	0.58065218E-07
75	0.22570859E-07	0.77671871E-07	0.77909647E-07	0.78995015E-07	0.85990962E-07	0.85851411E-07
90	0.18494472E-06	0.16740989E-06	0.15365777E-06	0.14811320E-06	0.13546395E-06	0.11276023E-06
95	0.33302257E-06	0.26129362E-06	0.23807456E-06	0.22218791E-06	0.16813664E-06	0.12399533E-06
99	0.94607157E-06	0.56286763E-06	0.45282661E-06	0.38698556E-06	0.27706278E-06	0.14644928E-06
99.5	0.13263107E-05	0.73711476E-06	0.56172013E-06	0.48058166E-06	0.37332501E-06	0.15398774E-06
99.9	0.27695505E-05	0.11265029E-05	0.93776180E-06	0.85681131E-06	0.55677998E-06	0.16290767E-06
100	0.65669201E-05	0.14959496E-05	0.12713581E-05	0.10003M147E-05	0.62457548E-06	0.17522251E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=40000.0 Y=315.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.16415163E-08	0.70380430E-08
50	0.0	0.0	0.29444877E-10	0.19349189E-08	0.84524601E-08	0.16231006E-07
75	0.0	0.60881220E-08	0.16645327E-07	0.19264924E-07	0.20944906E-07	0.24019435E-07
90	0.71701356E-09	0.53886090E-07	0.53000878E-07	0.51506767E-07	0.45976762E-07	0.37226414E-07
95	0.75505227E-07	0.99917543E-07	0.91774098E-07	0.81905512E-07	0.63249729E-07	0.41935728E-07
99	0.41088538E-06	0.22936251E-06	0.17727916E-06	0.15149800E-06	0.11761983E-06	0.55074278E-07
99.5	0.58249861E-06	0.28886063E-06	0.21878833E-06	0.18809868E-06	0.13698002E-06	0.55732723E-07
99.9	0.13457156E-05	0.43757666E-06	0.32631584E-06	0.82002043E-06	0.30040792E-06	0.58093370E-07
100	0.12992401E-04	0.25675909E-05	0.12838909E-05	0.86207729E-06	0.33248313E-06	0.58379729E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=40000.0 Y=337.5 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.35253844E-09	0.53102625E-08
50	0.0	0.0	0.67349663E-13	0.23781399E-09	0.38566021E-08	0.76761992E-08
75	0.0	0.29737235E-09	0.68120087E-08	0.97082982E-08	0.12000356E-07	0.11549659E-07
90	0.0	0.26685257E-07	0.26478013E-07	0.29063639E-07	0.24013083E-07	0.13988611E-07
95	0.76828286E-08	0.51009202E-07	0.48923273E-07	0.42713317E-07	0.31614430E-07	0.15853630E-07
99	0.25945445E-06	0.12263331E-06	0.88253955E-07	0.72231558E-07	0.46019963E-07	0.23189518E-07
99.5	0.36299582E-06	0.15003980E-06	0.10998303E-06	0.91423658E-07	0.50830963E-07	0.24025013E-07
99.9	0.69772511E-06	0.27427109E-06	0.15174987E-06	0.11625650E-06	0.69469593E-07	0.24556485E-07
100	0.17996481E-05	0.34739230E-06	0.17438481E-06	0.12516523E-06	0.76494473E-07	0.24756925E-07

TABLE 2.3-41

CUMULATIVE FREQUENCY DISTRIBUTION AT X=40000.0 Y=0.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.69631855E-13	0.20553141E-08
50	0.0	0.0	0.0	0.0	0.14375043E-08	0.34248908E-08
75	0.0	0.0	0.38991144E-09	0.25732849E-08	0.51838285E-08	0.49844573E-08
90	0.0	0.63576451E-08	0.12781506E-07	0.12009970E-07	0.85428482E-08	0.77559221E-08
95	0.0	0.24914229E-07	0.21519909E-07	0.17115667E-07	0.11925533E-07	0.95558512E-08
99	0.11585513E-06	0.58398037E-07	0.43353321E-07	0.38138676E-07	0.35706218E-07	0.13780717E-07
99.5	0.22897450E-06	0.80479367E-07	0.68184363E-07	0.57399269E-07	0.48447543E-07	0.13859413E-07
99.9	0.43573579E-06	0.31790933E-06	0.21758177E-06	0.21185264E-06	0.10697067E-06	0.13931686E-07
100	0.50844637E-05	0.63555797E-06	0.31777898E-06	0.31517305E-06	0.10728007E-06	0.13931686E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=40000.0 Y=22.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.22825323E-14	0.10253214E-08
50	0.0	0.0	0.0	0.0	0.37899350E-09	0.15423209E-08
75	0.0	0.0	0.60948772E-11	0.50212967E-09	0.25850413E-08	0.26817426E-08
90	0.0	0.94948893E-09	0.64822387E-08	0.66115007E-08	0.59713088E-08	0.46137707E-08
95	0.0	0.13425790E-07	0.13770709E-07	0.11709076E-07	0.89902592E-08	0.53256137E-08
99	0.47268532E-07	0.42771248E-07	0.31585852E-07	0.26871376E-07	0.14729572E-07	0.65359025E-08
99.5	0.14044599E-06	0.62980973E-07	0.42434817E-07	0.32666609E-07	0.18163774E-07	0.65801409E-08
99.9	0.32420206E-06	0.10505386E-06	0.62342622E-07	0.86487489E-07	0.32380285E-07	0.66406116E-08
100	0.20757006E-05	0.25946258E-06	0.12973129E-06	0.97140855E-07	0.32380285E-07	0.66714563E-08

CUMULATIVE FREQUENCY DISTRIBUTION AT X=40000.0 Y=45.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.0	0.13420420E-08
50	0.0	0.0	0.0	0.0	0.30277159E-09	0.19575030E-08
75	0.0	0.0	0.98163491E-12	0.26609581E-09	0.33844989E-08	0.35684173E-08
90	0.0	0.54799099E-09	0.85229992E-08	0.87782972E-08	0.71423401E-08	0.58347283E-08
95	0.0	0.17286126E-07	0.17707460E-07	0.14583417E-07	0.11001159E-07	0.67578902E-08
99	0.60002037E-07	0.51943729E-07	0.43389754E-07	0.41945697E-07	0.31346310E-07	0.10047788E-07
99.5	0.16618321E-06	0.83851432E-07	0.64742324E-07	0.49712447E-07	0.37315683E-07	0.12012599E-07
99.9	0.38219673E-06	0.23010818E-06	0.13909863E-06	0.11604084E-06	0.39791168E-07	0.12678129E-07
100	0.27849810E-05	0.34812706E-06	0.17406353E-06	0.11604232E-06	0.46856055E-07	0.12763220E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=40000.0 Y=67.5 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.0	0.86270013E-09
50	0.0	0.0	0.0	0.0	0.18723927E-09	0.16437189E-08
75	0.0	0.0	0.19616964E-11	0.29583536E-09	0.29029823E-08	0.27648739E-08
90	0.0	0.45677395E-09	0.62707031E-08	0.75733055E-08	0.63489232E-08	0.35549321E-08
95	0.0	0.12763330E-07	0.13008794E-07	0.11139939E-07	0.89749221E-08	0.43838959E-08
99	0.33933272E-07	0.47509751E-07	0.33027465E-07	0.24448063E-07	0.15027211E-07	0.84558280E-08
99.5	0.13824439E-06	0.66054895E-07	0.40431871E-07	0.30825113E-07	0.16482073E-07	0.85466070E-08
99.9	0.38007801E-06	0.10414516E-06	0.57283035E-07	0.38188688E-07	0.17925203E-07	0.86144318E-08
100	0.10713347E-05	0.21426695E-06	0.82410338E-07	0.51015938E-07	0.21639782E-07	0.87671630E-08

CUMULATIVE FREQUENCY DISTRIBUTION AT X=40000.0 Y=90.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.	0	0.36115860E-10	0.15164872E-08
50	0.0	0.0	0.0	0.41449916E-11	0.10705856E-08	0.26077296E-08
75	0.0	0.82237873E-12	0.68779538E-09	0.23202409E-08	0.42293387E-08	0.43136446E-08
90	0.0	0.65096906E-08	0.10405316E-07	0.10173373E-07	0.89387164E-08	0.67975172E-08
95	0.15404657E-11	0.20744508E-07	0.19049246E-07	0.16678687E-07	0.13891455E-07	0.80735063E-08
99	0.93648168E-07	0.59597937E-07	0.5060003M3E-07	0.41623515E-07	0.30180381E-07	0.12550590E-07
99.5	0.18856923E-06	0.83812040E-07	0.69579073E-07	0.59586302E-07	0.34338292E-07	0.12818862E-07
99.9	0.44919136E-06	0.16302067E-06	0.14433573E-06	0.10652877E-06	0.42909726E-07	0.13341559E-07
100	0.25566906E-05	0.31958632E-06	0.16018430E-06	0.10678951E-06	0.57358410E-07	0.13782877E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=40000.0 Y=112.5 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.44786206E-13	0.20067781E-10	0.18704158E-08	0.47645052E-08
50	0.0	0.20901163E-11	0.88423890E-09	0.31391185E-08	0.74166522E-08	0.10151794E-07
75	0.0	0.72699855E-08	0.16055758E-07	0.16961692E-07	0.18201654E-07	0.21012170E-07
90	0.13907424E-08	0.44147285E-07	0.42939924E-07	0.39192841E-07	0.34247254E-07	0.29138306E-07
95	0.62572951E-07	0.77944890E-07	0.62158051E-07	0.56820568E-07	0.46285223E-07	0.32769321E-07
99	0.35544366E-06	0.16516822E-06	0.12869509E-06	0.10627093E-06	0.72404021E-07	0.37315800E-07
99.5	0.48561060E-06	0.20236121E-06	0.15044947E-06	0.12279213E-06	0.86895113E-07	0.39183174E-07
99.9	0.79982300E-06	0.30243359E-06	0.23379363E-06	0.18339102E-06	0.10336390E-06	0.40233235E-07
100	0.15243104E-05	0.50295489E-06	0.29130939E-06	0.19718789E-06	0.11713684E-06	0.40354642E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=40000.0 Y=135.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.17996013E-11	0.42544315E-08	0.87951726E-08	0.16500621E-07	0.26431923E-07
50	0.0	0.13923973E-07	0.23451772E-07	0.26243036E-07	0.33586602E-07	0.42776964E-07
75	0.12926265E-07	0.55918868E-07	0.56889608E-07	0.57493668E-07	0.62766333E-07	0.61176479E-07
90	0.13196137E-06	0.12335147E-06	0.11182908E-06	0.11028703E-06	0.97954683E-07	0.81658641E-07
95	0.24321042E-06	0.19214144E-06	0.17277318E-06	0.16163261E-06	0.12189474E-06	0.89847731E-07
99	0.69058160E-06	0.40855002E-06	0.32181708E-06	0.27994167E-06	0.20150475E-06	0.10477578E-06
99.5	0.10288722E-05	0.54148290E-06	0.39110370E-06	0.34589146E-06	0.27409010E-06	0.11023269E-06
99.9	0.19892714E-05	0.83666066E-06	0.69441830E-06	0.63590898E-06	0.40677452E-06	0.11642396E-06
100	0.49683067E-05	0.11077846E-05	0.94137056E-06	0.74459365E-06	0.45793308E-06	0.12506968E-06

CUMULATIVE FREQUENCY DISTRIBUTION AT X=50000.0 Y=315.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.11853969E-08	0.55911684E-08
50	0.0	0.0	0.15915338E-10	0.13169581E-08	0.67191372E-08	0.13015494E-07
75	0.0	0.43038213E-08	0.12962797E-07	0.15104355E-07	0.16195525E-07	0.18793767E-07
90	0.36273518E-09	0.42316998E-07	0.42017792E-07	0.40966444E-07	0.36742584E-07	0.29288746E-07
95	0.54992793E-07	0.78129574E-07	0.72207968E-07	0.64581457E-07	0.49703644E-07	0.32882163E-07
99	0.33280520E-06	0.18264325E-06	0.14367447E-06	0.12006387E-06	0.90980166E-07	0.44202082E-07
99.5	0.48250172E-06	0.23287839E-06	0.17397065E-06	0.15041292E-06	0.10630970E-06	0.44734783E-07
99.9	0.10621479E-05	0.34794130E-06	0.25547547E-06	0.67242036E-06	0.24399679E-06	0.46763812E-07
100	0.10739011E-04	0.20961907E-05	0.10481454E-05	0.70316611E-06	0.26973709E-06	0.46990777E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=50000.0 Y=337.5 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.23571856E-09	0.41648924E-08
50	0.0	0.0	0.19380348E-13	0.14101728E-09	0.30366298E-08	0.60264966E-08
75	0.0	0.17223564E-09	0.51954352E-08	0.76911952E-08	0.91056904E-08	0.90638999E-08
90	0.0	0.21044961E-07	0.20916513E-07	0.22435927E-07	0.18708000E-07	0.11039180E-07
95	0.45941526E-08	0.40792976E-07	0.38187956E-07	0.33429160E-07	0.24837654E-07	0.12232128E-07
99	0.20600692E-06	0.97148813E-07	0.70691158E-07	0.58054486E-07	0.36737926E-07	0.18491644E-07
99.5	0.28410670E-06	0.11614532E-06	0.87081730E-07	0.74278603E-07	0.39955427E-07	0.19061513E-07
99.9	0.55124275E-06	0.22283587E-06	0.11997895E-06	0.94546067E-07	0.56433741E-07	0.19482982E-07
100	0.14447642E-05	0.28267436E-06	0.14182086E-06	0.10229849E-06	0.62306071E-07	0.19628473E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=50000.0 Y=0.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.25092474E-13	0.16122919E-08
50	0.0	0.0	0.0	0.0	0.99717168E-09	0.26935338E-08
75	0.0	0.0	0.22834719E-09	0.18305966E-08	0.41064112E-08	0.39980179E-08
90	0.0	0.46071484E-08	0.10087042E-07	0.96617967E-08	0.68796489E-08	0.61201533E-08
95	0.0	0.20100636E-07	0.16803586E-07	0.14196448E-07	0.98225712E-08	0.76537745E-08
99	0.87091394E-07	0.45476163E-07	0.36103128E-07	0.31477288E-07	0.28805363E-07	0.10817931E-07
99.5	0.17823288E-06	0.65993390E-07	0.54441589E-07	0.45532108E-07	0.39213191E-07	0.10876999E-07
99.9	0.36152659E-06	0.25359185E-06	0.17618959E-06	0.16722169E-06	0.84218016E-07	0.10931323E-07
100	0.40133209E-05	0.50166511E-06	0.25083256E-06	0.24891347E-06	0.84423675E-07	0.10931323E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=50000.0 Y=22.5 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.0	0.78266593E-09
50	0.0	0.0	0.0	0.0	0.27549363E-09	0.12099282E-08
75	0.0	0.0	0.29529669E-11	0.33172798E-09	0.19754711E-08	0.20782787E-08
90	0.0	0.62962968E-09	0.48998245E-08	0.52175260E-08	0.45992010E-08	0.36191163E-08
95	0.0	0.10249266E-07	0.10755432E-07	0.94190682E-08	0.71207644E-08	0.41645407E-08
99	0.36101284E-07	0.35021920E-07	0.26912769E-07	0.21362300E-07	0.12348206E-07	0.52444129E-08
99.5	0.11135671E-06	0.50203244E-07	0.33474695E-07	0.25565598E-07	0.15020778E-07	0.52745222E-08
99.9	0.26841957E-06	0.82788972E-07	0.52334890E-07	0.68378199E-07	0.25531119E-07	0.53263918E-08
100	0.16410777E-05	0.20513471E-06	0.10256736E-06	0.76593324E-07	0.25531119E-07	0.53520317E-08

CUMULATIVE FREQUENCY DISTRIBUTION AT X=50000.0 Y=45.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.0	0.10340857E-08
50	0.0	0.0	0.0	0.0	0.18030499E-09	0.15494794E-08
75	0.0	0.0	0.42945606E-12	0.15061261E-09	0.25657800E-08	0.29090854E-08
90	0.0	0.33783154E-09	0.64839725E-08	0.69931403E-08	0.54554761E-08	0.45059920E-08
95	0.0	0.13322733E-07	0.13732663E-07	0.11582031E-07	0.86609830E-08	0.52724012E-08
99	0.43783377E-07	0.41510184E-07	0.33729030E-07	0.32147092E-07	0.24144676E-07	0.79870048E-08
99.5	0.13243022E-06	0.66692735E-07	0.49707701E-07	0.38130608E-07	0.28835643E-07	0.94942649E-08
99.9	0.30703234E-06	0.19617551E-06	0.10707288E-06	0.93966776E-07	0.31763602E-07	0.10014510E-07
100	0.22552031E-05	0.28190198E-06	0.14095099E-06	0.93967287E-07	0.37749661E-07	0.10081720E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=50000.0 Y=67.5 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.0	0.66957306E-09
50	0.0	0.0	0.0	0.0	0.13316517E-09	0.12845931E-08
75	0.0	0.0	0.85038216E-12	0.16582083E-09	0.22065068E-08	0.21248860E-08
90	0.0	0.25389424E-09	0.50534226E-08	0.57119536E-08	0.50176823E-08	0.28658880E-08
95	0.0	0.10465790E-07	0.10157930E-07	0.91188816E-08	0.68588584E-08	0.34008540E-08
99	0.24164102E-07	0.35183987E-07	0.27026164E-07	0.18310264E-07	0.11509790E-07	0.64450134E-08
99.5	0.11171124E-06	0.54052329E-07	0.32624158E-07	0.23696042E-07	0.13030974E-07	0.65142629E-08
99.9	0.28228010E-06	0.82283577E-07	0.44146546E-07	0.29431028E-07	0.14633770E-07	0.65637700E-08
100	0.84701344E-06	0.16940265E-06	0.65154836E-07	0.40333973E-07	0.16320641E-07	0.66900334E-08

CUMULATIVE FREQUENCY DISTRIBUTION AT X=50000.0 Y=90.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.0	0.0	0.20694460E-10	0.11800114E-08
50	0.0	0.0	0.0	0.19688175E-11	0.81600304E-09	0.20301589E-08
75	0.0	0.31775851E-12	0.43454973E-09	0.16164319E-08	0.33696921E-08	0.34307661E-08
90	0.0	0.45849120E-08	0.82456673E-08	0.81490867E-08	0.73634006E-08	0.54987304E-08
95	0.51647667E-12	0.16686656E-07	0.15317362E-07	0.13467044E-07	0.11251270E-07	0.63576735E-08
99	0.73126216E-07	0.48637965E-07	0.42723254E-07	0.32386758E-07	0.24012792E-07	0.97243422E-08
99.5	0.14855880E-06	0.67772135E-07	0.55989315E-07	0.47435265E-07	0.26773638E-07	0.98806865E-08
99.9	0.36986239E-06	0.14183593E-06	0.11295282E-06	0.83600185E-07	0.33485254E-07	0.10276274E-07
100	0.20064053E-05	0.25080067E-06	0.12563481E-06	0.83756504E-07	0.44745281E-07	0.10615800E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=50000.0 Y=112.5 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*24 Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.0	0.12215150E-13	0.94269731E-11	0.13790900E-08	0.36257559E-08
50	0.0	0.84960749E-12	0.57637051E-09	0.22458901E-08	0.57519642E-08	0.77996631E-08
75	0.0	0.50542823E-08	0.12315439E-07	0.13372876E-07	0.14208720E-07	0.16725675E-07
90	0.75359941E-09	0.34573283E-07	0.33278660E-07	0.30816604E-07	0.27703496E-07	0.22581052E-07
95	0.44954490E-07	0.63112225E-07	0.49922416E-07	0.45039464E-07	0.36442820E-07	0.25874247E-07
99	0.29372399E-06	0.13212582E-06	0.10093214E-06	0.83105817E-07	0.58743339E-07	0.29899567E-07
99.5	0.39168447E-06	0.16215336E-06	0.11737382E-06	0.97298368E-07	0.68320730E-07	0.31828201E-07
99.9	0.64866003E-06	0.23529321E-06	0.19623712E-06	0.14293482E-06	0.86031491E-07	0.32638635E-07
100	0.12040582E-05	0.39247425E-06	0.22610521E-06	0.16602985E-06	0.97728844E-07	0.32739788E-07

TABLE 2.3-41

CUMULATIVE FREQUENCY DISTRIBUTION AT X=50000.0 Y=135.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.61075624E-12	0.31693310E-08	0.66972738E-08	0.12690837E-07	0.20674896E-07
50	0.0	0.10099292E-07	0.18149407E-07	0.20214408E-07	0.26503898E-07	0.34343568E-07
75	0.83025924E-08	0.44163883E-07	0.44727006E-07	0.45825875E-07	0.49546831E-07	0.47610992E-07
90	0.10267865E-06	0.97962243E-07	0.87843659E-07	0.87005276E-07	0.77128107E-07	0.63993582E-07
95	0.19357014E-06	0.15113994E-06	0.13635895E-06	0.12813746E-06	0.95651558E-07	0.70772842E-07
99	0.54038247E-06	0.32589605E-06	0.24782901E-06	0.22050733E-06	0.16026121E-06	0.81001417E-07
99.5	0.80859462E-06	0.42909994E-06	0.30845001E-06	0.27604972E-06	0.21563494E-06	0.85226077E-07
99.9	0.16250297E-05	0.66202301E-06	0.54818537E-06	0.50672486E-06	0.31821128E-06	0.89998935E-07
100	0.40223113E-05	0.87409626E-06	0.74308213E-06	0.58785832E-06	0.35921062E-06	0.96486076E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=80000.0 Y=315.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.65276673E-09	0.35078707E-08
50	0.0	0.0	0.33894190E-11	0.60718031E-09	0.38736765E-08	0.80197466E-08
75	0.0	0.19328388E-08	0.77655038E-08	0.92039798E-08	0.10344905E-07	0.11389400E-07
90	0.76484916E-10	0.25294739E-07	0.25548477E-07	0.25730539E-07	0.22523533E-07	0.17330667E-07
95	0.29226076E-07	0.48009852E-07	0.43391363E-07	0.39309654E-07	0.30696153E-07	0.21243380E-07
99	0.20973710E-06	0.11531552E-06	0.87721560E-07	0.75226922E-07	0.55266543E-07	0.27757252E-07
99.5	0.30590235E-06	0.14711543E-06	0.10695692E-06	0.91145296E-07	0.67181190E-07	0.28096817E-07
99.9	0.63627107E-06	0.21613107E-06	0.15217483E-06	0.43817931E-06	0.15639898E-06	0.29515526E-07
100	0.70768247E-05	0.13542603E-05	0.67714234E-06	0.45366562E-06	0.17241052E-06	0.29666275E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=80000.0 Y=337.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.84785207E-10	0.24457543E-08
50	0.0	0.0	0.0	0.45899451E-10	0.17995252E-08	0.35901322E-08
75	0.0	0.51442711E-10	0.28513145E-08	0.44887614E-08	0.54845017E-08	0.56187446E-08
90	0.0	0.12769597E-07	0.12972219E-07	0.13275660E-07	0.10979658E-07	0.66874719E-08
95	0.14491313E-08	0.25121921E-07	0.22533687E-07	0.20588466E-07	0.15167060E-07	0.73721367E-08
99	0.12983469E-06	0.61765377E-07	0.44980368E-07	0.35164355E-07	0.23213261E-07	0.11513386E-07
99.5	0.18876881E-06	0.73988758E-07	0.51687838E-07	0.46414165E-07	0.24687512E-07	0.11739687E-07
99.9	0.32741599E-06	0.13922465E-06	0.73254171E-07	0.61163576E-07	0.36286373E-07	0.12001777E-07
100	0.90657738E-06	0.18304536E-06	0.91745505E-07	0.66791301E-07	0.40278195E-07	0.12073233E-07

CUMULATIVE FREQUENCY DISTRIBUTION AT X=80000.0 Y=0.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.25106469E-14	0.99453712E-09
50	0.0	0.0	0.0	0.0	0.55796257E-09	0.16581954E-08
75	0.0	0.0	0.80589202E-10	0.95763264E-09	0.24900506E-08	0.25746785E-08
90	0.0	0.22760132E-08	0.61041199E-08	0.57902909E-08	0.43640469E-08	0.38284007E-08
95	0.0	0.12208240E-07	0.10959212E-07	0.91982102E-08	0.62144672E-08	0.48259530E-08
99	0.48402377E-07	0.30818953E-07	0.21944107E-07	0.20212426E-07	0.19731385E-07	0.65269639E-08
99.5	0.11177519E-06	0.42664013E-07	0.32363843E-07	0.32549750E-07	0.25111248E-07	0.65584551E-08
99.9	0.24386242E-06	0.15219257E-06	0.11290774E-06	0.10185670E-06	0.50956118E-07	0.65897545E-08
100	0.24445608E-05	0.30557010E-06	0.15278505E-06	0.15143064E-06	0.51036523E-07	0.65897545E-08

CUMULATIVE FREQUENCY DISTRIBUTION AT X=80000.0 Y=22.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.0	0.46133164E-09
50	0.0	0.0	0.0	0.0	0.12066736E-09	0.72181616E-09
75	0.0	0.0	0.52365095E-12	0.12807970E-09	0.12061769E-08	0.12459991E-08
90	0.0	0.22165453E-09	0.29164720E-08	0.31495748E-08	0.29592928E-08	0.22402589E-08
95	0.0	0.59576664E-08	0.67835089E-08	0.59894063E-08	0.43192188E-08	0.26721245E-08
99	0.17530951E-07	0.22263201E-07	0.17530020E-07	0.13612464E-07	0.84006615E-08	0.33540584E-08
99.5	0.65391646E-07	0.31685744E-07	0.21958471E-07	0.17399760E-07	0.99603028E-08	0.33648779E-08
99.9	0.16862316E-06	0.52199283E-07	0.35745270E-07	0.41626812E-07	0.15459236E-07	0.34284178E-08
100	0.99904355E-06	0.12488044E-06	0.62440222E-07	0.46377711E-07	0.15459236E-07	0.34394771E-08

CUMULATIVE FREQUENCY DISTRIBUTION AT X=80000.0 Y=45.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.0	0.62765548E-09
50	0.0	0.0	0.0	0.0	0.86242069E-10	0.94237307E-09
75	0.0	0.0	0.60965772E-13	0.47255977E-10	0.15263830E-08	0.18479362E-08
90	0.0	0.10838444E-09	0.36246495E-08	0.43227182E-08	0.32060408E-08	0.26872247E-08
95	0.0	0.74232460E-08	0.83683602E-08	0.71442834E-08	0.56041856E-08	0.31772152E-08
99	0.23824935E-07	0.25591824E-07	0.20618280E-07	0.19569001E-07	0.15346156E-07	0.49528595E-08
99.5	0.82678980E-07	0.39745835E-07	0.29984275E-07	0.22488898E-07	0.16998818E-07	0.58194978E-08
99.9	0.19101225E-06	0.12275189E-06	0.69057705E-07	0.60217360E-07	0.20355181E-07	0.61314793E-08
100	0.14452171E-05	0.18065225E-06	0.90326125E-07	0.60217417E-07	0.24044109E-07	0.61726304E-08

TABLE 2.3-41

CUMULATIVE FREQUENCY DISTRIBUTION AT X=80000.0 Y=67.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.0	0.40123016E-09
50	0.0	0.0	0.0	0.0	0.57695071E-10	0.76099349E-09
75	0.0	0.0	0.11640369E-12	0.46190177E-10	0.12886678E-08	0.12265651E-08
90	0.0	0.74503903E-10	0.28157352E-08	0.34115928E-08	0.29993801E-08	0.18697315E-08
95	0.0	0.57772347E-08	0.62005761E-08	0.55078466E-08	0.41141384E-08	0.20978645E-08
99	0.11210879E-07	0.23970365E-07	0.16480179E-07	0.12894198E-07	0.64184746E-08	0.36281811E-08
99.5	0.69407463E-07	0.31309682E-07	0.20505123E-07	0.14411793E-07	0.79329112E-08	0.36715551E-08
99.9	0.19176292E-06	0.50278885E-07	0.25450035E-07	0.17403586E-07	0.90481507E-08	0.37003107E-08
100	0.51563939E-06	0.10312783E-06	0.39664567E-07	0.24554254E-07	0.95206083E-08	0.37798848E-08

CUMULATIVE FREQUENCY DISTRIBUTION AT X=80000.0 Y=90.0 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours(16978)	24 Hours(16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.0	0.59401772E-11	0.70902839E-09
50	0.0	0.0	0.0	0.26056636E-12	0.40447090E-09	0.12827850E-08
75	0.0	0.31595447E-13	0.16143946E-09	0.78973983E-09	0.20683550E-08	0.21955537E-08
90	0.0	0.21497186E-08	0.51419100E-08	0.51890368E-08	0.45642992E-08	0.33887582E-08
95	0.16535089E-13	0.10429602E-07	0.94783310E-08	0.87783860E-08	0.73252231E-08	0.39429153E-08
99	0.45823214E-07	0.33394400E-07	0.24858196E-07	0.21363114E-07	0.14849093E-07	0.56996932E-08
99.5	0.93907090E-07	0.46579807E-07	0.37967766E-07	0.33527250E-07	0.15856369E-07	0.57991549E-08
99.9	0.24400498E-06	0.10003M176E-06	0.67518158E-07	0.50156849E-07	0.19876644E-07	0.59569629E-08
100	0.12037644E-05	0.15047056E-06	0.75310766E-07	0.50207177E-07	0.26520990E-07	0.61537762E-08

CUMULATIVE FREQUENCY DISTRIBUTION AT X=80000.0 Y=112.5 Z=0.0

Percentage of Total Hours	Hourly (17127)	8 Hours (17140)	16 Hours (16978)	24 Hours (16827)	3 Days (17161)	26 Days (16606)
25	0.0	0.0	0.0	0.17013136E-11	0.72735729E-09	0.21114108E-08
50	0.0	0.90145935E-13	0.24937985E-09	0.10941141E-08	0.33791445E-08	0.45784425E-08
75	0.0	0.23630458E-08	0.70595156E-08	0.79780342E-08	0.87566576E-08	0.10210552E-07
90	0.18571251E-09	0.20381883E-07	0.20710111E-07	0.18896735E-07	0.17511347E-07	0.13531970E-07
95	0.21296035E-07	0.38957580E-07	0.31680546E-07	0.28699560E-07	0.22742029E-07	0.16043234E-07
99	0.18808896E-06	0.83608597E-07	0.64716914E-07	0.50944852E-07	0.37962000E-07	0.18823801E-07
99.5	0.25296754E-06	0.10035802E-06	0.73881438E-07	0.61447224E-07	0.41299103E-07	0.20404279E-07
99.9	0.39284362E-06	0.15931448E-06	0.12458219E-06	0.88562729E-07	0.57374056E-07	0.20854820E-07
100	0.72630843E-06	0.26562009E-06	0.13284415E-06	0.11288694E-06	0.65481743E-07	0.20917575E-07

TABLE 2.3-41

CUMULATIVE FREQUENCY DISTRIBUTION AT X=80000.0 Y=135.0 Z=0.0

Percentage of Total Hours	*Hourly (17127)*	*8 Hours (17140)*	*16 Hours (16978)*	*Hours (16827)*	*3 Days (17161)*	*26 Days (16606)*
25	0.0	0.53608065E-13	0.15507566E-08	0.37072401E-08	0.72642301E-08	0.12430696E-07
50	0.0	0.54108469E-08	0.10682420E-07	0.12110306E-07	0.16393336E-07	0.21119209E-07
75	0.31305785E-08	0.26878141E-07	0.27722947E-07	0.28005950E-07	0.30296825E-07	0.29032226E-07
90	0.61701087E-07	0.60302057E-07	0.54364037E-07	0.52630334E-07	0.46747747E-07	0.38486551E-07
95	0.11980205E-06	0.91934908E-07	0.84140481E-07	0.78212679E-07	0.58057847E-07	0.43278874E-07
99	0.33317684E-06	0.19935442E-06	0.14976899E-06	0.13110741E-06	0.99965803E-07	0.47624940E-07
99.5	0.48037498E-06	0.25995200E-06	0.18830934E-06	0.17129037E-06	0.13088629E-06	0.50088705E-07
99.9	0.10322974E-05	0.40344401E-06	0.33194232E-06	0.30887401E-06	0.18952687E-06	0.52885735E-07
100	0.25595009E-05	0.53274880E-06	0.45236368E-06	0.35789134E-06	0.21528973E-06	0.56458632E-07

TABLE 2.3-42

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STABILITY BASED ON VERTICAL TEMPERATURE GRADIENT MAY 1973-APRIL 1974 EXTREMELY UNSTABLE (ΔT < -1.9°C/100M)

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.8	5.1	9.6	15.1	21.1	39.6	*Row Sums*	*Row Avg.*
CALM	1	0	0	0	0	0	1	11.3
22.50	6	6	5	0	0	0	17	5.1
45.00	4	6	1	1	0	0	12	4.8
67.50	8	18	0	1	1	0	28	4.9
90.00	8	10	4	2	0	0	24	5.6
112.50	3	11	2	4	5	1	26	10.3
135.00	7	10	3	7	1	0	28	8.2
157.50	4	5	0	1	0	0	10	4.9
180.00	4	6	0	0	0	0	10	3.5
202.50	1	7	3	1	0	0	12	6.4
225.00	3	4	5	12	1	0	25	11.3
247.50	1	2	0	1	2	0	6	11.4
270.00	6	3	1	0	0	0	10	3.9
292.50	9	14	11	2	6	1	43	8.6
315.00	17	22	21	38	2	18	138	13.9
337.50	8	17	13	20	1	7	76	12.5
360.00	7	10	15	2	0	0	34	7.1
Column Sums	96	51	85	92	49	27	500	9.9

TABLE 2.3-43

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STABILITY BASED ON VERTICAL TEMPERATURE GRADIENT MAY 1973-APRIL 1974 MODERATELY UNSTABLE (ΔT -1.9° to -1.7°C/100M)

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph						Row Sums	Row Avg.
	1.8	5.1	9.6	15.1	21.1	39.6		
CALM	0	0	0	0	0	0	0	0.0
22.50	1	0	1	0	0	0	2	5.3
45.00	0	0	0	0	0	0	0	0.0
67.50	4	1	1	0	0	0	6	3.9
90.00	1	1	5	1	0	0	8	8.6
112.50	1	0	3	1	0	0	5	8.8
135.00	2	3	3	5	0	0	13	9.9
157.50	4	5	1	0	0	0	10	4.5
180.00	1	1	0	0	0	0	2	3.6
202.50	1	1	0	0	0	0	2	3.9
225.00	1	1	0	0	0	0	2	3.3
247.50	1	0	1	0	0	0	2	5.3
270.00	0	2	1	0	0	0	3	5.9
292.50	2	2	2	3	0	0	9	8.6
315.00	4	8	6	6	4	0	28	10.1
337.50	1	0	3	5	2	3	14	16.8
360.00	1	3	6	1	0	0	11	8.7
Column Sums	25	28	33	22	6	3	117	9.1

TABLE 2.3-44
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE - STABILITY BASED ON VERTICAL TEMPERATURE GRADIENT MAY 1973-APRIL 1974 SLIGHTLY UNSTABLE
(ΔT -1.7 to -1.5°C/100M)

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph						Row Sums	Row Avg.
	1.8	5.1	9.6	15.1	21.1	39.6		
CALM	0	0	0	0	0	0	0	0.0
22.50	2	1	2	0	0	0	5	6.0
45.00	2	1	3	0	0	0	6	6.1
67.50	1	2	1	0	0	0	4	4.4
90.00	1	0	1	0	0	0	2	4.8
112.50	1	2	0	0	1	1	5	12.6
135.00	1	8	6	11	0	0	26	10.1
157.50	2	8	2	0	1	0	13	6.7
180.00	1	5	0	0	2	0	8	7.7
202.50	1	3	0	0	0	0	4	3.4
225.00	0	2	0	0	0	0	2	4.1
247.50	2	0	0	0	0	0	2	2.3
270.00	1	2	0	0	0	0	3	4.2
292.50	4	12	7	1	0	0	24	6.1
315.00	1	4	4	2	1	0	12	10.0
337.50	1	3	8	13	4	4	33	15.4
360.00	0	2	2	1	0	0	5	8.5
Column Sums	21	55	36	28	9	5	154	9.2

TABLE 2.3-45

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STABILITY BASED ON VERTICAL TEMPERATURE GRADIENT MAY 1973-APRIL 1974 NEUTRAL

(ΔT -1.5 to -0.5°C/100M)

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph						Row Sums	Row Avg.
	1.8	5.1	9.6	15.1	21.1	39.6		
CALM	2	0	0	0	0	0	2	1.4
22.50	8	31	24	4	0	1	68	6.9
45.00	12	22	19	10	0	0	63	7.1
67.50	15	12	14	3	0	1	45	6.4
90.00	12	22	8	5	1	0	48	6.3
112.50	8	37	32	33	12	3	125	10.8
135.00	22	83	73	39	16	7	240	9.3
157.50	27	107	20	12	10	11	187	8.2
180.00	20	54	5	1	0	0	80	4.2
202.50	15	23	3	2	1	0	44	4.9
225.00	23	12	4	7	2	0	48	6.0
247.50	13	15	3	0	1	0	32	4.3
270.00	22	32	4	1	0	0	59	4.1
292.50	28	124	71	27	4	1	255	7.2
315.00	18	106	222	230	209	145	930	15.7
337.50	9	44	69	65	61	35	283	14.9
360.00	17	50	42	19	2	0	130	7.6
Column Sums	271	774	613	458	319	204	2639	11.2

TABLE 2.3-46

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STABILITY BASED ON VERTICAL TEMPERATURE GRADIENT MAY 1973-APRIL 1974 SLIGHTLY STABLE

(ΔT -0.5 to 1.5°C/100M)

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph 1.8*	*5.1*	*9.6*	*15.1*	*21.1*	*39.6*	*Row Sums*	*Row Avg.*
CALM	*8*	*0*	*0*	*0*	*0*	*0*	*8*	*4.9*
22.50	*39*	*125*	*44*	*7*	*0*	*0*	*215*	*5.4*
45.00	*52*	*92*	*48*	*16*	*3*	*0*	*211*	*6.0*
67.50	*48*	*39*	*25*	*20*	*4*	*0*	*136*	*6.6*
90.00	*56*	*64*	*25*	*6*	*2*	*0*	*153*	*5.1*
112.50	*41*	*95*	*49*	*29*	*19*	*1*	*234*	*8.0*
135.00	*34*	*167*	*109*	*37*	*5*	*2*	*354*	*7.5*
157.50	*27*	*99*	*23*	*8*	*3*	*1*	*161*	*6.1*
180.00	*25*	*26*	*5*	*0*	*0*	*0*	*56*	*4.0*
202.50	*15*	*10*	*4*	*0*	*0*	*0*	*29*	*4.1*
225.00	*21*	*16*	*3*	*3*	*3*	*1*	*47*	*6.1*
247.50	*19*	*16*	*1*	*2*	*1*	*0*	*39*	*4.5*
270.00	*19*	*16*	*6*	*1*	*0*	*0*	*42*	*4.6*
292.50	*28*	*116*	*53*	*39*	*13*	*5*	*254*	*8.2*
315.00	*48*	*203*	*202*	*298*	*275*	*185*	*1211*	*15.3*
337.50	*29*	*120*	*128*	*113*	*30*	*10*	*430*	*10.5*
360.00	*33*	*128*	*101*	*32*	*2*	*0*	*296*	*7.3*
Column Sums	*537*	*1336*	*827*	*611*	*360*	*205*	*3876*	*9.8*

TABLE 2.3-47

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STABILITY BASED ON VERTICAL TEMPERATURE GRADIENT MAY 1973-APRIL 1974 MODERATELY STABLE (ΔT +1.5 to +4.0°C/100M)

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.8	5.1	9.6	15.1	21.1	39.6	*Row Sums*	*Row Avg.*
CALM	0	0	0	0	0	0	0	0.0
22.50	11	15	2	0	0	0	28	4.2
45.00	14	13	7	2	2	0	38	5.9
67.50	14	7	2	0	0	0	23	3.4
90.00	24	13	1	0	0	0	38	3.3
112.50	18	26	1	0	0	0	45	3.6
135.00	15	33	22	1	0	0	71	5.8
157.50	9	20	4	0	0	0	33	5.1
180.00	9	9	0	0	0	0	18	3.8
202.50	4	2	0	0	0	0	6	2.9
225.00	3	2	0	1	0	0	6	5.2
247.50	4	3	1	0	0	0	8	4.2
270.00	2	0	3	0	0	0	5	6.8
292.50	7	20	12	14	15	9	77	13.2
315.00	13	38	72	78	81	68	350	16.6
337.50	8	23	15	12	3	0	61	8.7
360.00	4	14	4	0	0	0	22	5.4
Column Sums	159	238	146	108	101	77	829	10.8

TABLE 2.3-48

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - STABILITY BASED ON VERTICAL TEMPERATURE GRADIENT MAY 1973-APRIL 1974 EXTREMELY STABLE (ΔT GREATER THAN 4.0°C/100M)

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.8	5.1	9.6	15.1	21.1	39.6	Row Sums	Row Avg.
CALM	0	0	0	0	0	0	0	0.0
22.50	4	1	0	0	0	0	5	3.3
45.00	2	2	1	0	0	0	5	4.8
67.50	1	0	0	0	0	0	1	2.9
90.00	3	3	0	0	0	0	6	3.3
112.50	3	8	0	0	0	0	11	3.7
135.00	7	18	4	1	0	0	30	4.9
157.50	7	7	0	0	0	0	14	3.5
180.00	2	1	0	0	0	1	4	8.6
202.50	5	0	0	1	0	0	6	3.5
225.00	3	0	0	0	0	0	3	2.0
247.50	1	2	0	0	0	0	3	3.7
270.00	0	3	0	1	0	0	4	6.7
292.50	0	10	6	5	4	6	31	13.3
315.00	6	45	40	30	27	28	176	14.0
337.50	3	7	2	1	2	0	15	8.0
360.00	2	0	1	0	0	0	3	4.6
Column Sums	49	107	54	39	33	35	317	10.7

TABLE 2.3-49

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A ANNUAL

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	*0*	*4*	*5*	*2*	*0*	*0*	*0*	*0*	*11*	*8.9*
45.00	*0*	*1*	*1*	*3*	*0*	*0*	*0*	*0*	*5*	*11.7*
67.50	*0*	*3*	*1*	*1*	*0*	*0*	*0*	*0*	*5*	*7.4*
90.00	*0*	*1*	*7*	*1*	*0*	*0*	*0*	*0*	*9*	*10.5*
112.50	*0*	*3*	*2*	*5*	*5*	*2*	*0*	*0*	*17*	*15.4*
135.00	*1*	*9*	*6*	*9*	*4*	*1*	*0*	*0*	*30*	*12.3*
157.50	*1*	*10*	*1*	*2*	*0*	*0*	*0*	*0*	*14*	*7.2*
180.00	*1*	*6*	*1*	*1*	*0*	*1*	*0*	*0*	*10*	*7.7*
202.50	*0*	*2*	*0*	*1*	*0*	*1*	*0*	*0*	*4*	*12.6*
225.00	*1*	*3*	*2*	*1*	*0*	*0*	*0*	*0*	*7*	*6.6*
247.50	*0*	*3*	*0*	*1*	*0*	*0*	*0*	*0*	*4*	*7.8*
270.00	*1*	*2*	*1*	*1*	*0*	*0*	*0*	*0*	*5*	*7.0*
292.50	*1*	*15*	*2*	*1*	*3*	*2*	*0*	*0*	*24*	*9.3*
315.00	*2*	*11*	*14*	*20*	*11*	*24*	*0*	*0*	*82*	*17.6*
337.50	*2*	*5*	*10*	*12*	*13*	*7*	*0*	*0*	*49*	*15.5*
360.00	*1*	*6*	*9*	*3*	*4*	*0*	*0*	*0*	*23*	*10.6*
Column Sums	*11*	*84*	*62*	*64*	*40*	*8*	*0*	*0*	*299*	*13.2*

Hours of Calm = 0
Sums of this table: row totals = 299 and column totals = 299

TABLE 2.3-50

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS B ANNUAL

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	1	1	2	1	0	0	0	0	5	7.7
45.00	0	1	0	0	1	0	0	0	2	13.0
67.50	1	0	2	0	0	0	0	0	3	8.5
90.00	0	2	5	1	0	0	0	0	8	8.7
112.50	0	0	6	1	0	0	0	0	7	10.7
135.00	2	4	8	6	0	1	0	0	21	10.7
157.50	4	9	6	0	0	2	0	0	21	7.8
180.00	2	1	3	2	0	0	0	0	8	8.5
202.50	1	4	0	0	0	0	0	0	5	3.8
225.00	1	2	2	0	0	0	0	0	5	6.2
247.50	2	3	1	0	0	0	0	0	6	4.4
270.00	1	4	1	0	1	0	0	0	7	6.8
292.50	3	11	6	2	0	0	0	0	22	6.7
315.00	4	10	12	13	9	9	0	0	57	14.4
337.50	1	0	3	7	6	4	0	0	21	18.1
360.00	0	3	8	1	1	0	0	0	13	10.1
Column Sums	23	55	65	34	18	16	0	0	211	10.9

Hours of Calm = 3
Sums of this table: row totals = 211 and column totals = 211

TABLE 2.3-51

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C ANNUAL

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	1	1	2	1	0	0	0	0	5	8.5
45.00	1	1	3	0	0	0	0	0	5	6.7
67.50	1	0	1	0	0	0	0	0	2	4.9
90.00	1	1	1	0	0	0	0	0	3	5.1
112.50	1	2	0	1	3	1	0	0	8	14.4
135.00	1	8	0	12	0	0	0	0	31	10.2
157.50	3	15	7	2	5	0	0	0	32	8.6
180.00	2	10	1	0	2	1	0	0	16	7.8
202.50	1	3	0	0	0	0	0	0	4	3.4
225.00	2	4	0	0	0	0	0	0	6	3.7
247.50	4	2	0	0	0	0	0	0	6	3.1
270.00	1	5	1	0	0	0	0	0	7	5.3
292.50	4	14	11	3	0	0	0	0	32	7.0
315.00	1	9	15	29	27	17	2	0	100	17.9
337.50	1	1	8	29	9	6	0	0	54	17.0
360.00	0	3	3	2	0	0	0	0	8	9.0
Column Sums	25	79	63	79	46	25	2	0	319	12.6

Hours of Calm = 0
Sums of this table: row totals = 319 and column totals = 319

TABLE 2.3-52

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D ANNUAL

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	18	79	62	14	0	1	0	0	174	7.1
45.00	22	62	41	14	1	0	0	0	140	6.8
67.50	18	32	22	6	0	0	0	0	78	6.0
90.00	23	48	13	8	1	0	0	0	93	5.8
112.50	23	130	97	61	27	9	0	0	347	9.7
135.00	37	237	167	88	41	40	0	0	610	10.0
157.50	46	215	56	26	15	21	3	0	382	8.1
180.00	32	105	16	6	0	0	0	0	159	4.7
202.50	40	48	8	2	1	0	0	0	99	4.4
225.00	50	29	4	5	1	0	0	0	89	4.2
247.50	25	34	6	1	0	0	0	0	66	4.2
270.00	57	78	15	3	1	0	0	0	154	4.4
292.50	62	290	200	81	13	5	0	0	651	7.8
315.00	41	247	532	652	501	319	6	0	2298	15.6
337.50	22	143	230	202	156	77	3	0	833	13.9
360.00	31	113	101	36	3	0	0	0	284	7.6
Column Sums	547	1890	1570	1205	761	472	12	0	6457	11.3

Hours of Calm = 5
Sums of this table: row totals = 6457 and column totals = 6457

TABLE 2.3-53

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS E ANNUAL

FREQUENCY TABLE

Mean Wind Direction				*Mean Wind Speed, mph*					*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*77*	*270*	*82*	*16*	*1*	*0*	*0*	*0*	*446*	*5.5*
45.00	*123*	*207*	*76*	*20*	*3*	*0*	*0*	*0*	*429*	*5.3*
67.50	*127*	*114*	*53*	*37*	*4*	*1*	*0*	*0*	*336*	*5.8*
90.00	*127*	*130*	*28*	*8*	*2*	*0*	*0*	*0*	*295*	*4.4*
112.50	*107*	*188*	*74*	*41*	*27*	*5*	*0*	*0*	*442*	*7.1*
135.00	*66*	*281*	*150*	*46*	*5*	*10*		*0*	*559*	*7.3*
157.50	*46*	*139*	*31*	*10*	*3*	*2*	*0*	*0*	*231*	*5.8*
180.00	*41*	*39*	*5*	*1*	*0*	*0*	*0*	*0*	*86*	*3.8*
202.50	*26*	*22*	*6*	*1*	*0*	*0*	*0*	*0*	*55*	*4.1*
225.00	*37*	*27*	*9*	*12*	*3*	*1*	*0*	*0*	*89*	*6.4*
247.50	*25*	*24*	*3*	*3*	*3*	*0*	*0*	*0*	*58*	*5.2*
270.00	*44*	*28*	*13*	*3*	*1*	*0*	*0*	*0*	*89*	*4.8*
292.50	*70*	*216*	*121*	*81*	*42*	*18*	*0*	*0*	*548*	*9.0*
315.00	*85*	*358*	*441*	*611*	*502*	*353*	*14*	*0*	*2364*	*15.4*
337.50	*68*	*253*	*210*	*169*	*52*	*11*	*1*	*0*	*764*	*9.6*
360.00	*78*	*266*	*171*	*44*	*3*	*1*	*0*	*0*	*563*	*6.8*
Column Sums	*1147*	*2562*	*1473*	*1103*	*651*	*402*	*16*	*0*	*7354*	*9.6*

Hours of Calm = 12
Sums of this table: row totals = 7354 and column totals = 7354

TABLE 2.3-54

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F ANNUAL

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sum	Row Avg.
22.50	22	32	9	0	0	0	0	0	63	4.6
45.00	23	24	14	2	2	0	0	0	65	5.5
67.50	28	20	3	0	1	0	0	0	52	3.9
90.00	46	24	1	1	0	0	0	0	72	3.4
112.50	41	58	6	0	0	0	0	0	105	4.0
135.00	26	66	32	1	0	0	2	0	127	6.0
157.50	19	32	4	0	0	0	0	0	55	4.5
180.00	11	14	0	0	0	0	0	0	25	3.8
202.50	11	3	0	0	0	0	0	0	14	2.6
225.00	7	9	1	6	3	0	0	0	26	8.1
247.50	5	9	3	1	1	0	0	0	19	5.8
270.00	9	3	4	0	0	0	0	0	16	4.6
292.50	14	35	29	21	22	18	0	0	139	12.6
315.00	30	83	121	147	158	176	16	0	731	17.7
337.50	16	47	30	27	10	0	0	0	130	9.1
360.00	11	29	11	0	1	0	0	0	52	5.6
Column Sums	319	488	268	206	198	194	18	0	1691	11.4

Hours of Calm = 0
Sums of this table: row totals = 1691 and column totals = 1691

TABLE 2.3-55

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M, TEMP GRAD 76-10M STABILITY CLASS G ANNUAL

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*4*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*6*	*3.4*
45.00	*6*	*5*	*1*	*1*	*0*	*0*	*0*	*0*	*13*	*4.7*
67.50	*3*	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*6*	*3.0*
90.00	*7*	*6*	*0*	*0*	*0*	*0*	*0*	*0*	*13*	*3.3*
112.50	*14*	*21*	*3*	*1*	*0*	*0*	*0*	*0*	*39*	*4.4*
135.00	*22*	*39*	*8*	*1*	*0*	*0*	*0*	*0*	*70*	*4.6*
157.50	*12*	*13*	*0*	*0*	*0*	*0*	*0*	*0*	*25*	*3.4*
180.00	*5*	*3*	*0*	*1*	*0*	*1*	*0*	*0*	*10*	*6.6*
202.50	*5*	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*6*	*3.5*
225.00	*8*	*3*	*1*	*2*	*0*	*0*	*0*	*0*	*14*	*5.3*
247.50	*4*	*9*	*0*	*0*	*0*	*0*	*0*	*0*	*13*	*4.2*
270.00	*5*	*4*	*0*	*1*	*0*	*0*	*0*	*0*	*10*	*4.8*
292.50	*3*	*15*	*21*	*10*	*5*	*8*	*0*	*0*	*62*	*12.0*
315.00	*19*	*75*	*62*	*61*	*52*	*69*	*10*	*0*	*348*	*15.5*
337.50	*3*	*17*	*8*	*5*	*2*	*2*	*0*	*0*	*37*	*9.3*
360.00	*7*	*2*	*2*	*0*	*0*	*0*	*0*	*0*	*11*	*4.2*
Column Sums	*127*	*217*	*106*	*84*	*59*	*80*	*10*	*0*	*683*	*11.0*

Hours of Calm = 0
Sums of this table: row totals = 683 and column totals = 683

TABLE 2.3-56

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A JAN.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	2	1	2	0	0	0	0	5	10.7
45.00	0	0	0	3	0	0	0	0	3	14.7
67.50	0	0	1	0	0	0	0	0	1	9.0
90.00	0	0	1	0	0	0	0	0	1	11.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	1	2	2	0	1	0	0	6	14.3
157.50	0	1	0	0	0	0	0	0	1	5.0
180.00	0	0	0	0	0	1	0	0	1	25.0
202.50	0	0	0	0	0	1	0	0	1	28.0
225.00	0	0	2	0	0	0	0	0	2	9.5
247.50	0	0	0	1	0	0	0	0	1	17.1
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	0	2	0	0	0	0	0	2	8.0
315.00	0	1	6	2	1	2	0	0	12	15.3
337.50	0	0	1	0	0	0	0	0	1	9.0
360.00	0	0	0	1	4	0	0	0	5	20.0
Column Sums	0	5	16	11	5	5	0	0	42	14.4

Hours of Calm = 0
Sums of this table: row totals = 42 and column totals = 42

TABLE 2.3-57

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS B JAN.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	0	1	1	0	0	0	0	2	12.0
45.00	0	1	0	0	1	0	0	0	2	13.0
67.50	0	0	1	0	0	0	0	0	1	11.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	1	0	0	1	0	0	2	20.7
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	1	0	0	0	0	0	1	8.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	2	0	0	0	0	0	2	10.5
247.50	1	0	0	0	0	0	0	0	1	3.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	0	0	0	0	0	0	0	0	0.0
315.00	0	0	0	5	1	2	0	0	8	18.5
337.50	0	0	0	2	0	0	0	0	2	13.0
360.00	0	0	2	0	1	0	0	0	3	12.7
Column Sums	1	1	8	8	3	3	0	0	24	14.4

Hours of Calm = 0
Sums of this table: row totals = 24 and column totals = 24

TABLE 2.3-58

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C JAN.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	1	0	0	0	0	1	14.2
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	0	0	0	0	0	0	0	0	0.0
315.00	0	0	2	0	1	0	0	0	3	14.2
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	1	0	0	0	0	1	16.2
Column Sums	0	0	2	2	1	0	0	0	5	14.6

Hours of Calm = 0
Sums of this table: row totals = 5 and column totals = 5

TABLE 2.3-59

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D JAN.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*3*	*7*	*11*	*6*	*0*	*0*	*0*	*0*	*27*	*9.0*
45.00	*0*	*12*	*5*	*5*	*0*	*0*	*0*	*0*	*22*	*8.0*
67.50	*4*	*4*	*1*	*2*	*0*	*0*	*0*	*0*	*11*	*6.3*
90.00	*1*	*5*	*1*	*5*	*1*	*0*	*0*	*0*	*13*	*10.2*
112.50	*3*	*6*	*15*	*19*	*3*	*0*	*0*	*0*	*46*	*11.3*
135.00	*1*	*12*	*14*	*17*	*7*	*14*	*0*	*0*	*65*	*15.7*
157.50	*1*	*11*	*6*	*7*	*1*	*8*	*3*	*0*	*37*	*16.1*
180.00	*0*	*5*	*0*	*0*	*0*	*0*	*0*	*0*	*5*	*4.2*
202.50	*1*	*10*		*0*	*0*	*0*	*0*	*0*	*11*	*4.4*
225.00	*4*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*5*	*3.1*
247.50	*1*	*1*	*3*	*0*	*0*	*0*	*0*	*0*	*5*	*7.3*
270.00	*1*	*6*	*1*	*1*	*0*	*0*	*0*	*0*	*9*	*5.8*
292.50	*1*	*5*	*10*	*3*	*0*	*1*	*0*	*0*	*20*	*10.0*
315.00	*0*	*1*	*19*	*46*	*29*	*9*	*0*	*0*	*104*	*16.7*
337.50	*1*	*3*	*12*	*9*	*9*	*13*	*3*	*0*	*50*	*18.7*
360.00	*1*	*4*	*7*	*3*	*1*	*0*	*0*	*0*	*16*	*9.9*
Column Sums	*23*	*93*	*105*	*123*	*51*	*45*	*6*	*0*	*446*	*13.4*

Hours of Calm = 0
Sums of this table: row totals = 446 and column totals = 446

TABLE 2.3-60

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M, TEMP GRAD 76-10M STABILITY CLASS E JAN.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	10	45	11	1	0	0	0	0	67	5.5
45.00	14	23	10	2	0	0	0	0	49	5.3
67.50	11	18	13	5	2	0	0	0	49	7.1
90.00	9	19	7	2	1	0	0	0	38	5.9
112.50	12	24	18	10	5	0	0	0	69	8.1
135.00	3	36	38	11	2	1	1	0	92	9.0
157.50	5	7	4	6	0	1	0	0	23	9.5
180.00	4	1	3	1	0	0	0	0	9	6.5
202.50	2	1	2	0	0	0	0	0	5	5.4
225.00	4	1	1	0	0	0	0	0	6	3.3
247.50	1	2	1	1	0	0	0	0	5	6.7
270.00	4	1	3	1	0	0	0	0	9	6.6
292.50	2	4	3	2	5	0	0	0	16	11.8
315.00	3	17	18	46	21	2	0	0	107	13.5
337.50	5	13	29	13	1	0	0	0	61	9.2
360.00	4	38	24	2	0	0	0	0	68	6.5
Column Sums	93	250	185	103	37	4	1	0	673	8.4

Hours of Calm = 0
Sums of this table: row totals = 673 and column totals = 673

TABLE 2.3-61

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F JAN.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*5*	*11*	*5*	*0*	*0*	*0*	*0*	*0*	*21*	*5.2*
45.00	*5*	*3*	*4*	*0*	*1*	*0*	*0*	*0*	*13*	*5.7*
67.50	*4*	*2*	*1*	*0*	*0*	*0*	*0*	*0*	*7*	*4.3*
90.00	*8*	*6*	*0*	*0*	*0*	*0*	*0*	*0*	*14*	*3.6*
112.50	*7*	*17*	*1*	*0*	*0*	*0*	*0*	*0*	*25*	*4.6*
135.00	*3*	*11*	*7*	*0*	*0*	*0*	*0*	*0*	*21*	*6.1*
157.50	*5*	*6*	*0*	*0*	*0*	*0*	*0*	*0*	*11*	*3.6*
180.00	*0*	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*3*	*4.2*
202.50	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*3*	*2.3*
225.00	*2*	*4*	*0*	*0*	*0*	*0*	*0*	*0*	*6*	*4.1*
247.50	*1*	*3*	*2*	*0*	*0*	*0*	*0*	*0*	*6*	*4.7*
270.00	*3*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*5*	*3.4*
292.50	*1*	*6*	*5*	*0*	*0*	*0*	*0*	*0*	*12*	*6.0*
315.00	*3*	*14*	*8*	*8*	*2*	*0*	*0*	*0*	*35*	*9.3*
337.50	*6*	*6*	*2*	*0*	*0*	*0*	*0*	*0*	*14*	*4.5*
360.00	*1*	*5*	*0*	*0*	*0*	*0*	*0*	*0*	*6*	*4.1*
Column Sums	*57*	*99*	*35*	*8*	*3*	*0*	*0*	*0*	*202*	*5.5*

Hours of Calm = 0
Sums of this table: row totals = 202 and column totals = 202

TABLE 2.3-62

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M, TEMP GRAD 76-10M STABILITY CLASS G JAN.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	2	0	0	0	0	0	0	0	2	2.5
67.50	0	1	0	0	0	0	0	0	1	4.0
90.00	0	1	0	0	0	0	0	0	1	4.0
112.50	3	4	0	0	0	0	0	0	7	3.7
135.00	6	6	3	0	0	0	0	0	15	4.8
157.50	2	2	0	0	0	0	0	0	4	3.3
180.00	2	1	0	1	0	0	0	0	4	6.4
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	1	2	0	0	0	0	0	0	3	3.9
247.50	2	6	0	0	0	0	0	0	8	4.5
270.00	1	1	0	0	0	0	0	0	2	5.0
292.50	1	0	5	1	0	0	0	0	7	9.8
315.00	5	9	4	3	1	0	0	0	22	7.8
337.50	0	1	0	0	0	0	0	0	1	5.0
360.00	2	0	1	0	0	0	0	0	3	5.0
Column Sums	27	34	13	5	1	0	0	0	80	5.9

Hours of Calm = 0
Sums of this table: row totals = 80 and column totals = 80

TABLE 2.3-63
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A FEB.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	0	0	0	0	0	0	0	0	0.0
315.00	0	0	0	0	0	0	0	0	0	0.0
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	0	0	0	0	0	0	0	0	0	0.0

Hours of Calm = 0
Sums of this table: row totals = 0 and column totals = 0

TABLE 2.3-64

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS B FEB.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	1	0	0	0	0	0	1	11.5
157.50	0	0	1	0	0	1	0	0	2	19.7
180.00	1	0	0	0	0	0	0	0	1	2.7
202.50	0	1	0	0	0	0	0	0	1	3.7
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	1	0	0	0	0	0	0	0	1	2.7
292.50	0	0	0	0	0	0	0	0	0	0.0
315.00	0	0	0	0	0	0	0	0	0	0.0
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	2	1	2	0	0	1	0	0	6	10.0

Hours of Calm = 0
Sums of this table: row totals = 6 and column totals = 6

TABLE 2.3-65

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C FEB.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*0*	*0*	*0*	*0*	*1*	*0*	*0*	*0*	*1*	*21.2*
135.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
157.50	*0*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*5.6*
180.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*4.0*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
225.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.6*
247.50	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.6*
270.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
292.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
315.00	*0*	*0*	*0*	*0*	*1*	*0*	*0*	*0*	*1*	*23.8*
337.50	*0*	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*1*	*17.2*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*1*	*4*	*0*	*1*	*2*	*0*	*0*	*0*	*8*	*10.4*

Hours of Calm = 0
Sums of this table: row totals = 8 and column totals = 8

TABLE 2.3-66

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D FEB.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	2	7	12	2	0	0	0	0	23	7.6
45.00	0	3	8	4	0	0	0	0	15	9.9
67.50	0	1	3	1	0	0	0	0	5	9.5
90.00	2	4	0	1	0	0	0	0	7	5.6
112.50	0	11	3	4	6	0	0	0	24	10.9
135.00	0	14	25	16	7	9	0	0	71	12.8
157.50	2	27	18	5	2	8	0	0	62	10.6
180.00	4	11	9	3	0	0	0	0	27	6.8
202.50	1	4	2	0	0	0	0	0	7	5.4
225.00	4	5	0	1	0	0	0	0	10	4.4
247.50	1	3	0	0	0	0	0	0	4	3.8
270.00	1	2	3	1	0	0	0	0	7	7.5
292.50	5	11	1	0	0	0	0	0	17	4.1
315.00	0	13	12	32	22	8	0	0	87	15.1
337.50	0	4	24	34	19	33	0	0	114	18.6
360.00	4	12	13	8	0	0	0	0	37	8.4
Column Sums	26	132	133	112	56	58	0	0	517	12.3

Hours of Calm = 0
Sums of this table: row totals = 517 and column totals = 517

TABLE 2.3-67

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS E FEB.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*8*	*42*	*16*	*1*	*0*	*0*	*0*	*0*	*67*	*5.4*
45.00	*21*	*29*	*19*	*4*	*1*	*0*	*0*	*0*	*74*	*5.8*
67.50	*19*	*19*	*7*	*7*	*0*	*0*	*0*	*0*	*52*	*5.5*
90.00	*27*	*24*	*4*	*0*	*0*	*0*	*0*	*0*	*55*	*3.7*
112.50	*20*	*24*	*4*	*1*	*4*	*0*	*0*	*0*	*53*	*5.4*
135.00	*6*	*10*	*2*	*7*	*1*	*0*	*0*	*0*	*26*	*7.9*
157.50	*0*	*10*	*4*	*1*	*1*	*0*	*0*	*0*	*16*	*7.4*
180.00	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*1.7*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
225.00	*1*	*1*	*1*	*1*	*0*	*0*	*0*	*0*	*4*	*8.1*
247.50	*1*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*3*	*3.2*
270.00	*1*	*2*	*0*	*0*	*1*	*0*	*0*	*0*	*4*	*7.3*
292.50	*3*	*2*	*3*	*1*	*3*	*0*	*0*	*0*	*12*	*10.0*
315.00	*3*	*13*	*22*	*30*	*35*	*4*	*0*	*0*	*107*	*14.6*
337.50	*4*	*17*	*23*	*26*	*10*	*1*	*0*	*0*	*81*	*11.4*
360.00	*8*	*26*	*44*	*8*	*0*	*0*	*0*	*0*	*86*	*8.2*
Column Sums	*123*	*221*	*149*	*87*	*56*	*5*	*0*	*0*	*641*	*8.2*

Hours of Calm = 0
Sums of this table: row totals = 641 and column totals = 641

TABLE 2.3-68

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F FEB.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	5	1	0	0	0	0	0	6	4.5
45.00	1	4	1	2	1	0	0	0	9	8.9
67.50	2	3	0	0	0	0	0	0	5	3.5
90.00	7	4	0	0	0	0	0	0	11	2.8
112.50	5	1	1	0	0	0	0	0	7	3.2
135.00	4	3	0	0	0	0	0	0	7	3.4
157.50	0	2	0	0	0	0	0	0	2	5.8
180.00	1	0	0	0	0	0	0	0	1	2.2
202.50	1	0	0	0	0	0	0	0	1	2.9
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	2	1	0	0	0	0	0	0	3	3.1
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	1	1	0	0	0	0	0	0	2	3.0
315.00	1	3	7	9	9	1	0	0	30	14.7
337.50	0	8	3	1	0	0	0	0	12	6.9
360.00	0	2	0	0	0	0	0	0	2	4.8
Column Sums	25	37	13	12	10	1	0	0	98	7.8

Hours of Calm = 0
Sums of this table: row totals = 98 and column totals = 98

TABLE 2.3-69
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS G FEB.

FREQUENCY TABLE

Mean Wind Direction	*1.5*	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*	*Row Sums*	*Row Avg.*
				Mean Wind Speed, mph						
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*1*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*3.0*
135.00	*0*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*4.0*
157.50	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.8*
180.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
225.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*		*0*	*0.0*
247.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
270.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
292.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
315.00	*0*	*3*	*4*	*0*	*3*	*0*	*0*	*0*	*10*	*11.8*
337.50	*0*	*6*	*2*	*0*	*0*	*0*	*0*	*0*	*8*	*6.2*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*2*	*12*	*6*	*0*	*3*	*0*	*0*	*0*	*23*	*8.1*

Hours of Calm = 0
Sums of this table: row totals = 23 and column totals = 23

TABLE 2.3-70

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A MARCH

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	1	0	0	0	0	0	0	1	6.1
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	1	0	0	0	0	0	0	1	6.5
135.00	0	1	0	0	3	0	0	0	4	17.8
157.50	0	1	1	1	0	0	0	0	3	10.7
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	1	0	0	0	0	0	0	1	3.8
292.50	0	1	0	0	0	0	0	0	1	7.0
315.00	0	0	0	0	0	0	0	0	0	0.0
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	1	0	0	0	0	0	0	1	3.5
Column Sums	0	7	1	1	3	0	0	0	12	10.9

Hours of Calm = 0
Sums of this table: row totals = 12 and column totals = 12

TABLE 2.3-71

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS B MARCH

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.9*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.2*
112.50	*0*	*0*	*2*	*0*	*0*	*0*	*0*	*0*	*2*	*10.8*
135.00	*0*	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*1*	*12.2*
157.50	*0*	*1*	*1*	*0*	*0*	*1*	*0*	*0*	*3*	*12.8*
180.00	*0*	*0*	*1*	*2*	*0*	*0*	*0*	*0*	*3*	*14.4*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
225.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
247.50	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*4.8*
270.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.1*
292.50	*0*	*1*	*1*	*0*	*0*	*0*	*0*	*0*	*2*	*7.7*
315.00	*0*	*0*	*2*	*2*	*3*	*4*	*0*	*0*	*11*	*21.4*
337.50	*0*	*0*	*0*	*0*	*4*	*1*	*0*	*0*	*5*	*23.6*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*0*	*6*	*7*	*5*	*7*	*6*	*0*	*0*	*31*	*16.1*

Hours of Calm = 0
Sums of this table: row totals = 31 and column totals = 31

TABLE 2.3-72

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C MARCH

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	1	0	0	0	0	0	0	0	1	2.8
112.50	0	0	0	1	1		0	0	2	15.5
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	1	3	3	1	4	0	0	0	12	11.8
180.00	1	3	1	0	0	1	0	0	6	8.6
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	1	0	0	0	0	0	0	1	4.6
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	1	0	0	0	0	0	0	1	5.5
292.50	0	0	2	1	0	0	0	0	3	11.3
315.00	0	5	2	15	17	13	2	0	54	20.2
337.50	0	0	0	14	4	2	0	0	20	18.2
360.00	0	1	0	0	0	0	0	0	1	5.9
Column Sums	3	14	8	32	26	16	2	0	101	17.2

Hours of Calm = 0
Sums of this table: row totals = 101 and column totals = 101

TABLE 2.3-73

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M, TEMP GRAD 76-10M STABILITY CLASS D MARCH

FREQUENCY TABLE

Mean Wind Direction				*Mean Wind Speed, mph*					*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*1*	*7*	*3*	*0*	*0*	*0*	*0*	*0*	*11*	*5.7*
45.00	*3*	*9*	*9*	*0*	*0*	*0*	*0*	*0*	*21*	*6.4*
67.50	*1*	*2*	*0*	*1*	*0*	*0*	*0*	*0*	*4*	*7.3*
90.00	*2*	*3*	*3*	*0*	*0*	*0*	*0*	*0*	*8*	*5.9*
112.50	*1*	*6*	*11*	*9*	*9*	*1*	*0*	*0*	*37*	*13.2*
135.00	*1*	*13*	*30*	*24*	*20*	*11*	*0*	*0*	*99*	*14.6*
157.50	*1*	*20*	*7*	*5*	*9*	*1*	*0*	*0*	*43*	*10.5*
180.00	*1*	*10*	*2*	*1*	*0*	*0*	*0*	*0*	*14*	*5.6*
202.50	*4*	*4*	*1*	*1*	*0*	*0*	*0*	*0*	*10*	*5.1*
225.00	*5*	*4*	*0*	*0*	*0*	*0*	*0*	*0*	*9*	*2.9*
247.50	*4*	*1*	*2*	*1*	*0*	*0*	*0*	*0*	*8*	*6.4*
270.00	*1*	*4*	*2*	*1*	*0*	*0*	*0*	*0*	*8*	*6.5*
292.50	*4*	*24*	*13*	*4*	*3*	*1*	*0*	*0*	*49*	*8.0*
315.00	*5*	*25*	*42*	*78*	*97*	*28*	*0*	*0*	*275*	*16.3*
337.50	*4*	*15*	*50*	*48*	*53*	*7*	*0*	*0*	*177*	*14.7*
360.00	*1*	*23*	*8*	*1*	*0*	*0*	*0*	*0*	*33*	*6.7*
Column Sums	*39*	*170*	*183*	*174*	*191*	*9*	*0*	*0*	*806*	*13.2*

Hours of Calm = 0

Sums of this table: row totals = 806 and column totals = 806

TABLE 2.3-74

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS E MARCH

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	3	26	7	2	0	0	0	0	38	5.4
45.00	9	24	10	3	0	0	0	0	46	5.7
67.50	5	6	4	3	0	0	0	0	18	6.6
90.00	4	10	3	2	0	0	0	0	19	6.3
112.50	5	16	6	12	7	0	0	0	46	10.0
135.00	6	35	22	9	2	4	0	0	78	9.1
157.50	4	18	5	1	2	0	0	0	30	6.7
180.00	2	2	0	0	0	0	0	0	4	3.1
202.50	4	1	1	0	0	0	0	0	6	4.2
225.00	3	1	0	0	0	0	0	0	4	2.9
247.50	1	1	0	0	0	0	0	0	2	3.7
270.00	2	1	2	0	0	0	0	0	5	5.2
292.50	1	10	8	2	0	0	0	0	21	6.7
315.00	3	19	14	22	19	4	0	0	81	13.4
337.50	1	10	20	17	6	1	0	0	55	11.8
360.00	0	20	13	7	0	0	0	0	40	8.1
Column Sums	53	200	115	80	36	9	0	0	493	8.8

Hours of Calm = 0
Sums of this table: row totals = 493 and column totals = 493

TABLE 2.3-75

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F MARCH

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	1	2	0	0	0	0	0	3	9.7
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	1	1	0	0	0	0	0	0	2	3.0
112.50	0	1	0	0	0	0	0	0	1	3.3
135.00	0	0	1	0	0	0	0	0	1	8.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	1	0	0	0	0	0	0	0	1	2.4
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	3	0	2	3	0	0	0	8	13.2
315.00	1	3	2	2	7	0	0	0	15	14.4
337.50	0	2	0	0	0	0	0	0	2	6.1
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	3	11	5	4	10	0	0	0	33	11.6

Hours of Calm = 0
Sums of this table: row totals = 33 and column totals = 33

TABLE 2.3-76

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS G MARCH

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
135.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
157.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
180.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
225.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
247.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
270.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
292.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
315.00	*0*	*0*	*3*	*1*	*1*	*0*	*0*	*0*	*5*	*12.6*
337.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*0*	*0*	*3*	*1*	*1*	*0*	*0*	*0*	*5*	*12.6*

Hours of Calm = 0
Sums of this table: row totals = 5 and column totals = 5

TABLE 2.3-77

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A APRIL

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
135.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*6.0*
157.50	*0*	*5*	*0*	*0*	*0*	*0*	*0*	*0*	*5*	*5.5*
180.00	*0*	*4*	*1*	*0*	*0*	*0*	*0*	*0*	*5*	*5.6*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
225.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*5.1*
247.50	*0*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*5.0*
270.00	*1*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*3.1*
292.50	*0*	*2*	*0*	*1*	*0*	*0*	*0*	*0*	*3*	*7.7*
315.00	*0*	*1*	*2*		*0*	*0*	*0*	*0*	*3*	*9.0*
337.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*1*	*17*	*3*	*1*	*0*	*0*	*0*	*0*	*22*	*6.0*

Hours of Calm = 0
Sums of this table: row totals = 22 and column totals = 22

TABLE 2.3-78

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M, TEMP GRAD 76-10M STABILITY CLASS B APRIL

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	3	1	0	0	0	0	0	4	5.8
157.50	0	3	4	0	0	0	0	0	7	7.1
180.00	0	0	1	0	0	0	0	0	1	7.1
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	1	0	0	0	0	0	0	1	4.0
292.50	1	4	2	0	0	0	0	0	7	5.8
315.00	1	4	6	2	2	2	0	0	17	12.5
337.50	0	0	0	0	1	1	0	0	2	24.5
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	2	15	14	2	3	3	0	0	39	9.9

Hours of Calm = 0
Sums of this table: row totals = 39 and column totals = 39

TABLE 2.3-79

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C APRIL

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	1	0	0	0	0	0	0	1	5.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	1	2	0	0	0	0	0	3	8.6
157.50	0	1	1	1	0	0	0	0	3	7.9
180.00	0	2	0	0	0	0	0	0	2	5.6
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	2	0	0	0	0	0	0	0	2	2.9
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	1	1	0	0	0	0	0	2	7.9
292.50	1	2	2	0	0	0	0	0	5	7.4
315.00	0	0	7	12	6	4	0	0	29	17.0
337.50	0	0	0	0	1	0	0	0	1	19.2
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	3	8	13	13	7	4	0	0	48	13.3

Hours of Calm = 0
Sums of this table: row totals = 48 and column totals = 48

TABLE 2.3-80
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D APRIL

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	14	4	1	0	0	0	0	19	6.2
45.00	1	4	2	0	0	0	0	0	7	5.3
67.50	4	0	3	0	0	0	0	0	7	4.9
90.00	1	2	0	0	0	0	0	0	3	3.7
112.50	1	10	23	0	0	0	0	0	34	8.5
135.00	2	17	7	1	0	0	0	0	27	6.5
157.50	4	9	5	3	0	0	0	0	21	6.6
180.00	1	4	1	2	0	0	0	0	8	7.3
202.50	2	3	0	0	0	0	0	0	5	3.4
225.00	1	2	1	0	0	0	0	0	4	5.5
247.50	1	2	0	0	0	0	0	0	3	4.2
270.00	3	1	2	0	1	0	0	0	7	7.2
292.50	4	13	12	6	0	0	0	0	35	7.8
315.00	1	17	61	71	66	69	0	0	285	17.5
337.50	3	24	63	39	25	9	0	0	163	12.9
360.00	4	13	21	1	0	0	0	0	39	7.2
Column Sums	33	135	205	124	92	78	0	0	667	12.9

Hours of Calm = 0
Sums of this table: row totals = 667 and column totals = 667

TABLE 2.3-81

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS E APRIL

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	7	21	10	0	0	0	0	0	38	5.5
45.00	13	14	3	4	0	0	0	0	34	5.3
67.50	7	4	2	1	0	0	0	0	14	5.0
90.00	7	6	1	0	0	0	0	0	14	3.8
112.50	7	13	4	3	0	0	0	0	27	5.7
135.00	6	5	6	1	0	0	0	0	18	6.1
157.50	1	3	1	0	0	0	0	0	5	4.4
180.00	3	0	0	0	0	0	0	0	3	3.0
202.50	1	1	0	0	0	0	0	0	2	3.5
225.00	0	1	0	0	0	0	0	0	1	4.8
247.50	0	1	0	1	0	0	0	0	2	9.5
270.00	2	0	0	1	0	0	0	0	3	7.3
292.50	0	1	7	9	1	0	0	0	18	12.9
315.00	3	7	19	38	48	42	0	0	157	18.7
337.50	2	12	15	23	11	6	0	0	69	13.6
360.00	14	20	28	5	0	0	0	0	67	6.8
Column Sums	73	109	96	86	60	48	0	0	472	11.5

Hours of Calm = 1
Sums of this table: row totals = 472 and column totals = 472

TABLE 2.3-82

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F APRIL

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	2	2	0	0	0	0	0	0	4	4.6
45.00	3	3	3	0	0	0	0	0	9	5.2
67.50	6	4	0	0	0	0	0	0	10	3.2
90.00	5	5	0	0	0	0	0	0	10	3.7
112.50	3	0	0	0	0	0	0	0	3	2.7
135.00	3	1	1	0	0	0	0	0	5	5.0
157.50	2	1	0	0	0	0	0	0	3	3.4
180.00	3	1	0	0	0	0	0	0	4	3.4
202.50	1	1	0	0	0	0	0	0	2	3.0
225.00	1	0	0	0	0	0	0	0	1	2.9
247.50	0	1	0	0	0	0	0	0	1	5.2
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	1	2	2	2	0	0	0	7	12.7
315.00	3	3	12	9	22	15	0	0	64	18.2
337.50	1	3	3	1	0	0	0	0	8	7.4
360.00	0	3	1	0	0	0	0	0	4	6.3
Column Sums	33	29	22	12	24	15	0	0	135	11.4

Hours of Calm = 0
Sums of this table: row totals = 135 and column totals = 135

TABLE 2.3-83

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS G APRIL

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	2	0	0	0	0	0	0	0	2	2.4
45.00	0	0	1	0	0	0	0	0	1	8.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	2	2	0	0	0	0	0	0	4	3.5
112.50	0	1	0	0	0	0	0	0	1	3.6
135.00	0	1	0	0	0	0	0	0	1	4.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	1	0	0	0	0	0	0	0	1	1.6
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	3	1	0	0	0	0	0	4	5.9
315.00	0	8	8	3	10	2	0	0	31	13.6
337.50	0	1	2	0	0	0	0	0	3	9.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	5	16	12	3	10	2	0	0	48	10.6

Hours of Calm = 0
Sums of this table: row totals = 48 and column totals = 48

TABLE 2.3-84

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A MAY

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	1	0	0	0	0	0	0	0	1	3.0
315.00	1	4	0	0	0	0	0	0	5	4.8
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	2	4	0	0	0	0	0	0	6	4.5

Hours of Calm = 0
Sums of this table: row totals = 6 and column totals = 6

TABLE 2.3-85

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS B MAY

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
135.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
157.50	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*4.0*
180.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
225.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.8*
247.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
270.00	*0*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*4.1*
292.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
315.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
337.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*0*	*4*	*0*	*0*	*0*	*0*	*0*	*0*	*4*	*4.0*

Hours of Calm = 0
Sums of this table: row totals = 4 and column totals = 4

TABLE 2.3-86

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C MAY

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	0	1	0	0	0	0	0	0	1	6.5
180.00	0	1	0	0	0	0	0	0	1	4.4
202.50	0	1	0	0	0	0	0	0	1	3.1
225.00	0	1	0	0	0	0	0	0	1	3.8
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	1	0	0	0	0	0	0	1	5.4
292.50	0	2	1	0	0	0	0	0	3	5.6
315.00	0	0	0	0	0	0	0	0	0	0.0
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	0	7	1	0	0	0	0	0	8	5.0

Hours of Calm = 0
Sums of this table: row totals = 8 and column totals = 8

TABLE 2.3-87

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D MAY

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	*0*	*3*	*2*	*0*	*0*	*0*	*0*	*0*	*5*	*6.1*
45.00	*3*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*4*	*2.6*
67.50	*0*	*7*	*0*	*0*	*0*	*0*	*0*	*0*	*7*	*3.6*
90.00	*3*	*3*	*1*	*0*	*0*	*0*	*0*	*0*	*7*	*4.4*
112.50	*2*	*15*	*2*	*0*	*0*	*0*	*0*	*0*	*19*	*5.3*
135.00	*8*	*25*	*5*	*0*	*0*	*0*	*0*	*0*	*38*	*4.8*
157.50	*9*	*12*	*1*	*0*	*0*	*0*	*0*	*0*	*22*	*3.8*
180.00	*3*	*13*	*0*	*0*	*0*	*0*	*0*	*0*	*16*	*3.7*
202.50	*4*	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*7*	*3.0*
225.00	*6*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*7*	*2.7*
247.50	*4*	*5*	*0*	*0*	*0*	*0*	*0*	*0*	*9*	*3.4*
270.00	*9*	*12*	*1*	*0*	*0*	*0*	*0*	*0*	*22*	*3.5*
292.50	*8*	*32*	*25*	*6*	*0*	*0*	*0*	*0*	*71*	*6.9*
315.00	*2*	*23*	*72*	*94*	*79*	*80*	*2*	*0*	*352*	*17.7*
337.50	*1*	*15*	*22*	*8*	*18*	*11*	*0*	*0*	*75*	*15.0*
360.00	*2*	*5*	*2*	*0*	*0*	*0*	*0*	*0*	*9*	*4.7*
Column Sums	*64*	*175*	*133*	*108*	*97*	*91*	*2*	*0*	*670*	*12.8*

Hours of Calm = 0
Sums of this table: row totals = 670 and column totals = 670

TABLE 2.3-88

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS E MAY

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	4	11	5	0	0	0	0	0	20	5.2
45.00	5	3	1	0	0	0	0	0	9	4.1
67.50	4	2	1	0	0	0	0	0	7	4.1
90.00	2	3	1	0	0	0	0	0	6	5.0
112.50	1	12	5	1	0	0	0	0	19	6.3
135.00	6	22	1	1	0	0	0	0	30	4.6
157.50	3	7	2	0	0	0	0	0	12	4.8
180.00	5	2	0	0	0	0	0	0	7	2.9
202.50	3	4	0	0	0	0	0	0	7	4.2
225.00	5	3	0	0	0	0	0	0	8	2.9
247.50	6	2	0	0	0	0	0	0	8	2.8
270.00	6	3	1	0	0	0	0	0	10	3.9
292.50	7	32	2	10	6	1	0	0	58	8.3
315.00	13	42	47	45	47	56	0	0	250	15.7
337.50	5	27	23	15	9	1	0	0	80	9.8
360.00	6	28	8	1	0	0	0	0	43	5.6
Column Sums	81	203	97	73	62	58	0	0	574	10.6

Hours of Calm = 0
Sums of this table: row totals = 574 and column totals = 574

TABLE 2.3-89

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F MAY

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	1	1	0	0	0	0	0	0	2	3.9
45.00	0	0	1	0	0	0	0	0	1	7.1
67.50	0	1	0	0	0	0	0	0	1	3.9
90.00	1	1	0	0	0	0	0	0	2	3.1
112.50	0	1	0	0	0	0	0	0	1	6.3
135.00	0	4	3	0	0	0	0	0	7	6.6
157.50	0	0	1	0	0	0	0	0	1	7.5
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	1	0	1	0	0	0	0	2	9.3
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	2	0	0	0	0	0	2	10.3
292.50	1	3	1	5	4	2	0	0	16	15.2
315.00	2	6	10	15	14	37	0	0	84	20.6
337.50	0	1	1	4	1	0	0	0	7	13.7
360.00	1	1	0	0	0	0	0	0	2	3.1
Column Sums	6	20	19	25	19	39	0	0	128	17.2

Hours of Calm = 0
Sums of this table: row totals = 128 and column totals = 128

TABLE 2.3-90

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS G MAY

FREQUENCY TABLE

Mean Wind Direction				*Mean Wind Speed, mph*					*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.6*
45.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*4.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.6*
112.50	*1*	*3*	*3*	*1*	*0*	*0*	*0*	*0*	*8*	*7.3*
135.00	*2*	*3*	*2*	*0*	*0*	*0*	*0*	*0*	*7*	*5.3*
157.50	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.5*
180.00	*0*	*0*	*0*	*0*	*0*	*1*	*0*	*0*	*1*	*24.4*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
225.00	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.0*
247.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
270.00	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.7*
292.50	*0*	*2*	*2*	*1*	*0*	*1*	*0*	*0*	*6*	*10.5*
315.00	*3*	*9*	*15*	*8*	*7*	*19*	*0*	*0*	*61*	*16.5*
337.50	*0*	*2*	*1*	*1*	*0*	*2*	*0*	*0*	*6*	*14.7*
360.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*5.8*
Column Sums	*11*	*21*	*23*	*11*	*7*	*23*	*0*	*0*	*96*	*13.5*

Hours of Calm = 0
Sums of this table: row totals = 96 and column totals = 96

TABLE 2.3-91
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A JUNE

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	1	0	0	0	0	0	0	1	4.0
90.00	0	1	1	0	0	0	0	0	2	9.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	3	1	0	0	0	0	0	4	6.2
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	1	0	0	0	0	0	0	1	5.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	2	0	0	0	0	0	0	2	4.0
315.00	0	1	1	0	0	0	0	0	2	8.0
337.50	1	1	0	0	2	0	0	0	4	12.5
360.00	0	1	0	0	0	0	0	0	1	5.0
Column Sums	1	11	3	0	2	0	0	0	17	7.7

Hours of Calm = 0
Sums of this table: row totals = 17 and column totals = 17

TABLE 2.3-92

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS B JUNE

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	2	0	0	0	0	0	0	2	4.2
315.00	0	1	1	0	0	0	0	0	2	9.0
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	0	3	1	0	0	0	0	0	4	6.6

Hours of Calm = 0
Sums of this table: row totals = 4 and column totals = 4

TABLE 2.3-93

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C JUNE

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
135.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
157.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
180.00	*0*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*4.2*
202.50	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.7*
225.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
247.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
270.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
292.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
315.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
337.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*0*	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*3*	*4.1*

Hours of Calm = 0
Sums of this table: row totals = 3 and column totals = 3

TABLE 2.3-94

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D JUNE

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	2	0	0	0	0	0	0	2	3.6
45.00	0	3	0	0	0	0	0	0	3	4.5
67.50	2	1	0	0	0	0	0	0	3	2.8
90.00	1	3	1	1	0	0	0	0	6	7.1
112.50	0	7	11	2	0	0	0	0	20	8.5
135.00	4	19	10	3	0	0	0	0	36	6.3
157.50	6	17	3	0	0	0	0	0	26	4.6
180.00	3	6	0	0	0	0	0	0	9	3.5
202.50	2	1	0	0	0	0	0	0	3	3.2
225.00	4	3	0	0	0	0	0	0	7	3.1
247.50	3	2	0	0	0	0	0	0	5	3.2
270.00	8	8	0	0	0	0	0	0	16	3.4
292.50	3	18	19	0	0	0	0	0	40	6.9
315.00	8	29	62	73	47	37	0	0	256	15.3
337.50	2	10	8	13	11	3	0	0	47	13.5
360.00	1	7	1	0	0	0	0	0	9	5.0
Column Sum	47	136	115	92	58	40	0	0	488	11.5

Hours of Calm = 0
Sums of this table: row totals = 488 and column totals = 488

TABLE 2.3-95

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS E JUNE

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	4	5	4	0	0	0	0	0	13	5.2
45.00	4	1	0	0	0	0	0	0	5	2.6
67.50	3	0	0	0	0	0	0	0	3	2.3
90.00	4	2	0	0	0	0	0	0	6	2.8
112.50	4	12	10	3	0	0	0	0	29	6.7
135.00	4	16	8	1	0	0	0	0	29	5.9
157.50	4	13	3	0	0	0	0	0	20	5.0
180.00	4	4	0	0	0	0	0	0	8	3.4
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	1	2	0	0	0	0	0	0	3	3.7
247.50	3	0	0	0	0	0	0	0	3	2.4
270.00	4	0	0	0	0	0	0	0	4	2.4
292.50	7	22	11	4	2	1	0	0	47	7.7
315.00	7	29	46	87	60	66	7	0	302	17.8
337.50	10	19	10	6	2	0	0	0	47	7.4
360.00	4	21	1	1	0	1	0	0	28	5.6
Column Sums	67	1 46	93	107	64	68	7	0	547	12.6

Hours of Calm = 0
Sums of this table: row totals = 547 and column totals = 547

TABLE 2.3-96

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F JUNE

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*1*		*0*	*0*	*0*	*0*	*0*	*1*	*4.3*
112.50	*0*	*3*	*2*	*0*	*0*	*0*	*0*	*0*	*5*	*6.7*
135.00	*1*	*3*	*3*	*0*	*0*	*0*	*0*	*0*	*7*	*6.2*
157.50	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*1*	*10.1*
180.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
247.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
270.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.7*
292.50	*0*	*0*	*2*	*2*	*1*	*0*	*0*	*0*	*5*	*14.2*
315.00	*1*	*8*	*3*	*21*	*23*	*24*	*6*	*0*	*86*	*21.2*
337.50	*2*	*2*	*3*	*5*	*6*	*0*	*0*	*0*	*18*	*13.1*
360.00	*1*	*0*	*2*	*0*	*0*	*0*	*0*	*0*	*3*	*7.0*
Column Sums	*6*	*18*	*16*	*28*	*30*	*24*	*6*	*0*	*128*	*17.6*

Hours of Calm = 0
Sums of this table: row totals = 128 and column totals = 128

TABLE 2.3-97

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS G JUNE

FREQUENCY TABLE

Mean Wind Direction	*1.5*	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*	*Row Sums*	*Row Avg.*
				Mean Wind Speed, mph						
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*4.7*
112.50	*1*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*3*	*3.1*
135.00	*2*	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*5*	*4.1*
157.50	*3*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*5*	*3.7*
180.00	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*1.8*
202.50	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*3*	*1.6*
225.00	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*1.1*
247.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
270.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
292.50	*0*	*1*	*3*	*4*	*0*	*1*	*0*	*0*	*9*	*14.2*
315.00	*1*	*15*	*11*	*18*	*13*	*30*	*7*	*0*	*95*	*19.7*
337.50	*2*	*1*	*2*	*1*	*1*	*0*	*0*	*0*	*7*	*9.3*
360.00	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.4*
Column Sums	*15*	*25*	*16*	*23*	*14*	*31*	*7*	*0*	*131*	*16.2*

Hours of Calm = 0
Sums of this table: row totals = 131 and column totals = 131

TABLE 2.3-98

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A JULY

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	2	0	0	0	0	0	0	2	5.1
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	1	0	0	0	0	0	0	1	4.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	2	0	0	3	0	0	0	5	14.9
315.00	0	0	3	10	7	11	0	0	31	20.8
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	0	5	3	10	10	11	0	0	39	18.8

Hours of Calm = 0
Sums of this table: row totals = 39 and column totals = 39

TABLE 2.3-99

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M, TEMP GRAD 76-10M STABILITY CLASS B JULY

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*1*	*12.0*
135.00	*1*	*0*	*2*	*0*	*0*	*0*	*0*	*0*	*3*	*6.8*
157.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
180.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
202.50	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*4.0*
225.00	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.9*
247.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
270.00	*0*	*0*	*0*	*0*	*0*	*0*		*0*	*0*	*0.0*
292.50	*0*	*3*	*1*	*0*	*0*	*0*	*0*	*0*	*4*	*6.5*
315.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
337.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*2*	*4*	*4*	*0*	*0*	*0*	*0*	*0*	*10*	*6.5*

Hours of Calm = 0
Sums of this table: row totals = 10 and column totals = 10

TABLE 2.3-100

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C JULY

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.5*
135.00	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*1*	*7.1*
157.50	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*4.6*
180.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
225.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
247.50	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.6*
270.00	*1*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*3.6*
292.50	*1*	*3*	*1*	*0*	*0*	*0*	*0*	*0*	*5*	*5.2*
315.00	*0*	*1*	*0*	*0*	*1*	*0*	*0*	*0*	*2*	*10.7*
337.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*4*	*6*	*2*	*0*	*1*	*0*	*0*	*0*	*13*	*5.5*

Hours of Calm = 0
Sums of this table: row totals = 13 and column totals = 13

TABLE 2.3-101

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D JULY

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	3	6	0	0	0	0	0	0	9	3.4
45.00	4	0	0	0	0	0	0	0	4	2.4
67.50	1	1	0	0	0	0	0	0	2	3.9
90.00	1	3	0	0	0	0	0	0	4	4.3
112.50	2	22	8	5	0	0	0	0	37	7.1
135.00	6	21	17	2	0	0	0	0	46	6.6
157.50	7	29	4	0	0	0	0	0	40	5.0
180.00	7	10	0	0	0	0	0	0	17	3.3
202.50	3	2	0	0	0	0	0	0	5	2.5
225.00	9	5	1	0	0	0	0	0	15	3.5
247.50	3	5	0	0	0	0	0	0	8	3.2
270.00	6	8	0	0	0	0	0	0	14	3.4
292.50	9	46	23	14	0	0	0	0	92	7.5
315.00	7	55	84	49	56	39	3	0	293	14.4
337.50	4	18	8	3	0	0	0	0	33	6.8
360.00	7	8	0	0	0	0	0	0	15	3.5
Column Sums	79	239	145	73	56	39	3	0	634	9.8

Hours of Calm = 0
Sums of this table: row totals = 634 and column totals = 634

TABLE 2.3-102

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M, TEMP GRAD 76-10M STABILITY CLASS E JULY

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	1	2	0	0	0	0	0	0	3	3.8
45.00	1	1	0	0	0	0	0	0	2	3.2
67.50	1	0	0	0	0	0	0	0	1	2.1
90.00	5	2	0	0	0	0	0	0	7	2.7
112.50	3	7	3	0	0	0	0	0	13	5.8
135.00	6	15	9	2	0	0	0	0	32	5.9
157.50	4	15	2	0	0	0	0	0	21	4.7
180.00	6	6	0	0	0	0	0	0	12	3.3
202.50	3	2	1	0	0	0	0	0	6	3.9
225.00	3	2	0	0	0	0	0	0	5	2.7
247.50	2	4	0	0	0	0	0	0	6	3.3
270.00	7	4	2	0	0	0	0	0	13	3.6
292.50	15	24	25	15	2	2	0	0	83	8.2
315.00	15	59	83	86	80	74	5	0	402	15.9
337.50	7	20	11	7	0	0	0	0	45	7.3
360.00	5	6	3	0	0	0	0	0	14	4.9
Column Sums	84	169	139	110	82	76	5	0	665	12.1

Hours of Calm = 0

Sums of this table: row totals = 665 and column totals = 665

TABLE 2.3-103

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F JULY

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	1	0	0	0	0	0	1	9.3
135.00	1	5	2	0	0	0	0	0	8	5.7
157.50	0	1	0	0	0	0	0	0	1	6.8
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	1	0	0	0	0	0	0	0	1	2.7
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	1	0	0	0	0	0	0	0	1	1.9
270.00	1	0	0	0	0	0	0	0	1	2.5
292.50	1	3	4	1	0	4	0	0	13	14.0
315.00	2	6	3	12	8	31	4	0	66	22.0
337.50	0	0	0	1	0	0	0	0	1	17.0
360.00	0	0	0	0	1	0	0	0	1	19.0
Column Sums	7	15	10	14	9	35	4	0	94	18.5

Hours of Calm = 0
Sums of this table: row totals = 94 and column totals = 94

TABLE 2.3-104

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS G JULY

FREQUENCY TABLE

Mean Wind Direction				*Mean Wind Speed, mph*					*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.1*
112.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
135.00	*0*	*2*	*1*	*0*	*0*	*0*	*0*	*0*	*3*	*6.5*
157.50	*0*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*4.6*
180.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*6.4*
202.50	*0*	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*1*	*12.2*
225.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
247.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
270.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
292.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
315.00	*0*	*1*	*1*	*0*	*2*	*3*	*1*	*0*	*8*	*23.0*
337.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*0*	*7*	*2*	*1*	*2*	*3*	*1*	*0*	*16*	*14.7*

Hours of Calm = 0
Sums of this table: row totals = 16 and column totals = 16

TABLE 2.3-105

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A AUG.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	0	1	0	0	0	0	0	0	1	5.8
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	1	0	0	0	0	0	0	1	4.2
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	5	0	0	0	0	0	0	5	4.4
315.00	1	2	0	0	0	1	0	0	4	9.7
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	1	9	0	0	0	1	0	0	11	6.4

Hours of Calm = 0
Sums of this table: row totals = 11 and column totals = 11

TABLE 2.3-106

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS B AUG.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	1	0	0	0	0	0	0	1	3.7
225.00	0	1	0	0	0	0	0	0	1	3.5
247.50	0	2	0	0	0	0	0	0	2	4.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	1	0	0	0	0	0	0	0	1	2.5
315.00	0	1	0	0	0	0	0	0	1	5.4
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	1	5	0	0	0	0	0	0	6	3.8

Hours of Calm = 0
Sums of this table: row totals = 6 and column totals = 6

TABLE 2.3-107

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C AUG.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	1	0	0	0	0	0	1	7.9
157.50	0	0	1	0	0	0	0	0	1	7.7
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	2	0	0	0	0	0	0	2	4.4
270.00	0	1	0	0	0	0	0	0	1	3.2
292.50	0	1	2	1	0	0	0	0	4	9.6
315.00	0	0	0	0	0	0	0	0	0	0.0
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	0	4	4	1	0	0	0	0	9	7.3

Hours of Calm = 0
Sums of this table: row totals = 9 and column totals = 9

TABLE 2.3-108

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D AUG.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	3	4	0	0	0	0	0	0	7	3.1
45.00	2	3	0	0	0	0	0	0	5	3.2
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	1	1	0	0	0	0	0	0	2	3.3
112.50	3	3	0	0	0	0	0	0	6	3.2
135.00	0	13	5	2	0	0	0	0	20	6.9
157.50	7	21	1	0	0	0	0	0	29	4.3
180.00	5	15	0	0	0	0	0	0	20	3.7
202.50	11	5	0	0	0	0	0	0	16	3.1
225.00	5	4	0	0	0	0	0	0	9	3.0
247.50	5	5	0	0	0	0	0	0	10	2.9
270.00	13	15	3	0	0	0	0	0	31	4.2
292.50	17	75	43	25	5	1	0	0	166	7.8
315.00	9	33	79	67	36	14	0	0	238	12.9
337.50	2	17	3	0	0	0	0	0	22	6.0
360.00	4	11	0	0	0	0	0	0	15	3.5
Column Sums	87	225	134	94	41	15	0	0	596	8.7

Hours of Calm = 2
Sums of this table: row totals = 596 and column totals = 596

TABLE 2.3-109

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS E AUG.

FREQUENCY TABLE

Mean Wind Direction	*1.5*	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*	*Row Sums*	*Row Avg.*
				Mean Wind Speed, mph						
22.50	*8*	*7*	*0*	*0*	*0*	*0*	*0*	*0*	*15*	*3.2*
45.00	*8*	*4*	*0*	*0*	*0*	*0*	*0*	*0*	*12*	*3.3*
67.50	*3*	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*4*	*3.5*
90.00	*5*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*6*	*2.5*
112.50	*7*	*2*	*3*	*0*	*0*	*0*	*0*	*0*	*12*	*4.2*
135.00	*5*	*11*	*6*	*1*	*0*	*0*	*0*	*0*	*23*	*5.9*
157.50	*7*	*18*	*2*	*0*	*0*	*0*	*0*	*0*	*27*	*4.4*
180.00	*5*	*4*	*0*	*0*	*0*	*0*	*0*	*0*	*9*	*3.4*
202.50	*2*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*4*	*3.0*
225.00	*4*	*4*	*1*	*0*	*0*	*0*	*0*	*0*	*9*	*4.3*
247.50	*5*	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*8*	*3.3*
270.00	*6*	*4*	*2*	*0*	*0*	*0*	*0*	*0*	*12*	*4.5*
292.50	*11*	*52*	*19*	*10*	*7*	*8*	*0*	*0*	*107*	*9.3*
315.00	*18*	*68*	*75*	*107*	*70*	*61*	*2*	*0*	*401*	*15.0*
337.50	*10*	*31*	*7*	*0*	*0*	*0*	*0*	*0*	*48*	*5.3*
360.00	*14*	*17*	*0*	*0*	*0*	*0*	*0*	*0*	*31*	*4.1*
Column Sums	*118*	*228*	*116*	*118*	*77*	*69*	*2*	*0*	*728*	*11.0*

Hours of Calm = 3
Sums of this table: row totals = 728 and column totals = 728

TABLE 2.3-110

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F AUG.

FREQUENCY TABLE

Mean Wind Direction	*1.5*	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*	*Row Sums*	*Row Avg.*
				Mean Wind Speed, mph						
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*1*	*7.6*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
135.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
157.50	*1*	*4*	*0*	*0*	*0*	*0*	*0*	*0*	*5*	*4.3*
180.00	*1*	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*4*	*4.4*
202.50	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.4*
225.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
247.50	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*1*	*9.0*
270.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
292.50	*1*	*2*	*2*	*3*	*6*	*9*	*0*	*0*	*23*	*20.3*
315.00	*2*	*1*	*5*	*5*	*14*	*22*	*6*	*0*	*55*	*24.3*
337.50	*2*	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*3*	*6.8*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*7*	*11*	*9*	*9*	*20*	*31*	*6*	*0*	*93*	*20.3*

Hours of Calm = 0
Sums of this table: row totals = 93 and column totals = 93

TABLE 2.3-111

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS G AUG.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
90.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
112.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
135.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
157.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
180.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
225.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
247.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
270.00	*0*	*3*	*0*	*1*	*0*	*0*	*0*	*0*	*4*	*6.7*
292.50	*0*	*1*	*0*	*0*	*2*	*3*	*0*	*0*	*6*	*20.1*
315.00	*0*	*0*	*0*	*1*	*2*	*0*	*1*	*0*	*4*	*24.0*
337.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
360.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
Column Sums	*0*	*4*	*0*	*2*	*4*	*3*	*1*	*0*	*14*	*17.4*

Hours of Calm = 0
Sums of this table: row totals = 14 and column totals = 14

TABLE 2.3-112

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A SEPT.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	1	0	0	0	0	0	0	1	4.4
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	1	0	0	0	0	0	0	0	1	2.4
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	2	0	0	0	0	0	0	2	6.5
315.00	0	0	0	0	0	3	0	0	3	28.7
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	1	3	0	0	0	3	0	0	7	15.1

Hours of Calm = 0
Sums of this table: row totals = 7 and column totals = 7

TABLE 2.3-113

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M, TEMP GRAD 76-10M STABILITY CLASS B SEPT.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	1	0	0	0	0	0	0	1	3.8
315.00	1	0	0	0	0	0	0	0	1	2.9
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	1	1	0	0	0	0	0	0	2	3.3

Hours of Calm = 0

Sums of this table: row totals = 2 and column totals = 2

TABLE 2.3-114

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C SEPT.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	0	1	0	0	0	0	0	0	1	7.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	2	0	0	0	0	0	0	0	2	2.2
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	1	2	2	0	0	0	0	0	5	6.1
315.00	0	0	1	0	0	0	0	0	1	11.2
337.50	0	0	0	1	0	0	0	0	1	16.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	3	3	3	1	0	0	0	0	10	6.9

Hours of Calm = 0
Sums of this table: row totals = 10 and column totals = 10

TABLE 2.3-115
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D SEPT.

FREQUENCY TABLE

Mean Wind Direction	*1.5*	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*	*Row Sums*	*Row Avg.*
				Mean Wind Speed, mph						
22.50	4	2	0	0	0	0	0	0	6	3.5
45.00	1	2	0	0	0	0	0	0	3	3.2
67.50	0	2	0	0	0	0	0	0	2	3.5
90.00	1	2	0	0	0	0	0	0	3	3.6
112.50	5	21	4	1	0	0	0	0	31	5.2
135.00	9	54	9	1	0	0	0	0	73	5.4
157.50	3	15	4	0	0	0	0	0	22	5.0
180.00	3	14	0	0	0	0	0	0	17	3.5
202.50	7	3	0	0	0	0	0	0	10	2.8
225.00	6	1	0	0	0	0	0	0	7	2.9
247.50	1	5	1	0	0	0	0	0	7	4.8
270.00	3	7	1	0	0	0	0	0	11	4.1
292.50	3	30	23	13	3	0	0	0	72	8.9
315.00	4	24	42	40	24	6	0	0	140	12.6
337.50	1	15	5	2	0	0	0	0	23	6.3
360.00	4	4	0	0	0	0	0	0	8	3.5
Column Sums	55	201	89	57	27	6	0	0	435	8.0

Hours of Calm = 0
Sums of this table: row totals = 435 and column totals = 435

TABLE 2.3-116

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS E SEPT.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	7	13	1	0	0	0	0	0	21	3.9
45.00	6	9	1	0	0	0	0	0	16	4.0
67.50	5	6	1	1	0	0	0	0	13	5.0
90.00	8	3	0	0	0	0	0	0	11	2.9
112.50	7	15	1	0	0	0	0	0	23	4.2
135.00	8	20	10	0	0	0	0	0	38	5.2
157.50	4	11	5	0	0	0	0	0	20	5.5
180.00	1	3	0	0	0	0	0	0	4	3.7
202.50	3	4	0	1	0	0	0	0	8	4.3
225.00	9	1	0	0	0	0	0	0	10	2.6
247.50	2	3	0	1	1	0	0	0	7	8.0
270.00	6	2	0	1	0	0	0	0	9	4.5
292.50	4	27	17	10	2	1	0	0	61	8.6
315.00	4	36	46	52	56	27	0	0	221	15.0
337.50	3	25	10	7	1	1	0	0	47	7.8
360.00	2	12	1	0	0	0	0	0	15	4.4
Column Sums	79	190	93	73	60	29	0	0	524	9.7

Hours of Calm = 2
Sums of this table: row totals = 524 and column totals = 524

TABLE 2.3-117

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F SEPT.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	4	2	0	0	0	0	0	0	6	3.3
45.00	3	3	0	0	0	0	0	0	6	3.5
67.50	2	0	0	0	0	0	0	0	2	1.5
90.00	4	1	0	0	0	0	0	0	5	2.6
112.50	2	5	0	0	0	0	0	0	7	4.1
135.00	2	3	1	0	0	0	0	0	6	5.3
157.50	1	9	0	0	0	0	0	0	10	5.3
180.00	1	5	0	0	0	0	0	0	6	4.3
202.50	2	0	0	0	0	0	0	0	2	1.7
225.00	2	1	0	0	0	0	0	0	3	3.3
247.50	1	2	0	0	0	0	0	0	3	4.0
270.00	1	0	1	0	0	0	0	0	2	5.2
292.50	2	7	1	1	3	0	0	0	14	9.7
315.00	2	14	40	1	28	38	0	0	153	17.2
337.50	1	5	6	1	1	0	0	0	14	9.2
360.00	3	5	0	0	0	0	0	0	8	4.2
Column Sums	33	62	49	33	32	8	0	0	247	12.8

Hours of Calm = 0
Sums of this table: row totals = 247 and column totals = 247

TABLE 2.3-118

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS G SEPT.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	1	1	0	0	0	0	0	0	2	4.4
45.00	2	1	0	0	0	0	0	0	3	4.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	2	1	0	0	0	0	0	0	3	2.9
135.00	1	6	0	0	0	0	0	0	7	4.0
157.50	0	1	0	0	0	0	0	0	1	3.1
180.00	1	0	0	0	0	0	0	0	1	1.8
202.50	1	0	0	0	0	0	0	0	1	2.5
225.00	1	0	0	0	0	0	0	0	1	1.8
247.50	1	1	0	0	0	0	0	0	2	4.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	1	2	1	1	3	0	0	8	17.8
315.00	1	4	7	9	4	10	1	0	36	17.8
337.50	1	0	0	0	1	0	0	0	2	11.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	12	16	9	10	6	13	1	0	67	13.1

Hours of Calm = 0
Sums of this table: row totals = 67 and column totals = 67

TABLE 2.3-119

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A OCT.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	0	0	0	0	0	0	0	0.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	1	0	0	0	0	0	0	0	1	2.8
202.50	0	1	0	0	0	0	0	0	1	4.9
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	0	0	0	0	0	0	0	0	0.0
315.00	0	0	0	0	0	0	0	0	0	0.0
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	1	1	0	0	0	0	0	0	2	3.8

Hours of Calm = 0
Sums of this table: row totals = 2 and column totals = 2

TABLE 2.3-120

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS B OCT.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	5	1	0	0	0	0	6	10.2
90.00	0	0	1	0	0	0	0	0	1	8.2
112.50	1	0	0	0	0	0	0	0	1	3.0
135.00	2	0	0	0	0	0	0	0	2	3.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	1	0	0	0	0	0	1	9.3
292.50	0	0	1	2	0	0	0	0	3	13.8
315.00	0	2	2	1	0	0	0	0	5	8.6
337.50	0	0	0	0	0	0	0	0	0	0.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	3	2	10	4	0	0	0	0	19	9.1

Hours of Calm = 0
Sums of this table: row totals = 19 and column totals = 19

TABLE 2.3-121

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C OCT.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	1	0	1	0	0	0	0	2	8.7
157.50	0	1	0	0	0	0	0	0	1	3.2
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	1	0	1	0	0	0	0	2	8.2
315.00	0	0	0	0	0	0	0	0	0	0.0
337.50	1	0	0	0	0	0	0	0	1	2.0
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	1	3	0	2	0	0	0	0	6	6.5

Hours of Calm = 0
Sums of this table: row totals = 6 and column totals = 6

TABLE 2.3-122

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D OCT.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	3	0	0	0	0	0	0	3	4.4
45.00	2	6	1	0	0	0	0	0	9	4.5
67.50	0	3	0	0	0	0	0	0	3	3.5
90.00	4	4	4	0	0	0	0	0	12	5.7
112.50	5	17	12	2	0	0	0	0	36	6.7
135.00	4	25	21	5	0	0	0	0	55	7.1
157.50	2	18	1	0	0	0	0	0	21	4.7
180.00	3	7	0	0	0	0	0	0	10	3.4
202.50	2	6	1	0	0	0	0	0	9	4.8
225.00	4	2	0	2	0	0	0	0	8	5.0
247.50	0	1	0	0	0	0	0	0	1	3.3
270.00	8	10	0	0	0	0	0	0	18	3.5
292.50	5	25	23	6	0	1	0	0	60	7.9
315.00	3	22	29	31	4	18	1	0	108	13.8
337.50	1	11	9	2	0	0	0	0	23	7.7
360.00	0	3	1	0	0	0	0	0	4	5.4
Column Sums	43	163	102	48	4	19	1	0	380	8.4

Hours of Calm = 2
Sums of this table: row totals = 380 and column totals = 380

TABLE 2.3-123

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS E OCT.

FREQUENCY TABLE

Mean Wind Direction				*Mean Wind Speed, mph*					*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*7*	*28*	*2*	*2*	*0*	*0*	*0*	*0*	*39*	*5.0*
45.00	*10*	*17*	*1*	*4*	*2*	*0*	*0*	*0*	*34*	*6.2*
67.50	*27*	*23*	*0*	*2*	*0*	*1*	*0*	*0*	*53*	*4.1*
90.00	*20*	*22*	*4*	*1*	*0*	*0*	*0*	*0*	*47*	*4.1*
112.50	*17*	*15*	*11*	*3*	*2*	*0*	*0*	*0*	*48*	*6.7*
135.00	*7*	*53*	*22*	*8*	*0*	*0*	*0*	*0*	*90*	*6.6*
157.50	*5*	*9*	*0*	*0*	*0*	*0*	*0*	*0*	*14*	*3.7*
180.00	*5*	*6*	*0*	*0*	*0*	*0*	*0*	*0*	*11*	*3.3*
202.50	*4*	*4*	*2*	*0*	*0*	*0*	*0*	*0*	*10*	*4.7*
225.00	*3*	*2*	*5*	*10*	*1*	*0*	*0*	*0*	*21*	*11.8*
247.50	*3*	*1*	*1*	*0*	*2*	*0*	*0*	*0*	*7*	*9.2*
270.00	*4*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*6*	*3.1*
292.50	*12*	*20*	*20*	*8*	*6*	*3*	*0*	*0*	*69*	*9.4*
315.00	*11*	*23*	*21*	*37*	*19*	*9*	*0*	*0*	*120*	*12.9*
337.50	*14*	*24*	*13*	*4*	*2*	*0*	*0*	*0*	*57*	*6.7*
360.00	*5*	*16*	*6*	*2*	*0*	*0*	*0*	*0*	*29*	*6.1*
Column Sums	*154*	*265*	*108*	*81*	*34*	*13*	*0*	*0*	*655*	*7.6*

Hours of Calm = 3
Sums of this table: row totals = 655 and column totals = 655

TABLE 2.3-124

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M, TEMP GRAD 76-10M STABILITY CLASS F OCT.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	4	4	2	0	0	0	0	0	10	4.9
45.00	3	1	1	0	0	0	0	0	5	4.4
67.50	5	5	0	0	1	0	0	0	11	5.1
90.00	4	1	0	1	0	0	0	0	6	5.2
112.50	5	12	1	0	0	0	0	0	18	4.3
135.00	3	10	5	1	0	0	0	0	19	6.2
157.50	3	1	0	0	0	0	0	0	4	2.4
180.00	2	0	0	0	0	0	0	0	2	2.1
202.50	0	1	0	0	0	0	0	0	1	4.0
225.00	0	2	1	5	3	0	0	0	11	13.5
247.50	0	1	0	1	1	0	0	0	3	12.4
270.00	3	0	1	0	0	0	0	0	4	4.1
292.50	3	5	0	4	2	3	0	0	27	11.2
315.00	10	11	14	15	15	7	0	0	72	12.8
337.50	3	10	7	7	0	0	0	0	27	8.7
360.00	3	4	4	0	0	0	0	0	11	6.3
Column Sums	51	68	46	34	22	10	0	0	231	9.1

Hours of Calm = 0

Sums of this table: row totals = 231 and column totals = 231

TABLE 2.3-125

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS G OCT.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	1	0	0	0	0	0	0	1	4.2
45.00	1	2	0	1	0	0	0	0	4	6.5
67.50	2	1	0	0	0	0	0	0	3	2.7
90.00	3	0	0	0	0	0	0	0	3	2.8
112.50	2	6	0	0	0	0	0	0	8	4.4
135.00	8	6	0	0	0	0	0	0	14	3.5
157.50	3	5	0	0	0	0	0	0	8	3.3
180.00	0	1	0	0	0	0	0	0	1	4.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	1	0	2	0	0	0	0	3	13.1
247.50	1	1	0	0	0	0	0	0	2	2.5
270.00	2	0	0	0	0	0	0	0	2	2.9
292.50	2	3	6	2	2	0	0	0	15	9.7
315.00	8	11	5	14	7	5	0	0	50	12.2
337.50	0	2	0	2	0	0	0	0	4	10.4
360.00	3	1	0	0	0	0	0	0	4	2.7
Column Sums	35	41	11	21	9	5	0	0	122	8.3

Hours of Calm = 0
Sums of this table: row totals = 122 and column totals = 122

TABLE 2.3-126

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A NOV.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	*0*	*1*	*2*	*0*	*0*	*0*	*0*	*0*	*3*	*6.9*
45.00	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*1*	*7.9*
67.50	*0*	*2*	*0*	*1*	*0*	*0*	*0*	*0*	*3*	*8.0*
90.00	*0*	*0*	*4*	*1*	*0*	*0*	*0*	*0*	*5*	*11.2*
112.50	*0*	*2*	*2*	*4*	*5*	*1*	*0*	*0*	*14*	*15.4*
135.00	*1*	*1*	*1*	*7*	*1*	*0*	*0*	*0*	*11*	*13.5*
157.50	*0*	*2*	*0*	*1*	*0*	*0*	*0*	*0*	*3*	*9.3*
180.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
202.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
225.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*3.6*
247.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
270.00	*0*	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*1*	*10.1*
292.50	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
315.00	*0*	*2*	*2*	*8*	*3*	*5*	*0*	*0*	*20*	*18.4*
337.50	*1*	*2*	*9*	*11*	*11*	*7*	*0*	*0*	*41*	*16.4*
360.00	*1*	*4*	*7*	*2*	*0*	*0*	*0*	*0*	*14*	*8.5*
Column Sums	*3*	*17*	*29*	*35*	*20*	*13*	*0*	*0*	*117*	*14.3*

Hours of Calm = 0
Sums of this table: row totals = 117 and column totals = 117

TABLE 2.3-127

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS B NOV.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.1*
45.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
67.50	*1*	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*2*	*7.2*
90.00	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*5.2*
112.50	*0*	*0*	*2*	*1*	*0*	*0*	*0*	*0*	*3*	*11.1*
135.00	*0*	*1*	*3*	*5*	*0*	*0*	*0*	*0*	*9*	*12.6*
157.50	*2*	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*5*	*4.2*
180.00	*1*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*3.6*
202.50	*1*	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*3.9*
225.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
247.50	*1*	*0*	*1*	*0*	*0*	*0*	*0*	*0*	*2*	*5.3*
270.00	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0.0*
292.50	*1*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1*	*2.9*
315.00	*2*	*2*	*1*	*3*	*3*	*0*	*0*	*0*	*11*	*12.0*
337.50	*1*	*0*	*2*	*3*	*1*	*2*	*0*	*0*	*9*	*15.9*
360.00	*0*	*3*	*5*	*1*	*0*	*0*	*0*	*0*	*9*	*9.2*
Column Sums	*11*	*12*	*15*	*13*	*4*	*2*	*0*	*0*	*57*	*10.1*

Hours of Calm = 0
Sums of this table: row totals = 57 and column totals = 57

TABLE 2.3-128

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C NOV.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	1	1	2	0	0	0	0	0	4	7.0
45.00	1	1	3	0	0	0	0	0	5	6.7
67.50	1	0	1	0	0	0	0	0	2	4.9
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	2	0	0	1	1	0	0	4	15.2
135.00	1	6	5	10	0	0	0	0	22	10.6
157.50	2	5	2	0	1	0	0	0	10	7.1
180.00	1	1	0	0	1	0	0	0	3	8.4
202.50	1	1	0	0	0	0	0	0	2	3.4
225.00	0	1	0	0	0	0	0	0	1	4.4
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	1	3	0	0	0	0	0	0	4	3.5
315.00	0	3	3	2	1	0	0	0	9	11.6
337.50	0	1	4	10	4	4	0	0	23	17.8
360.00	0	0	1	1	0	0	0	0	2	12.2
Column Sums	9	25	21	23	8	5	0	0	91	11.3

Hours of Calm = 0
Sums of this table: row totals = 91 and column totals = 91

TABLE 2.3-129

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D NOV.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph*								*Row Sums*	*Row Avg.*
	1.5	*5.1*	*9.6*	*15.1*	*21.1*	*29.6*	*40.1*	*50.1*		
22.50	*2*	*14*	*9*	*1*	*0*	*1*	*0*	*0*	*27*	*7.4*
45.00	*6*	*12*	*4*	*0*	*1*	*0*	*0*	*0*	*23*	*5.7*
67.50	*6*	*4*	*9*	*0*	*0*	*0*	*0*	*0*	*19*	*5.8*
90.00	*5*	*8*	*3*	*0*	*0*	*0*	*0*	*0*	*16*	*4.5*
112.50	*0*	*5*	*1*	*5*	*4*	*3*	*0*	*0*	*18*	*15.3*
135.00	*2*	*17*	*16*	*9*	*1*	*2*	*0*	*0*	*47*	*9.6*
157.50	*4*	*20*	*2*	*1*	*2*	*1*	*0*	*0*	*30*	*7.4*
180.00	*1*	*2*	*2*	*0*	*0*	*0*	*0*	*0*	*5*	*5.5*
202.50	*2*	*3*	*4*	*1*	*1*	*0*	*0*	*0*	*11*	*8.5*
225.00	*2*	*0*	*2*	*2*	*1*	*0*	*0*	*0*	*7*	*11.2*
247.50	*0*	*2*	*0*	*0*	*0*	*0*	*0*	*0*	*2*	*4.5*
270.00	*2*	*1*	*2*	*0*	*0*	*0*	*0*	*0*	*5*	*5.7*
292.50	*2*	*6*	*6*	*2*	*2*	*0*	*0*	*0*	*18*	*10.0*
315.00	*0*	*3*	*24*	*38*	*24*	*3*	*0*	*0*	*92*	*15.3*
337.50	*1*	*5*	*13*	*22*	*10*	*0*	*0*	*0*	*51*	*13.5*
360.00	*2*	*5*	*26*	*12*	*2*	*0*	*0*	*0*	*47*	*10.5*
Column Sums	*37*	*107*	*123*	*93*	*48*	*10*	*0*	*0*	*418*	*10.7*

Hours of Calm = 0
Sums of this table: row totals = 418 and column totals = 418

TABLE 2.3-130

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS E NOV.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	12	20	9	1	0	0	0	0	42	5.4
45.00	22	33	10	1	0	0	0	0	66	4.6
67.50	20	14	5	7	0	0	0	0	46	5.7
90.00	17	13	2	1	0	0	0	0	33	4.1
112.50	9	18	0	2	1	1	0	0	31	6.1
135.00	5	28	5	3	0	0	0	0	41	6.1
157.50	3	14	1	2	0	0	0	0	20	5.4
180.00	1	2	0	0	0	0	0	0	3	3.6
202.50	2	0	0	0	0	0	0	0	2	2.5
225.00	1	4	1	1	1	1	0	0	9	10.4
247.50	1	2	1	0	0	0	0	0	4	5.0
270.00	1	3	2	0	0	0	0	0	6	6.6
292.50	5	12	3	3	5	0	0	0	28	9.2
315.00	1	21	24	28	16	7	0	0	97	13.3
337.50	3	19	13	18	7	1	0	0	61	11.0
360.00	7	23	14	6	3	0	0	0	53	7.6
Column Sums	110	226	90	73	33	10	0	0	542	7.9

Hours of Calm = 0
Sums of this table: row totals = 542 and column totals = 542

TABLE 2.3-131

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F NOV.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	3	5	0	0	0	0	0	0	8	3.7
45.00	4	8	1	0	0	0	0	0	13	4.2
67.50	5	4	1	0	0	0	0	0	0	4.1
90.00	9	2	0	0	0	0	0	0	11	2.9
112.50	8	7	0	0	0	0	0	0	15	3.2
135.00	3	9	5	0	0	0	0	0	17	5.3
157.50	4	0	1	0	0	0	0	0	5	4.0
180.00	0	2	0	0	0	0	0	0	2	4.8
202.50	1	0	0	0	0	0	0	0	1	2.7
225.00	2	1	0	0	0	0	0	0	3	2.5
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	2	2	2	0	0	0	0	0	6	5.1
315.00	2	9	9	8	6	1	0	0	35	11.6
337.50	1	3	1	1	2	0	0	0	8	10.4
360.00	1	4	1	0	0	0	0	0	6	4.5
Column Sums	45	56	21	9	8	1	0	0	140	6.3

Hours of Calm = 0
Sums of this table: row totals = 140 and column totals = 140

TABLE 2.3-132

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS G NOV.

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	1	1	0	0	0	0	0	0	2	2.8
67.50	0	1	0	0	0	0	0	0	1	3.3
90.00	1	1	0	0	0	0	0	0	2	3.0
112.50	4	1	0	0	0	0	0	0	5	2.7
135.00	2	7	1	1	0	0	0	0	11	5.3
157.50	0	1	0	0	0	0	0	0	1	4.1
180.00	1	0	0	0	0	0	0	0	1	2.2
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	3	0	0	0	0	0	0	0	3	2.3
247.50	0	1	0	0	0	0	0	0	1	4.7
270.00	1	0	0	0	0	0	0	0	1	2.2
292.50	0	2	2	1	0	0	0	0	5	9.0
315.00	1	7	2	3	1	0	0	0	14	9.0
337.50	0	1	0	0	0	0	0	0	1	5.9
360.00	0	0	0	0	0	0	0	0	0	0.0
Column Sums	14	23	5	5	1	0	0	0	48	5.9

Hours of Calm = 0
Sums of this table: row totals = 48 and column totals = 48

TABLE 2.3-133

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS A DEC.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	2	0	0	0	0	0	2	8.7
45.00	0	1	0	0	0	0	0	0	1	6.8
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	1	0	0	0	0	0	1	9.5
112.50	0	0	0	1	0	1	0	0	2	20.3
135.00	0	0	2	0	0	0	0	0	2	10.7
157.50	1	0	0	0	0	0	0	0	1	3.0
180.00	0	1	0	1	0	0	0	0	2	8.2
202.50	0	0	0	1	0	0	0	0	1	12.7
225.00	0	0	0	1	0	0	0	0	1	12.1
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	1	0	0	0	0	1	15.1
292.50	0	1	0	0	0	2	0	0	3	18.6
315.00	0	0	0	0	0	2	0	0	2	29.4
337.50	0	2	0	1	0	0	0	0	3	9.0
360.00	0	0	2	0	0	0	0	0	2	8.3
Column Sums	1	5	7	6	0	5	0	0	24	13.1

Hours of Calm = 0
Sums of this table: row totals = 24 and column totals = 24

TABLE 2.3-134

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS B DEC.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	1	0	0	0	0	0	1	8.6
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	1	0	0	0	0	0	0	1	5.0
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	1	0	0	0	1	20.3
292.50	0	0	1	0	0	0	0	0	1	7.5
315.00	0	0	0	0	0	1	0	0	1	25.3
337.50	0	0	1	2	0	0	0	0	3	14.4
360.00	0	0	1	0	0	0	0	0	1	10.9
Column Sums	0	1	4	2	1	1	0	0	9	13.4

Hours of Calm = 0
Sums of this table: row totals = 9 and column totals = 9

TABLE 2.3-135

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS C DEC.
HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED
FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	0	0	0	0	0	0	0	0	0	0.0
90.00	0	0	1	0	0	0	0	0	1	7.5
112.50	0	0	0	0	0	0	0	0	0	0.0
135.00	0	0	1	1	0	0	0	0	2	11.5
157.50	0	0	0	0	0	0	0	0	0	0.0
180.00	0	0	0	0	1	0	0	0	1	19.3
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	0	1	0	0	0	0	0	1	10.8
315.00	1	0	0	0	0	0	0	0	1	2.0
337.50	0	0	4	3	0	0	0	0	7	12.6
360.00	0	2	2	0	0	0	0	0	4	6.4
Column Sums	1	2	9	4	1	0	0	0	17	10.4

Hours of Calm = 0
Sums of this table: row totals = 17 and column totals = 17

TABLE 2.3-136

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS D DEC.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	10	21	4	0	0	0	0	35	8.8
45.00	0	7	12	5	0	0	0	0	24	9.3
67.50	0	7	6	2	0	0	0	0	15	8.0
90.00	1	10	0	1	0	0	0	0	12	5.5
112.50	1	7	7	14	5	5	0	0	39	14.6
135.00	0	7	8	8	6	4	0	0	33	14.0
157.50	0	16	4	5	1	3	0	0	29	10.4
180.00	1	8	2	0	0	0	0	0	11	5.1
202.50	1	4	0	0	0	0	0	0	5	4.4
225.00	0	1	0	0	0	0	0	0	1	5.0
247.50	2	2	0	0	0	0	0	0	4	3.5
270.00	2	4	0	0	0	0	0	0	6	4.7
292.50	1	5	2	2	0	1	0	0	11	9.2
315.00	2	2	6	33	17	8	0	0	68	16.9
337.50	2	6	13	22	11	1	0	0	55	13.2
360.00	1	18	22	11	0	0	0	0	2	8.7
Column Sums	14	114	103	107	40	22	0	0	400	11.5

Hours of Calm = 0
Sums of this table: row totals = 400 and column totals = 400

TABLE 2.3-137

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS E DEC.

FREQUENCY TABLE

Mean Wind Direction	*Mean Wind Speed, mph* 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	*Row Sums*	*Row Avg.*
22.50	*6*	*50*	*17*	*9*	*1*	*0*	*0*	*0*	*83*	*6.7*
45.00	*10*	*49*	*21*	*2*	*0*	*0*	*0*	*0*	*82*	*5.8*
67.50	*22*	*22*	*19*	*11*	*2*	*0*	*0*	*0*	*76*	*6.8*
90.00	*19*	*25*	*6*	*2*	*1*	*0*	*0*	*0*	*53*	*4.8*
112.50	*15*	*30*	*9*	*6*	*8*	*4*	*0*	*0*	*72*	*8.7*
135.00	*4*	*30*	*21*	*2*	*0*	*5*	*0*	*0*	*62*	*8.6*
157.50	*6*		*2*	*0*	*0*	*1*	*0*	*0*	*23*	*5.9*
180.00	*4*	*9*	*2*	*0*	*0*	*0*	*0*	*0*	*15*	*4.4*
202.50	*2*	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*75*	*3.3*
225.00	*3*	*5*	*0*	*0*	*1*	*0*	*0*	*0*	*9*	*5.5*
247.50	*0*	*3*	*0*	*0*	*0*	*0*	*0*	*0*	*3*	*4.9*
270.00	*1*	*6*	*1*	*0*	*0*	*0*	*0*	*0*	*8*	*4.8*
292.50	*3*	*10*	*3*	*7*	*3*	*2*	*0*	*0*	*28*	*11.0*
315.00	*4*	*24*	*26*	*33*	*31*	*1*	*0*	*0*	*119*	*13.1*
337.50	*4*	*36*	*36*	*33*	*3*	*0*	*1*	*0*	*113*	*10.0*
360.00	*9*	*39*	*29*	*12*	*0*	*0*	*0*	*0*	*89*	*7.3*
Column Sums	*112*	*355*	*192*	*117*	*50*	*13*	*1*	*0*	*840*	*8.3*

Hours of Calm = 0
Sums of this table: row totals = 840 and column totals = 840

TABLE 2.3-138

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS F DEC.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	2	2	1	0	0	0	0	0	5	5.3
45.00	4	1	1	0	0	0	0	0	6	3.8
67.50	4	1	0	0	0	0	0	0	5	2.8
90.00	7	2	1	0	0	0	0	0	10	3.6
112.50	11	11	0	0	0	0	0	0	22	3.1
135.00	6	17	4	0	0	0	2	0	29	6.9
157.50	3	8	1	0	0	0	0	0	12	4.6
180.00	2	0	0	0	0	0	0	0	2	2.8
202.50	2	0	0	0	0	0	0	0	2	2.4
225.00	0	0	0	0	0	0	0	0	0	0.0
247.50	0	1	0	0	0	0	0	0	1	6.3
270.00	1	0	0	0	0	0	0	0	1	3.0
292.50	2	2	0	1	1	0	0	0	6	8.2
315.00	1	5	8	12	10	0	0	0	36	13.8
337.50	0	7	4	5	0	0	0	0	16	9.6
360.00	1	5	3	0	0	0	0	0	9	6.4
Column Sums	46	62	23	18	11	0	2	0	162	7.4

Hours of Calm = 0
Sums of this table: row totals = 162 and column totals = 162

TABLE 2.3-139

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE - MAY 1973 - APRIL 1975 WIND DATA 10M,
TEMP GRAD 76-10M STABILITY CLASS G DEC.

FREQUENCY TABLE

Mean Wind Direction	Mean Wind Speed, mph 1.5	5.1	9.6	15.1	21.1	29.6	40.1	50.1	Row Sums	Row Avg.
22.50	0	0	0	0	0	0	0	0	0	0.0
45.00	0	0	0	0	0	0	0	0	0	0.0
67.50	1	0	0	0	0	0	0	0	1	2.9
90.00	0	0	0	0	0	0	0	0	0	0.0
112.50	0	2	0	0	0	0	0	0	2	5.3
135.00	1	3	1	0	0	0	0	0	5	5.0
157.50	2	0	0	0	0	0	0	0	2	2.6
180.00	0	0	0	0	0	0	0	0	0	0.0
202.50	0	0	0	0	0	0	0	0	0	0.0
225.00	1	0	1	0	0	0	0	0	2	5.4
247.50	0	0	0	0	0	0	0	0	0	0.0
270.00	0	0	0	0	0	0	0	0	0	0.0
292.50	0	2	0	0	0	0	0	0	2	3.3
315.00	0	8	2	1	1	0	0	0	12	8.0
337.50	0	3	1	1	0	0	0	0	5	8.1
360.00	1	0	1	0	0	0	0	0	2	5.7
Column Sums	6	18	6	2	1	0	0	0	33	6.3

Hours of Calm = 0
Sums of this table: row totals = 33 and column totals = 33

TABLE 2.3-141

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

RANGES OF STABILITY CLASSIFICATION PARAMETERS
FOR EACH STABILITY CATEGORY AT DCPP SITE

Pasquil Stability Class[a]	*σ_θ Range, (deg)*	*ΔT Range, (°C/100m)*	*R_i Range* $\frac{g}{\theta} = \frac{\theta_{76m} - \theta_{10m}}{U^2}$
A	$\sigma_\theta \geq 22.5$	*< -1.9*	*< -0.02*
B	$22.5 > \sigma_\theta \geq 17.5$	*-1.9 to -1.7*	*-0.02 to- .01*
C	$17.5 > \sigma_\theta \geq 12.5$	*-1.7 to -1.5*	*-0.01 to -.001*
D	$12.5 > \sigma_\theta \geq 7.5$	*-1.5 to -0.5*	*-0.001 to +0.005*
E	$7.5 > \sigma_\theta \geq 3.8$	*-0.5 to +1.5*	*+0.005 to +0.02*
F	$3.8 > \sigma_\theta \geq 2.1$	*+1.5 to +4.0*	*+0.02 to +0.07*
G	$2.1 > \sigma_\theta$	*≥ +4.0*	*≥ +0.07*

(a) See Reference 17, Section 2.3.9.

TABLE 2.3-142

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

SUMMARY OF METEOROLOGICAL DATA
FOR DIFFUSION EXPERIMENTS AT DCPP SITE

Trial No.	*Date*	*Release Time (Local Time)*	*h (ft)*	*Wind Dir. (deg)*	*m (mph)*	*H m (ft)*	*Δ T 250 30 (°F)*
				Trials with Northwesterly Flow			
1	*11-20-68*	*1552-1652*	*250*	*304*	*11*	*1000*	*11.7*
2	*11-21-68*	*1411-1510*	*250*	*313*	*15*	*800*	*3.1*
3	*11-22-68*	*1540-1632*	*250*	*303*	*20*	*400*	*5.9*
4	*11-24-68*	*1036-1135*	*250*	*310*	*19*	*2500*	*-2.0*
9	*03-04-69*	*1110-1210*	*250*	*294*	*16*	*800*	*-3.0*
10	*03-06-69*	*1220-1320*	*250*	*311*	*26*	*2400*	*-3.0*
11	*03-07-69*	*1100-1200*	*250*	*297*	*16*	*4600*	*-4.2*
12	*03-08-69*	*1418-1518*	*250*	*306*	*14*	*1400*	*-2.0*
15	*05-20-69*	*1100-1200*	*250*	*305*	*15*	*1000*	*-0.2*
16	*05-20-69*	*1445-1545*	*250*	*306*	*18*	*600*	*-0.6*
17	*05-21-69*	*1240-1340*	*250*	*308*	*24*	*800*	*+1.5*
18	*05-22-69*	*1230-1330*	*250*	*310*	*20*	*1000*	*+0.3*
20	*07-15-69*	*1412-1512*	*250*	*305*	*27*	*600*	*+4.5*
22	*07-16-69*	*1500-1600*	*250*	*304*	*16*	*500*	*+1.0*
24	*07-24-69*	*1238-1338*	*250*	*305*	*24*	*600*	*+1.7*
25	*07-25-69*	*1054-1155*	*250*	*306*	*20*	*1500*	*+0.1*
				Trials with Southeasterly Flow			
6	*01-12-69*	*0940-1040*	*250*	*133*	*15*	*2500*	*-1.3*
8	*02-22-69*	*1300-1400*	*250*	*168*	*9*	*2500*	*-2.7*
13	*04-02-69*	*0930-1030*	*250*	*146*	*10*	*1500*	*-0.4*
14	*04-02-69*	*1300-1400*	*250*	*148*	*9*	*2500*	*-2.2*
23	*07-17-69*	*0205-0305*	*250*	*131*	*8*	*500*	*+1.9*
30	*10-15-69*	*0742-0842*	*25*	*143*	*6*	*2500*	*+0.4*
				Trials with Light and Variable Winds[a]			
19	*07-15-69*	*0201-0301*	*250*				*-0.8*
21	*07-16-69*	*0433-0500*	*250*				*0.9*
26	*09-29-69*	*0037-0137*	*25*				*-1.1*
27	*09-30-69*	*0220-0322*	*25*				*+0.4*
28	*10-01-69*	*0250-0350*	*25*				*+0.3*

[a] *Wind speed of 2 mph was assumed.*

TABLE 2.3-144

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

DCPP SITE NIGHTTIME P-G STABILITY CATEGORIES BASED ON σ_θ

If the σ_θ Stability Class is:	And if the 10m Wind Speed, u is: m/s	mi/hr	The Stability Class for the σ_z is:
A	$u<2.9$	$u<6.4$	*F*
	$2.9 \leq u<3.6$	$6.4 \leq u<7.9$	*E*
	$3.6 \leq u$	$7.9 \leq u$	*D*
B	$u<2.4$	$u<5.3$	*F*
	$2.4 \leq u<3.0$	$5.3 \leq u<6.6$	*E*
	$3.0<u$	$6.6 \leq u$	*D*
C	$u<2.4$	$u<5.3$	*E*
	$2.4<u$	$5.3<u$	*D*
D, E, F, or G	*wind speed not considered*		

TABLE 2.4-1

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

PROBABLE MAXIMUM PRECIPITATION (PMP) AS A FUNCTION OF DURATION AT DCPP SITE AS DETERMINED FROM USWB HMR NO. 36

Duration, hours	*PMP, inches*
1	*4.3*
3	*7.1*
6	*9.1*
12	*12.0*
18	*14.8*
24	*16.6*

TABLE 2.5-1

LISTING OF EARTHQAKES WITHIN 75 MILES OF THE DIABLO CANYON POWER PLANT SITE
SELECTED EARTHQUAKES

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
-?/-?/1800	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
03/25/1806	08--?--?	34.50	119.67	D			F	VIII AT SANTA BARBARA.
12/21/1812	18--?--?	34.50	120.00	D			F	VIII AT SAN FERNANDO.
12/21/1812	19--?--?	34.50	120.00	D			F	IX AT SAN FERNANDO.
01/18/1815	-?--?--?	34.50	119.67	D			F	SANTA BARBARA; 5 SHOCKS.
01/30/1815	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
07/08/1815	-?--?--?	34.50	119.67	D			F	SANTA BARBARA; 6 SHOCKS ON THE EIGHTH AND NINTH.
-?/-?/1830	-?--?--?	35.25	120.67	D			F	VIII AT SAN LUIS OBISPO.
07/03/1841	-?--?--?	36.30	122.30		6.3			(CALTECH FILE)
06/13/1851	-?--?--?	35.25	120.67	D			F	V AT SAN LUIS OBISPO.
10/26/1852	-?--?--?	35.67	121.17	D			F	X AT SAN SIMEON; 11 SHOCKS.
12/17/1852	-?--?--?	35.25	120.67	D			F	IX AT SAN LUIS OBISPO; 2 SHOCKS.
01/10/1853	-?--?--?	35.25	120.67	D			F	DANA RANCHO.
01/29/1853	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
02/01/1853	21--?--?	35.67	121.17	D			F	VIII AT SAN SIMEON.
02/14/1853	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
03/01/1853	-?--?--?	34.50	119.67	D			F	V AT SAN LUIS OBISPO.
04/20/1854	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
04/29/1854	-?--?--?	34.50	119.67	D			F	III AT SANTA BARBARA.
05/03/1854	13-10--?	34.50	119.67	D			F	SANTA BARBARA; 3 SEVERE SHOCKS.
05/13/1854	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
05/29/1854	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
05/31/1854	12-50--?	34.50	119.67	D			F	VI AT SANTA BARBARA; 3 SHOCKS.
01/14/1855	02-30--?	35.75	120.67	D			F	SAN BENITO AND SAN MIGUEL.
06/25/1855	22--?--?	34.50	119.67	D			F	V AT SANTA BARBARA.
01/08/1857	14--?--?	34.50	119.67	D			F	SANTA BARBARA.
01/08/1857	17--?--?	34.50	119.67	D			F	SANTA BARBARA.
01/08/1857	18--?--?	34.50	119.67	D			F	SANTA BARBARA.
01/09/1857	07-20--?	34.50	119.67	D			F	IX AT SANTA BARBARA.
01/21/1857	-?--?--?	36.50	121.08	D			F	III AT A POINT NORTHWEST OF SAN BENITO.
03/14/1857	23--?--?	34.50	119.67	D			F	V AT MONTECITO AND SANTA BARBARA.
09/02/1858	-?--?--?	34.50	119.67	D			F	V AT SANTA BARBARA.
04/03/1860	04--?--?	36.50	121.08	D			F	VI AT SAN JOSE.
04/17/1860	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
-?/-?/1862	-?--?--?	34.42	119.63	D			F	VIII AT GOLETA.
09/13/1869	-?--?--?	35.25	120.67	D			F	V AT SAN LUIS OBISPO.
09/14/1869	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
12/15/1869	-?--?--?	35.25	120.67				F	V AT SAN LUIS OBISPO.
02/06/1872	-?--?--?	34.50	119.67	D			F	SANTA BARBARA; FIRST SINCE APRIL 1860.
11/07/1875	-?--?--?	36.50	121.08	D			F	V IN SAN BENITO COUNTY.
12/21/1875	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
05/10/1876	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
05/30/1877	-?--?--?	35.67	120.67	D			F	V AT PASO ROBLES.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/24/1877	07-30--?	34.50	119.67	D			F	SANTA BARBARA.
01/08/1878	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
11/13/1880	06-30--?	34.50	119.67	D			F	SANTA BARBARA.
02/02/1881	-?--?--?	36.37	121.67	D			F	III AT SALINAS.
08/31/1881	03--?--?	34.50	119.67	D			F	III AT SANTA BARBARA.
09/13/1883	22-30--?	34.50	119.67	D			F	IV AT SANTA BARBARA.
08/03/1884	-?--?--?	34.50	119.67	D			F	III AT SANTA BARBARA; NIGHT.
08/04/1884	09--?--?	34.50	119.67	D			F	III AT SANTA BARBARA; 3 SHOCKS.
03/31/1885	-?--?--?	36.30	121.00		7.0			(CALTECH FILE)
04/07/1885	10--?--?	34.50	119.67	D			F	SANTA BARBARA AND SAN BUENAVENTURA.
04/09/1885	-?--?--?	35.58	121.08	D			F	CAMBRIA.
04/12/1885	04-05--?	36.25	120.80	D			F	IX IN CENTRAL CALIFORNIA; FELT OVER AN AREA OF 125,000 SQ. MI.- EPICENTER PROBABLY EAST OF KING CITY.
04/12/1885	11--?--?	36.33	119.67	D			F	HANFORD.
07/09/1885	09-15--?	34.50	119.67	D			F	V AT SANTA BARBARA.
07/09/1885	16-15--?	34.50	119.67	D			F	V AT SANTA BARBARA; 5 EARTHQUAKES.
10/03/1888	20-52--?	35.75	120.67	D			F	III AT SAN MIGUEL.
10/03/1888	21-02--?	35.75	120.67	D			F	VI AT SAN MIGUEL.
10/04/1888	-?--?--?	35.67	120.67	D			F	PASO ROBLES.
05/01/1889	19-55--?	34.67	120.42	D			F	SUSANVILLE.
05/26/1889	15-13--?	36.50	121.42	D			F	GONZALES, SAN FRANCISCO, AND SANTA CRUZ; RECORDED AT MT. HAMILTON.
07/10/1889	-?--?--?	35.17	120.58	D			F	ARROYO GRANDE; SHOCKS FOR SEVERAL DAYS.
09/30/1889	20-17--?	36.50	119.58	D			F	KINGSBURG.
01/-?/1890	23-30--?	34.50	119.67	D			F	SANTA BARBARA.
11/13/1892	-?--?--?	36.30	122.00		6.0			(CALTECH FILE)
05/19/1893	-?-35--?	34.17	119.50	D			F	VII FELT FROM SAN DIEGO TO LOMPOC, INLAND TO SAN BERNADINO. MOST SEVERE SE OF VENTURA. POSSIBLY OF SUBMARINE ORIGIN OFF THE COAST OF VENTURA COUNTY
06/01/1893	12--?--?	34.50	119.67	D			F	VII AT NORDHOFF (OJAI), SANTA BARBARA, AND VENTURA.
06/01/1893	12--?--?	34.50	119.67	D			F	NORDHOFF, SANTA BARBARA, AND VENTURA.
06/01/1893	12-10--?	34.50	119.67	D			F	NORDHOFF, SANTA BARBARA, AND VENTURA.
12/06/1893	04-56--?	35.67	121.33	D			F	PIEDRAS BLANCAS LIGHTHOUSE.
07/27/1895	-?-10--?	34.50	119.67	D			F	SANTA BARBARA.
12/24/1895	05-30--?	34.50	119.67	D			F	SANTA BARBARA.
06/24/1897	14-10--?	34.50	119.67	D			F	SANTA BARBARA.
07/18/1897	-?--?--?	34.50	119.67	D			F	CASTLE PINCKNEY.
07/20/1897	07-45--?	34.50	119.67	D			F	SANTA BARBARA.
05/30/1898	03-03--?	34.50	119.67	D			F	SANTA BARBARA.
06/04/1898	06-20--?	34.67	120.08	D			F	LOS OLIVOS; FELT THROUGHOUT THE SANTA YNEZ VALLEY; AT SANTA BARBARA THE HEAVIEST FOR SOME YEARS.
02/08/1899	04-55--?	36.33	121.92	D			F	POINT SUR LIGHT STATION.
06/05/1899	-?--?--?	35.83	120.83	D			F	BRADLEY.
06/25/1899	-?--?--?	35.75	120.67	D			F	SAN MIGUEL.
06/09/1900	-?--?--?	36.00	120.92	D			F	SAN ARDO.
10/18/1900	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
03/03/1901	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/03/1901	07-45--?	36.08	120.58	D			F	IX AT STONE CANYON - SURFACE CRACKS IN THE GROUND; ALSO FELT AT ADELAIDA, ESTRELLA, PARKFIELD, PASO ROBLES, PORTERVILLE, SAN JOSE, SAN LUIS OBISPO, AND SAN MIGUEL.
03/05/1901	-?--?--?	35.67	120.67	D			F	PASO ROBLES.
03/06/1901	-?--?--?	36.00	120.92	D			F	SAN ARDO AND SAN LUIS OBISPO.
06/03/1901	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/30/1901	19--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
08/14/1901	11-11--?	35.42	120.92	D			F	CAYUCOS, HOLLISTER, SALINAS, SAN LUIS OBISPO, AND SANTA CRUZ.
02/07/1902	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
02/09/1902	15--?--?	34.50	119.67	D			F	PINE CREST, SAN LUIS OBISPO, SANTA BARBARA, AND VENTURA.
04/06/1902	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/21/1902	-?--?--?	34.75	120.00	D			F	PINE CREST.
07/28/1902	06-57--?	34.75	120.25	D			F	IX AT LOMPOC AND LOS ALAMOS; CONFINED TO THE NORTHERN PART OF SANTA BARBARA COUNTY.
07/28/1902	13--8--?	35.25	120.67	D			F	SAN LUIS OBISPO; AFTERSHOCK OF 06-57-?.
07/31/1902	09-20--?	34.75	120.25	D			F	IX AT LOS ALAMOS AND SURROUNDING COUNTRY; FISSURES, CRACKS IN THE GROUND, AND LANDSLIDES.
08/01/1902	-?--?--?	34.75	120.25	D			F	LOS ALAMOS. SEVERAL SHOCKS.
08/01/1902	03-30--?	34.75	120.25	D			F	VIII AT LOS ALAMOS.
08/02/1902	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
08/03/1902	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
08/04/1902	10 -5--?	34.75	120.25	D			F	LOS ALAMOS.
08/04/1902	11-18--?	34.75	120.25	D			F	LOS ALAMOS.
08/04/1902	12-15--?	34.75	120.25	D			F	LOS ALAMOS.
08/04/1902	21-29--?	34.75	120.25	D			F	LOS ALAMOS.
08/04/1902	23-40--?	34.75	120.25	D			F	LOS ALAMOS.
08/05/1902	-?-55--?	34.75	120.25	D			F	LOS ALAMOS.
08/10/1902	-?--?--?	34.75	120.25	D			F	LOS ALAMOS; DISTINCT EARTHQUAKE DETONATION AND TREMOR.
08/10/1902	10-40--?	34.75	120.25	D			F	LOS ALAMOS; HEAVY DETONATION FOLLOWED BY TREMBLING.
08/10/1902	22-40--?	34.50	119.67	D			F	SANTA BARBARA.
08/14/1902	10-15--?	34.75	120.25	D			F	LOS ALAMOS.
08/14/1902	11-05--?	34.75	120.25	D			F	LOS ALAMOS.
08/14/1902	11-20--?	34.75	120.25	D			F	LOS ALAMOS; SHOOK GROUND VIOLENTLY.
08/14/1902	21-50--?	34.75	120.25	D			F	LOS ALAMOS.
08/14/1902	23-50--?	34.75	120.25	D			F	LOS ALAMOS.
08/28/1902	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
08/31/1902	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
09/11/1902	05-30--?	34.25	120.25	D			F	V AT LOS ALAMOS.
10/21/1902	21-45--?	34.75	120.25	D			F	LOMPOC AND LOS ALAMOS.
10/21/1902	22-15--?	34.75	120.25	D			F	LOMPOC AND LOS ALAMOS.
10/22/1902	10--?--?	34.75	120.25	D			F	LOS ALAMOS.
12/12/1902	-?--?--?	34.75	120.25	D			F	VIII AT LOS ALAMOS -3 SHOCKS IN 5 MINUTES; FELT THROUGHOUT THE NORTHERN PART OF SANTA BARBARA COUNTY, ESPECIALLY AT LOMPOC, LOS ALAMOS, SAN LUIS OBISPO, SANTA BARBARA, AND SANTA MARIA.
01/11/1903	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/07/1903	-?--?--?	36.50	121.42	D			F	GONZALES.
03/24/1903	-?--?--?	36.50	121.42	D			F	GONZALES AND SANTA MARGARITA.
04/24/1903	-?--?--?	35.42	120.58	D			F	SANTA MARGARITA.
07/29/1903	07-13--?	35.67	121.33	D			F	V AT POINT PIEDRAS BLANCAS LIGHTHOUSE.
07/29/1903	10-30--?	35.67	121.33	D			F	POINT PIEDRAS BLANCAS LIGHTHOUSE.
08/24/1903	-?--?--?	34.67	120.08	D			F	LOS OLIVOS.
01/22/1904	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
01/23/1904	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
09/10/1904	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
05/26/1905	05-49--?	35.25	120.67	D			F	LOS GATOS, SALINAS, SAN FRANCISCO, SAN LUIS OBISPO, SANTA CRUZ AND SOLEDAD.
07/06/1906	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/22/1906	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
08/01/1906	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
12/07/1906	06-40--?	35.67	121.33	D			F	VII AT SAN LUIS OBISPO AND SANTA MARIA; DURATION 30 SECONDS, FOLLOWED BY SECOND SHOCK HALF AN HOUR LATER.
+12/08/1906	06-55--?	35.75	120.67	D			F	SAN MIGUEL.
06/19/1907	12--?--?	36.17	120.67	D			F	PRIEST VALLEY.
07/02/1907	18-10--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/21/1907	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/29/1907	05-10--?	34.75	120.00	D			F	PINE CREST.
08/-?/1907	-?--?--?	34.75	120.00	D			F	PINE CREST AND SANTA BARBARA.
12/27/1907	09-15--?	34.50	119.67	D			F	SANTA BARBARA; ALSO FELT AT VENTURA; REPORTED FROM OJAI AND PINE CREST.
04/27/1908	10-50--?	36.00	121.17	D			F	JOLON, PASO ROBLES, PRIEST VALLEY, SAN LUIS OBISPO, SANTA MARGARETA, AND SAN MIGUEL.
05/19/1908	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
09/16/1908	-?--?--?	36.17	120.67	D			F	PRIEST VALLEY.
11/-?/1908	19-30--?	36.17	120.67	D			F	PRIEST VALLEY.
01/23/1909	14-58--?	34.50	119.67	D			F	PINE CREST AND SANTA BARBARA.
04/10/1909	-?--?--?	34.50	119.67	D			F	MONO RANCH AND SANTA PAULA CANYON.
06/17/1909	08-20--?	36.42	121.33	D			F	SOLEDAD.
07/03/1909	07--?--?	34.50	119.67	D			F	MONTECITO AND SANTA BARBARA.
07/05/1909	06-10--?	34.50	119.67	D			F	III AT SANTA BARBARA.
07/16/1909	10-28--?	34.50	119.67	D			F	IV AT LOS ANGELES AND SANTA BARBARA.
07/31/1909	19-37--?	34.50	119.67	D			F	IV AT OJAI AND SANTA BARBARA.
08/18/1909	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
11/24/1909	15--?--?	36.00	121.17	D			F	JOLON.
03/08/1910	09-30--?	36.17	120.67	D			F	PRIEST VALLEY.
04/30/1910	18-25--?	36.17	120.67	D			F	PRIEST VALLEY; 3 SHOCKS, THE SECOND ONE QUITE VIOLENT.
11/-?/1910	-?--?--?	34.50	119.67	D			F	SANTA BARBARA; 2 SLIGHT QUAKES DURING NOVEMBER.
02/02/1911	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
03/22/1911	10-55--?	35.75	120.67	D			F	SAN MIGUEL; QUITE SEVERE.
06/02/1911	-?--?--?	36.17	120.67	D			F	PRIEST VALLEY.
06/18/1912	22-27--?	36.00	121.17	D			F	JOLON. (RECORDED AT BERKELEY.)
10/20/1913	11-25--?	35.25	120.67	D			F	BETTERAVIA, PASO ROBLES, SAN LUIS OBISPO,AND SANTA MARIA.
11/27/1913	19--?--?	34.50	119.67	D			F	MONO RANCH.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/26/1913	12--?--?	36.17	121.00	D			F	SAN LUCAS.
11/24/1914	04-25--?	35.25	120.67	D			F	II AT SAN LUIS OBISPO; ABRUPT TREMBLING, LASTING 20 SECONDS.
01/12/1915	-?--?--?	34.92	120.50	D			F	BETTERAVIA.
01/12/1915	04-31--?	34.75	120.25	B			F	VIII AT LOS ALAMOS - EPICENTER 2 OR 3 MI. EAST OF LOS ALAMOS; FELT FROM SAN JOSE TO LOS ANGELES; SHAKEN AREA IN EXCESS OF 50,000 SQ. MI. - PRACTICALLY EVERY CHIMNEY DAMAGED AT LOS ALAMOS, VII AT LOMPOC, VI-VII AT SANTA MARIA, V AT SAN LUIS OBISPO AND SANTA BARBARA, IV AT PASO ROBLES, AND II AT LOS ANGELES, WEATHER BUREAU REPORTED V-VI AT SANTA BARBARA, V AT OZENA AND SAN LUIS OBISPO, IV AT PASO ROBLES, III AT OJAI, AND II IN PRIEST VALLEY; ALSO II AT BAKERSFIELD.
01/14/1915	-?--?--?	34.92	120.50	D			F	BETTERAVIA.
01/15/1915	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
01/20/1915	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
01/26/1915	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
01/27/1915	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
04/21/1915	09-58--?	35.25	120.67	D			F	IV AT SAN LUIS OBISPO; ALSO FELT 3 MI. NW OF PRIEST VALLEY.
08/23/1915	23-15--?	34.75	119.75	D			F	HILL CAMP.
08/31/1915	21--?--?	34.75	119.75	D			F	HILL CAMP.
09/08/1915	12-45--?	35.67	120.67	D			F	V IN REGION EAST OF PASO ROBLES; ANTELOPE - 2 SHOCKS, FIRST THE HEAVIER, OIL CAME UP WITH WATER IN WELL AFTER SHOCK. AT SHANDON A SEATED MAN WAS SHAKEN SO HARD HE THOUGHT A PERSON WAS SHAKING HIM. AT CRESTON THE SHOCK WAS SHORT AND SHARP. A SLIGHT LANDSLIDE AT PORT SAN LUIS. WEATHER BUREAU REPORTS -PASO ROBLES V AND SAN LUIS OBISPO III-IV.
09/14/1915	-?--?--?	34.75	119.75	D			F	HILL CAMP; 3 HARD SHOCKS - EARTH TREMBLED FOR 15 MINUTES AFTERWARDS.
02/27/1916	13-26--?	34.75	120.25	D			F	LOS ALAMOS.
03/01/1916	19-15--?	34.75	120.25	D			F	LOS ALAMOS.
05/06/1916	03-45--?	34.75	120.25	D			F	III AT LOS ALAMOS. FELT BY MANY AT EL ROBLAR RANCH, 2 MI. SE OF LOS ALAMOS.
08/06/1916	-?--?--?	36.00	121.00		7.0			(CALTECH FILE)
10/24/1916	13-03--?	35.25	120.67	D			F	II AT SAN LUIS OBISPO; PROBABLY NEXT SHOCK, WITH TIME ERROR.
10/24/1916	13-30--?	36.00	121.17	D			F	V AT JOLON; III AT A POINT 3.5 MI. NW OF PRIEST VALLEY.
12/01/1916	22-53--?	35.17	120.75	D			F	VII AT AVILA - CONSIDERABLE GLASS BROKEN AND GOODS IN STORES THROWN FROM SHELVES. FELT AT SAN LUIS OBISPO; WATER IN BAY DISTURBED, PLASTER IN COTTAGES JARRED LOOSE, SMOKESTACKS OF UNION OIL CO. REFINERY TOPPLED OVER. SEVERE AT PORT SAN LUIS; III AT SANTA MARIA.
02/01/1917	05-18--?	34.92	120.42	D			F	III AT SANTA MARIA.
04/05/1917	19--?--?	34.67	120.33	D			F	IV AT SANTA RITA; ALSO FELT AT LOMPOC.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
04/13/1917	03-59--?	34.25	119.67	D			F	VI AT SANTA BARBARA CHANNEL REGION; FELT OVER AN AREA OF COAST SOUTH AND EAST OF SANTA BARBARA AS FAR AS VENTURA, AND ON SANTA CRUZ ISLAND.
04/21/1917	06-59--?	34.25	119.67	D			F	V AT SANTA BARBARA CHANNEL; PERCEPTIBLE OVER AN AREA OF PERHAPS 4000 SQ. MI.
07/07/1917	20-57--?	35.25	120.50	D			F	LOPEZ CANYON; ALSO AT SAN LUIS OBISPO.
07/07/1917	21-02--?	35.25	120.50	D			F	LOPEZ CANYON.
07/07/1917	21-15--?	35.25	120.50	D			F	LOPEZ CANYON.
07/08/1917	03-20--?	34.92	120.42	D			F	II AT SANTA MARIA.
07/08/1917	11-29--?	35.25	120.50	D			F	IV IN LOPEZ CANYON.
07/09/1917	22-22--?	35.25	120.50	D			F	VII IN LOPEZ CANYON; IV AT SAN LUIS OBISPO.
07/09/1917	22-38--?	35.25	120.50	D			F	LOPEZ CANYON.
07/10/1917	-?-43--?	35.25	120.50	D			F	LOPEZ CANYON.
07/10/1917	-?-45--?	35.25	120.50	D			F	LOPEZ CANYON.
07/26/1917	08-31--?	34.92	120.42	D			F	V AT SANTA MARIA - FURNITURE MOVED. IV AT LOS OLIVOS - AWAKENED SLEEPERS AT SAN LUIS OBISPO
12/05/1918	02-38--?	35.67	120.67	D			F	IV AT PASO ROBLES; II AT SAN LUIS OBISPO.
12/05/1918	04-30--?	35.25	120.67	D			F	SAN LUIS OBISPO.
03/01/1919	04-19--?	36.17	120.67	D			F	IV IN PRIEST VALLEY.
03/15/1919	07-53--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/31/1919	21-31--?	36.33	120.67	D			F	V IN SAN BENITO COUNTY; FELT AT IDRIA - ORIGIN SOME DISTANCE FROM IDRIA
08/26/1919	12-12--?	34.50	119.67	D			F	V IN SANTA BARBARA COUNTY - FELT AT OJAI, SAN LUIS OBISPO (3 SHOCKS), SANTA BARBARA.
08/26/1919	14.57--?	34.50	119.67	D			F	V IN SANTA BARBARA COUNTY - THIS SHOCK STRONGER AT SANTA BARBARA THAN PREVIOUS SHOCK. BUILDINGS AND WHARVES SWAYED; FELT AT OJAI.
12/18/1919	07-15--?	35.67	120.67	D			F	PASO ROBLES.
01/30/1920	23-30--?	34.50	119.67	D			F	III AT SANTA BARBARA.
01/30/1920	23-33--?	34.50	119.67	D			F	II AT SANTA BARBARA.
01/30/1920	23-35--?	34.50	119.67	D			F	II AT SANTA BARBARA.
01/30/1920	23-38--?	34.50	119.67	D			F	II AT SANTA BARBARA.
01/31/1920	01--?--?	34.50	119.67	D			F	III AT SANTA BARBARA.
01/31/1920	01-03--?	34.50	119.67	D			F	III AT SANTA BARBARA.
01/31/1920	01-07--?	34.50	119.67	D			F	III AT SANTA BARBARA.
03/20/1920	07-04--?	35.25	120.67	D			F	II AT SAN LUIS OBISPO.
05/07/1920	01-59--?	35.25	120.67	D			F	IV AT SAN LUIS OBISPO.
06/28/1920	09-01--?	35.25	120.67	D			F	V AT SAN LUIS OBISPO.
12/01/1920	01-30--?	35.17	119.50	D			F	VI AT TAFT - MANY PEOPLE MADE "SEASICK", DISHES SHAKEN FROM SHELVES, IV AT MARICOPA.
12/05/1920	11-58--?	34.50	119.67	D			F	V IN SANTA BARBARA COUNTY MOUNTAINS, V AT LOMPOC, LOS ALAMOS, MARICOPA, OJAI, AND SANTA BARBARA.
12/06/1920	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
03/10/1922	11-21-20	35.75	120.25	C	6.5	43	F	IX IN CHOLAME VALLEY REGION OF SAN ANDREAS FAULT. FELT OVER AN AREA OF 100,000 SQ. MI. - CRACKS IN THE GROUND AND NEW SPRINGS. VII-VIII AT PARKFIELD AND SHANDON. VI-VII AT SAN LUIS OBISPO AND SIMMLER, AND V AT LOS ANGELES.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/16/1922	23-10--?	35.75	120.33	D			F	VI IN CHOLAME VALLEY - RATHER STRONG AFTERSHOCKS, V AT PASO ROBLES AND SAN LUIS OBISPO, AND IV AT ANTELOPE VALLEY; ALSO IV AT SHANDON.
03/19/1922	11--?--?	35.67	120.67	D			F	III AT PASO ROBLES.
03/23/1922	10--?--?	35.67	120.67	D			F	III AT PASO ROBLES.
03/25/1922	12--?--?	35.67	120.67	D			F	III AT PASO ROBLES.
05/31/1922	01-25--?	35.67	120.67	D			F	III AT PASO ROBLES; 2 SHOCKS.
07/05/1922	19--?--?	34.75	120.25	D			F	LOS ALAMOS.
07/09/1922	12--?--?	34.75	120.25	D			F	LOS ALAMOS.
07/11/1922	03--?--?	34.75	120.25	D			F	LOS ALAMOS.
07/11/1922	15-30--?	34.75	120.25	D			F	LOS ALAMOS.
08/18/1922	05-12--?	35.75	120.33	D			F	VII IN CHOLAME VALLEY; V AT PASO ROBLES AND SAN LUIS OBISPO.
08/20/1922	21-14--?	35.50	120.67	D			F	III AT ATASCADERO.
09/04/1922	10-15--?	35.67	120.67	D			F	IV AT PASO ROBLES.
09/05/1922	09-05--?	35.25	120.67	D			F	V AT SAN LUIS OBISPO; 2 SHOCKS.
12/29/1922	11--?--?	35.67	120.67	D			F	III AT PASO ROBLES.
12/29/1922	12--?--?	35.67	120.67	D			F	III AT PASO ROBLES.
03/12/1923	06--?--?	34.75	120.25	D			F	LOS ALAMOS.
05/04/1923	22-45--?	35.25	120.67	D			F	V AT SAN LUIS OBISPO; 2 SHOCKS, SECOND EQUALED INTENSITY II.
05/08/1923	05-02--?	35.75	120.33	D			F	II AT CHOLAME.
06/16/1923	20-40--?	35.67	120.67	D			F	IV AT PASO ROBLES - DURATION 15-20 SECONDS.
06/25/1923	13-21--?	35.25	120.67	D			F	II AT SAN LUIS OBISPO.
12/19/1923	07-35--?	34.92	120.42	D			F	II AT SANTA MARIA - DURATION 20 SECONDS.
07/02/1924	58-02--?	34.50	119.67	D			F	SANTA BARBARA.
12/30/1924	12-17--?	34.50	119.67	D			F	SANTA BARBARA.
12/30/1924	14-15--?	34.50	119.67	D			F	SANTA BARBARA.
06/29/1925	14-42-16	34.30	119.80	B	6.3	1	F	IX AT SANTA BARBARA; FELT OVER AN AREA OF 100,000 SQ. MI. - RECORDED WORLD-WIDE. RUPTURE AT DEPTH ON THE MESA AND RECORDED WORLD-WIDE. RUPTURE AT DEPTH ON THE MESA AND SANTA YNEZ FAULTS (BAILEY WILLIS); A FEW DEATHS, SEVERAL MILLION DOLLARS DAMAGE; IX AT GOLETA, NAPLES, AND SANTA BARBARA; VIII AT GAVIOTA, MIRAMAR, AND SANTA YNEZ, LOS ALAMOS, LOS OLIVOS; VII AT ARROYO GRANDE, NIPOMO, ORCOTT, ALAMOS, LOS OLIVOS; VII AT ARROYO GRANDE, NIPOMO, ORCOTT, ALAMOS, LOS OLIVOS; VII AT ARROYO GRANDE, NIPOMO, ORCOTT, PISMO BEACH, SANTA MARIA, AND VENTURA, AND VI AT AVILA, LOMPOC, AND PORT SAN LUIS.
06/29/1925	15-20--?	35.25	120.67	D			F	III AT SAN LUIS OBISPO.
06/29/1925	16-35--?	34.50	119.67	D			F	SANTA BARBARA; II AT OXNARD.
06/29/1925	18-54--?	34.50	119.67	D			F	IV AT SANTA BARBARA; II AT OXNARD - STRONGEST AFTERSHOCK OF THE DAY.
06/30/1925	01-37--?	34.50	119.67	D			F	SANTA BARBARA.
06/30/1925	02-47--?	34.50	119.67	D			F	SANTA BARBARA.
06/30/1925	09-19--?	34.50	119.67	D			F	SANTA BARBARA - VIOLENT; FELT AT OJAI AND OXNARD.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/03/1925	16-38--?	34.50	119.67	D			F	VII AT SANTA BARBARA; III AT PASADENA AND OJAI - STIFF TREMOR AT VENTURA.
07/03/1925	18-21--?	34.50	119.67	D			F	VII AT SANTA BARBARA - STRONGEST AFTERSHOCK; FELT AT LOS ANGELES, OJAI, AND PASADENA.
07/03/1925	18-46--?	34.50	119.67	D			F	SANTA BARBARA.
07/04/1925	19-18--?	34.50	119.67	D			F	SANTA BARBARA - ANOTHER SHOCK FELT LATER IN DAY.
07/05/1925	12--?--?	34.50	119.67	D			F	SANTA BARBARA; 11 SHOCKS IN THE NEXT 19 HOURS.
07/06/1925	21-45--?	34.50	119.67	D			F	SANTA BARBARA - SEVERAL FAIRLY SEVERE SHOCKS.
07/09/1925	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
07/20/1925	09-50--?	34.50	119.67	D			F	SANTA BARBARA.
07/29/1925	14--?--?	34.50	119.67	D			F	V AT WASIOJA - CEMENT WALK CRACKED.
07/30/1925	09-50--?	34.50	119.67	D			F	SANTA BARBARA.
07/30/1925	12--?--?	34.50	119.67	D			F	SANTA BARBARA.
08/13/1925	11--?--?	34.50	119.67	D			F	SANTA BARBARA - 5 LIGHT SHOCKS DURING NIGHT; THE STRONGEST TOOK PLACE JUST BEFORE 11--?--?.
10/04/1925	-?-50--?	34.50	119.67	D			F	SANTA BARBARA.
10/08/1925	21-30--?	34.50	119.67	D			F	SANTA BARBARA.
10/30/1925	09-45--?	34.50	119.67	D			F	SANTA BARBARA.
10/30/1925	13-30--?	34.50	119.67	D			F	SANTA BARBARA AND VENTURA.
02/18/1926	18-18--?	34.17	119.50	D			F	VII ORIGIN AT SEA, SW OF VENTURA; FELT ALONG COAST FROM SAN LUIS OBISPO ON NW TO SOUTH OF SANTA ANA, A DISTANCE OF 200 MI. AT SANTA BARBARA WINDOWS OF A SCHOOL WERE BROKEN, WATER PIPE IN ROUNDHOUSE WAS BROKEN. THERE WAS DAMAGE TO TELEPHONE EQUIPMENT AT SIMI. ALSO FELT AT LOS ANGELES, PASADENA, SANTA MONICA, SANTA SUSANA, AND VENTURA.
04/29/1926	12-18--?	34.67	120.17	D			F	IV AT BUELLTON.
06/18/1926	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
06/24/1926	15-30--?	34.50	119.67	D			F	V AT SANTA BARBARA.
06/29/1926	23-21--?	34.50	119.67	D			F	VII-VIII AT SANTA BARBARA - ONE PERSON KILLED BY FALLING CHIMNEY. VI AT BUELLTON AND VENTURA; ALSO FELT AT CAMARILLO, LOS ANGELES, OJAI, OXNARD, PORT HUENEME, AND SANTA PAULA - POSSIBLY SUBMARINE ORIGIN; FELT OVER AN AREA OF 30,000 SQ. MI.
07/03/1926	23--?--?	34.50	119.67	D			F	II AT SANTA BARBARA.
07/06/1926	17-45--?	34.50	119.67	D			F	V AT SANTA BARBARA.
07/25/1926	-?--?--?	36.30	120.30					(CALTECH FILE)
08/06/1926	17-42--?	34.50	119.67	D			F	IV IN SANTA BARBARA REGION; 2 SHOCKS AT OJAI - LASTED 30 SECONDS AT VENTURA WITH SHARP SHOCK AT SANTA BARBARA.
08/09/1926	04-12--?	34.50	119.67	D			F	V AT SANTA BARBARA; 2 SHOCKS AT VENTURA.
10/22/1926	10-10--?	35.67	120.67	D			F	III AT PASO ROBLES.
10/22/1926	-?--?--?	36.45	122.00					(CALTECH FILE)
12/09/1926	-?-03--?	35.67	120.67	D			F	IV AT PASO ROBLES - PROBABLY MISTIMED REPORT OF SHOCK AT -? 41-?.
12/09/1926	-?-41--?	35.25	120.67	D			F	NE OF SAN LUIS OBISPO; AT SAN LUIS OBISPO DURATION 20 SECONDS; FELT AT COALINGA WITH ORIGIN ABOUT 120 MI. FROM MT HAMILTON.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/27/1926	09-19--?	36.17	120.33	D			F	VI NEAR COALINGA; FELT OVER AN AREA OF 25,000 SQ. MI. FELT AT FIREBAUGH, FRESNO, LOS BANOS, MENDOTA, OAKDALE, OILFIELDS, PORTERVILLE, AND SAN LUIS OBISPO.
11/04/1927	11--?--?	34.58	120.67	D			F	LOMPOC, POINT ARGUELLO, AND SAN LUIS OBISPO.
11/04/1927	11-30--?	34.58	120.67	D			F	LOMPOC.
11/04/1927	13-50-53	34.54	121.40	A	7.3	3	F	X AT SEA, WEST OF POINT ARGUELLO. AREA SHAKEN WITH INTENSITY VI OR GREATER WAS 40,000 SQ. MI. A SMALL SEA WAVE WAS PRODUCED, RECORDED ON TIDE GAUGES AT SAN DIEGO AND SAN FRANCISCO, AND OBSERVED AS 6 FEET HIGH AT SURF; IX AT HONDA, ROBERDS RANCH, SURF, AND WHITE HILLS, VIII AT ARLIGHT, ARROYO GRANDE, BERROS, BETTERAVIA, CAMBRIA, CASMALIA, CAYUCOS, GUADOCEANO, PISMO BEACH, POINT CONCEPTION, SAN JULIAN RANCH, SAN LUIS OBISPO, AND SANTA MARIA, VI-VII AT GUADOCEANO, PISMO BEACH, POINT CONCEPTION, SAN JULIAN RANCH, SAN LUIS OBISPO, AND SANTA MARIA, VI-VII AT ALUPE, HALCYON, HARRISTON, HUASNO, LOMPOC, LOS ALAMOS, LOS OLIVOS, MORRO BAY, NIPOMO, ADELAIDA, ATASCADERO, BAKERSFIELD, BICKNELL, BUTTONWILLOW,CARPINTERIA CHOLAME, CRESTON, EDNA GAVIOTA, GOLETA, HARMONY, KING CITY, LAS CRUCES, NAPLES, OXNARD, PASO ROBLES, REWARD, SANTA BARBARA, SANTA MARGARITA, SANTA YNEZ, SOLVANG, TAFT, TEMPLETON, VENTURA, AND WASIOJA, AND IV-V AT ANNETTE, BIG SUR, CASTROVILLE, COALINGA, FELLOWS, GONZALES, GORMAN, HOLLISTER, LOCKWOOD, LUCIA, MCKITTRICK, MONTEREY, PARKFIELD, PATTIWAY, PORT SAN LUIS, POZO, PRIEST, SALINAS, SANGER, SAN LUCAS, SAN SIMEON, SANTA PAULA, SCHEIDECK, SESPE, SIMMLER, SOLEDAD, AND TEHACHAPI. DATA FROM BSSA V. 17, P. 258 AND V. 20, P. 53.
11/04/1927	14-12--?	34.58	120.67	D			F	SANTA MARIA - AFTERSHOCK.
11/04/1927	14-14--?	34.58	120.67	D			F	SANTA MARIA - AFTERSHOCK.
11/04/1927	15--?--?	34.58	120.67	D			F	SAN LUIS OBISPO - AFTERSHOCK.
11/04/1927	15-42--?	34.58	120.67	D			F	SANTA MARIA - AFTERSHOCK.
1/05/1927	08-17--?	34.58	120.67	D			F	POINT ARGUELLO - AFTERSHOCK; MILD AT SURF.
11/05/1927	09--?--?	34.58	120.67	D			F	POINT ARGUELLO - AFTERSHOCK; REPORTED FROM PASO ROBLES TO HADLEY TOWER.
11/05/1927	11-37--?	34.58	120.67	D			F	POINT ARGUELLO - AFTERSHOCK; REPORTED FROM SURF TO HADLEY TOWER, AND SOUTH OF SAN LUIS OBISPO.
11/06/1927	-?-06--?	34.67	120.17	D			F	IV AT BUELLTON.
11/06/1927	02-25--?	34.67	120.17	D			F	POINT ARGUELLO - AFTERSHOCK; STRONGEST IMMEDIATE AFTERSHOCK AT LOMPOC.
11/06/1927	03-10--?	34.67	120.17	D			F	IV AT BUELLTON.
11/06/1927	22-10--?	34.67	120.17	D			F	OFF POINT CONCEPTION.
11/06/1927	22-50--?	34.67	120.17	D			F	IV AT BUELLTON.
11/06/1927	23-10--?	34.67	120.17	D			F	OFF POINT CONCEPTION.
11/08/1927	10-10--?	34.67	120.17	D			F	IV AT BUELLTON - SHARP BUMPING AT 10-02--?, AROUSED NEARLY ALL. AT LOMPOC MANY AWAKENED BY SHOCK AT 10-15--?.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
11/19/1927	03-32--?	34.92	120.42	D			F	VII AT SANTA MARIA - CENTERED TO NW OF ORIGIN OF NOVEMBER 4 QUAKE -WEAKER, YET NEARLY AS STRONG AT SANTA MARIA,AND VI AT BETTERAVIA AND BICKNELL; REPORTED FROM SAN MIGUEL AND PARKFIELD ON THE NORTH TO SANTA BARBARA CHANNEL ON THE SOUTH.
12/05/1927	11-45--?	34.58	120.67	D			F	IV AT POINT ARGUELLO, AND IV AT BUELLTON WITH 2 SHOCKS 15 SECONDS APART; FELT AT GUADALUPE, SANTA MARGARITA, SANTA MARIA AND SURF.
12/31/1927	10-10--?	34.58	120.67	D			F	V AT POINT ARGUELLO.
03/15/1928	12-03--?	34.92	120.42	D			F	SANTA MARIA.
03/15/1928	12-20--?	34.50	119.67	D			F	SANTA BARBARA.
03/16/1928	14-30--?	34.92	120.42	D			F	SANTA MARIA.
03/29/1928	06-25--?	34.92	120.42	D			F	VII AT SANTA MARIA.
06/09/1928	08-22--?	35.17	119.50	D			F	TAFT.
06/09/1928	08-31--?	35.17	119.50	D			F	TAFT.
06/09/1928	12-25--?	35.17	119.50	D			F	TAFT.
09/03/1928	04-01-54	34.50	122.50	D	5.0	1		OFF POINT ARGUELLO - LICK OBSERVATORY S-P= 39 SECONDS.
11/02/1928	05--?--?	34.67	120.42	D			F	LOMPOC.
05/28/1929	07-10--?	36.17	120.33	D			F	COALINGA.
07/03/1929	09-24--?	34.50	119.67	D			F	SANTA BARBARA.
07/12/1929	13-10--?	36.17	120.33	D			F	COALINGA.
08/28/1929	18-10--?	34.50	119.67	D			F	SANTA BARBARA.
09/09/1929	05-15--?	34.50	119.67	D			F	GAVIOTA, NAPLES, AND SANTA BARBARA.
09/16/1929	03-16--?	35.42	120.92	D			F	CAYUCOS.
09/16/1929	06-15--?	35.42	120.92	D			F	CAYUCOS.
10/05/1929	20-03--?	36.17	120.33	D			F	COALINGA AND LIGHTHIPE.
10/06/1929	21-14--?	36.17	120.33	D			F	COALINGA.
10/07/1929	08--?--?	36.17	120.33	D			F	COALINGA.
10/07/1929	11-30--?	34.83	120.42	D			F	ORCUTT.
10/11/1929	17-55--?	36.17	120.33	D			F	COALINGA.
10/15/1929	22-02--?	36.17	120.67	D			F	COALINGA, KETTLEMEN HILLS, OILFIELDS, AND PRIEST VALLEY,.
11/07/1929	06-30--?	36.33	119.67	D			F	HANFORD.
11/09/1929	02-30--?	36.17	120.33	D			F	BITTER WATER, COALINGA, AND MCKITTRICK.
11/20/1929	22-50--?	36.42	121.00	D			F	BITTER WATER.
11/24/1929	09-54--?	36.42	121.00	D			F	LONOAK, BITTER WATER, AND LEWIS CREEK.
11/26/1929	08-05--?	36.42	121.00	D			F	V AT BITTER WATER AND SAN ARDO; FELT FROM HOLLISTER TO SANTA MARGARITA.
11/26/1929	09--?--?	36.42	120.83	D			F	HERNANDEZ.
11/26/1929	18-06--?	36.42	121.00	D			F	BITTER WATER.
12/05/1929	07-40--?	36.33	119.67	D			F	HANFORD.
03/11/1930	23-59--?	36.42	121.25	D			F	PINNACLES.
06/21/1930	05-15--?	34.83	120.50	D			F	CASMALIA.
08/05/1930	11-25--?	34.42	119.50	D			F	NEAR SANTA BARBARA - FELT OVER AN AREA OF 9000 SQ. MI. V-VI AT CARPINTERIA, GOLETA, OJAI, OXNARD, AND SANTA BARBARA,.
08/08/1930	16.46--?	34.42	119.67	D			F	SANTA BARBARA AND GOLETA.
08/18/1930	13-09--?	34.33	120.58	D			F	OFF POINT CONCEPTION; V OVER A LAND AREA OF 500 SQ. MI. NEAR POINT CONCEPTION.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
08/28/1930	05-15--?	36.42	121.33	D			F	SOLEDAD.
09/02/1930	13-35--?	35.00	121.00	D			F	OFF COAST - FELT AT HALCYON AND SAN LUIS OBISPO.
09/09/1930	05-27--?	34.42	119.50	D			F	SANTA BARBARA.
10/02/1930	14-18--?	34.58	120.67	D			F	OFF POINT ARGUELLO - FELT AT HALCYON.
10/28/1930	13-57--?	35.42	120.92	D			F	OFF COAST NEAR CAYUCOS - FELT AT NIPOMO.
12/08/1930	01-23--?	34.50	119.67	D			F	GOLETA AND SANTA BARBARA.
12/08/1930	01-29--?	34.50	119.67	D			F	GOLETA AND SANTA BARBARA.
02/21/1931	08-10--?	35.67	121.33	D			F	NW OF SAN LUIS OBISPO - FELT AT BRYSON AND PIEDRAS BLANCAS.
02/23/1931	10-01--?	35.83	120.50	D			F	OVER AN AREA OF 5000 SQ. MI.; V AT CAYUCOS, PARKFIELD, AND TEMPLETON.
02/23/1931	10-33--?	35.83	120.50	D			F	SAME AS ABOVE.
04/05/1931	03--?--?	36.17	121.00	D			F	SE OF KING CITY.
07/15/1931	18-40--?	35.00	120.58	D			F	GUADALUPE, NIPOMO, AND SANTA MARGARITA.
07/21/1931	03-25--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/21/1931	12-08--?	35.25	120.67	D			F	IV AT HALCYON, LOS ALAMOS, NIPOMO, OCEANO, AND TEMPLETON:ALSO FELT AT CAMBRIA, GAVIOTA, PIEDRAS BLANCAS, PORT SAN LUIS, SAN LUIS OBISPO, SANTA MARGARITA, AND SANTA MARIA
09/03/1931	13-50--?	34.50	119.67	D			F	SANTA BARBARA.
09/10/1931	14-35--?	35.50	120.67	D			F	ATASCADERO.
09/30/1931	14-35--?	35.50	120.67	D			F	ATASCADERO.
10/13/1931	12-25--?	36.33	121.67	D			F	JAMESBURG.
10/18/1931	19-58--?	36.33	121.67	D			F	IV AT HOLLISTER, JAMESBURG, AND SPRECKLES; ALSO FELT AT APTOS, CARMEL, CHUALAR, MOSS LANDING, MONTEREY, PARAISO, SALINAS, AND SANTA CRUZ.
12/04/1931	-?-53--?	36.50	121.67	D			F	10 MI. S OF SPRECKELS. FELT AT HOLLISTER, METZ, PIGEON POINT, SPRECKELS, AND SANTA CRUZ.
02/04/1932	16-02-58	34.55	119.73	C	3.0	1	F	SANTA BARBARA AND VENTURA.
02/05/1932	04-14-45	35.83	121.47	C	3.5	1	F	COAST OF MONTEREY COUNTY; FELT AT PIEDRAS BLANCAS LIGHT AND SALMON CREEK.
02/05/1932	06-46-54	35.83	121.47	C	3.5	1	F	COAST OF MONTEREY COUNTY; FELT AT PIEDRAS BLANCAS LIGHT AND SALMON CREEK.
02/05/1932	07-10--?	35.83	121.47	C			F	AFTERSHOCK OF PRECEDING.
02/26/1932	16-58--?	36.00	121.00		5.0		F	IV AT APTOS, ASILOMAR, CARMEL, DEL MONTE, GONZALES, METZ, MONTEREY, PACIFIC GROVE, AND PEBBLE BEACH.
03/13/1932	23-09-24	34.44	120.17	B	3.5	1	F	OFF POINT CONCEPTION; FELT AT BUELLTON.
04/21/1932	03-36-20	35.50	120.67	D	3.0		F	ATASCADERO.
05/06/1932	03-37-08	36.00	120.50	C	3.0	1	F	PARKFIELD.
06/27/1932	05-17-25	36.00	122.00	D	4.0			COAST OF MONTEREY COUNTY.
10/24/1932	04-45--?	35.75	120.75	D			F	PASO ROBLES.
01/30/1933	17--?--?	34.67	120.42	D			F	LOMPOC.
02/26/1933	09-34-32	36.40	121.30	D			F	III AT HOLLISTER, SALINAS, AND SPRECKLES.
04/12/1933	10-03--?	36.33	121.75	D			F	IV AT PORTERVILLE AND VISALIA.
06/26/1933	06-26--?	34.42	120.50	D			F	V AT BUELLTON AND POINT CONCEPTION.
06/26/1933	06-29--?	34.42	120.50	D			F	V AT BUELLTON AND POINT CONCEPTION.
01/09/1934	12-48--?	35.13	120.08	C	3.0			

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
01/12/1934	12-50--?	34.45	120.15	D			F	IV AT LOS ALAMOS.
02/01/1934	16-09--?	34.55	119.53	B	3.5		F	II AT SANTA BARBARA.
02/11/1934	15-16--?	34.55	119.53	C	2.0			
03/20/1934	11-48--?	36.00	120.00	D	3.0			
05/06/1934	20-14--?	35.83	120.75	B	3.5			
05/10/1934	11-28--?	34.50	119.58	C	3.0			
05/19/1934	06-37--?	34.58	120.75	D	3.0			
05/24/1934	06-52--?	34.42	119.75	C	2.5			
05/24/1934	09-04--?	34.42	119.75	C	2.5			
05/24/1934	11-18--?	34.42	119.75	C	2.0			
06/05/1934	09-51--?	35.80	120.33	D			F	COALINGA AND KETTLEMAN HILLS; ALSO FELT AT MONTEREY AND SANTA CRUZ.
06/05/1934	11-30--?	35.80	120.33	D			F	SAN MIGUEL AND SHANDON.
06/05/1934	11-47--?	35.80	120.33	B	3.0			
06/05/1934	13-46--?	35.80	120.33	C	3.0			
06/05/1934	21-30--?	35.80	120.33	D			F	SAN MIGUEL.
06/05/1934	21-48--?	35.80	120.33	B	5.0		F	V AT ADELAIDA, PARKFIELD, AND PRIEST, IV AT ATASCADERO, AVENAL, BIG SUR, BRYSON, CARMEL, HANFORD, KING CITY, LEMOORE, LONOAK, PARAISO, SAN MIGUEL, SANTA CRUZ, SHANDON, AND TEMPLETON, III AT APTOS, BOULDER CREEK, CAMBRIA, CHUALAR, COALINGA, GONZALES, HOLLISTER, MONTEREY, MORRO BAY, PASO ROBLES, SALINAS, SAN FRANCISCO, SAN JOAQUIN VALLEY, SAN LUIS OBISPO, SOLEDAD, SPRECKLES, ETC.; NOT FELT AT ANTIOCH, ETC., BAKERSFIELD, FRESNO, GILROY, LIVERMORE, LOS GATOS, MARICOPA, MERCED, MODESTO, MORGAN HILL, REDWOOD CITY, SAN JOSE, SANTA MARIA, TULARE, OR WATSONVILLE.
06/05/1934	22-52--?	35.80	120.33	C	4.0		F	VI AT ADELAIDA; IV AT ATASCADERO.
06/05/1934	23-30--?	35.80	120.33	D			F	V AT LEMOORE; ALSO FELT AT CASTROVILLE.
06/06/1934	-?-55--?	35.80	120.33	C	3.0			
06/06/1934	16-40--?	35.80	120.33	C	3.5			
06/06/1934	22-40--?	35.80	120.33	C	3.5		F	ADELAIDA, GRAEAGLE, AND PAYNES CREEK.
06/07/1934	22-30--?	35.80	120.33	D			F	STONE CANYON.
06/08/1934	04-15--?	35.80	120.33	D			F	IV AT GONZALES AND MCKITTRICK.
06/08/1934	04-30--?	35.80	120.33	B	5.0		F	VI TO VII AT CHOLOME RANCH, PARKFIELD, AND STONE CANYON DURATION 30 SECONDS, DAMAGE SLIGHT, V AT ATASCADERO, AT ANTELOPE, BIG SUR, CAMBRIA, CASTROVILLE, DELANO, MONTEREY, PASO ROBLES, SAN LUIS OBISPO, SANTA BARBARASANTA MARGARITA, SANTA MARIA, SOLEDAD, TAFT, VENTURA, VISALIA, ETC., AND III OR LESS AT ARVIN, BAKERSFIELD, FRESNO, KERNVILLE, LOMPOC, LOS ANGELES, MENDOTA, PORTERVILLE, SALINAS, SAN BENITO, SANTA ANA, SANTA BARBARA, TULARE, WATSONVILLE, ETC.; NOT FELT AT BIG BASIN, CAJON, COYOTE, GILROY, HUNTINGTON BEACH, INDEPENDENCE, INYOKERN, LANCASTER, MERCED, POMONA, OR SAN JOSE.
06/08/1934	04-37--?	35.60	121.30	D			F	IV AT PIEDRAS BLANCAS, SAN LUIS OBISPO, AND SANTA CRUZ; ALSO FELT AT BRYSON AND LOS ALAMOS.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/08/1934	04-45--?	35.80	120.33	D			F	ATASCADERO, COALINGA, LOCKWOOD, PASO ROBLES, PORT SAN LUIS, PRIEST, SAN MIGUEL, AND WESTHAVEN.
06/08/1934	04-47--?	35.80	120.33	B	6.0		F	WITHIN A RADIUS OF 250 KM FROM THE EPICENTER NEAR THE SOUTHEASTERN ANGLE OF MONTEREY COUNTY; VII TO VIII AT PARKFIELD, VI AT COALINGA, KETTLEMAN CITY, LEMOORE, AND STONE CANYON, V AT ATASCADERO, DUDLEY, HOLLISTER, KING CITY, OILFIELDS, SAN MIGUEL, SEASIDE, SHALE PUMP STATION, AND SHANDON, IV AT ANTELOPE, AVILA, CANOGA PARK, HANFORD, LOS ALAMOS, MARICOPA, MORRO BAY, NIPOMO, PASO ROBLES, PRIEST, SAN LUIS OBISPO, SANTA CRUZ, SANTA MARIA, SOLEDAD, VISALIA ETC., AND III OR LESS AT APTOS, FRESNO, KERNVILLE, LONE PINE, LOS BANOS, MENDOTA, MONTEREY, OAKLAND HARBOR, SALINAS, SAN BENITO, SANTA ANA, TEHACHAPI, TULARE, ETC.
06/08/1934	05--?--?	35.60	121.30	D			F	PIEDRAS BLANCAS LIGHT; ALSO BRYSON, KERNVILLE, LA PANZA, LEMOORE, PARKFIELD, SANDBERG, AND SAN FERNANDO.
06/08/1934	05-20--?	35.80	120.33	D			F	III AT ATASCADERO.
06/08/1934	05-23--?	35.80	120.33	C	3.5		F	ATASCADERO AND SAN MIGUEL.
06/08/1934	05-36--?	35.80	120.33	C	3.0			
06/08/1934	05-42--?	35.80	120.33	B	4.5		F	ATASCADERO, BIG SUR, COALINGA, KING CITY, PASO ROBLES, AND WESTHAVEN.
06/08/1934	05-50--?	35.80	120.33	D			F	IV AT ATASCADERO; ALSO FELT AT COALINGA AND SAN LUIS OBISPO.
06/08/1934	09-30--?	35.80	120.33	B	4.0		F	ATASCADERO AND PARKFIELD.
06/08/1934	15-30--?	35.80	120.33	C	3.5			
06/08/1934	16-30--?	35.80	120.33	D			F	PARKFIELD.
06/08/1934	23-23--?	35.80	120.33	B	4.0		F	NEAR PARKFIELD.
06/10/1934	06-47--?	35.80	120.33	C	3.0			
06/10/1934	08-03--?	35.80	120.33	B	4.5		F	NEAR PARKFIELD; IV AT SAN MIGUEL.
06/10/1934	20-02--?	35.80	120.33	D			F	IV AT SAN MIGUEL; ALSO PARKFIELD AND WOODY.
06/11/1934	03-25--?	35.80	120.33	C	3.0			
06/12/1934	10-47--?	35.80	120.33	C	3.5			
06/14/1934	14-55--?	35.80	120.33	C	4.0		F	IV AT ATASCADERO; ALSO FELT AT SAN MIGUEL AND TEMPLETON.
06/14/1934	15-54--?	35.80	120.33	C	4.0		F	III AT ATASCADERO AND SAN MIGUEL.
06/14/1934	19-26--?	35.80	120.33	C	4.5		F	ATASCADERO AND TEMPLETON.
06/14/1934	22-02--?	35.80	120.33	C	3.5		F	ATASCADERO.
06/15/1934	04-48--?	35.80	120.33	C	3.0			
06/16/1934	23-03--?	36.50	121.00	D	4.0		F	IV AT HOLLISTER AND MONTEREY, AND III AT GONZALES, PARKFIELD, AND SALINAS.
07/02/1934	18-44--?	35.80	120.33	B	3.0			
08/04/1934	-?-18--?	35.80	120.33	B	3.0			
08/21/1934	03-37--?	36.08	120.58	D			F	IV IN STONE CANYON.
08/25/1934	18-52--?	34.42	119.75	C	2.5			
08/26/1934	03-02--?	35.57	119.85	B	3.0			
09/06/1934	23-24--?	36.00	120.55	C	3.0			
09/16/1934	14-38--?	35.83	120.33	C	3.5			
10/07/1934	-?-18--?	34.55	120.78	C	3.5			

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
10/08/1934	04-57--?	34.50	119.58	C	2.0			
10/10/1934	10-52--?	34.55	120.78	C	3.0			
10/19/1934	15-39--?	35.80	120.33	C	3.0			
11/04/1934	22-17--?	34.53	119.67	B	3.0			
11/21/1934	01-02--?	34.58	119.62	B	2.5			
12/01/1934	13-05--?	36.00	121.50	D			F	15 MI. S OF PARAISO; V AT PIEDRAS BLANCAS LIGHT AND IV AT PARAISO.
12/02/1934	16-07--?	35.97	120.58	C	4.0		F	SAN MIGUEL.
12/03/1934	01-54--?	35.95	121.50	C	4.5		F	IV AT BRYSON, KING CITY, AND PARAISO; ALSO FELT AT PARKFIELD, PASO ROBLES, SAN LUCAS, AND SAN MIGUEL.
12/17/1934	11-10--?	34.58	120.33	B	4.5		F	VI AT LOS ALAMOS.
12/17/1934	13-51--?	34.58	120.33	C	2.5		F	LOS ALAMOS.
12/17/1934	15-16--?	34.55	119.67	C	2.5			
12/17/1934	15-35--?	34.58	120.33	C	2.5			
12/18/1934	03-09--?	34.58	120.33	C	4.0		F	LOS ALAMOS.
12/18/1934	04-34--?	34.58	120.33	C	3.0		F	LOS ALAMOS.
12/18/1934	05-28--?	34.58	120.33	C	3.0		F	LOS ALAMOS.
12/19/1934	20-39--?	34.28	119.50	B	2.5			
12/20/1934	12-37--?	34.58	120.33	C	2.5		F	LOS ALAMOS.
12/20/1934	12-39--?	34.58	120.33	C	3.0			
12/20/1934	22-21--?	34.58	120.33	C	3.0			
12/23/1934	16-08--?	34.58	120.33	C	2.5			
12/24/1934	10-22--?	34.58	120.33	B	3.0		F	LOS ALAMOS.
12/24/1934	16-26--?	35.93	120.48	B	5.0		F	IV AT LOS ALAMOS AND SHANDON; ALSO FELT AT KING CITY TEMPLETON.
12/25/1934	04-03--?	34.58	120.33	C	3.0			
01/06/1935	04-04--?	35.98	120.48	C	4.0		F	IV AT PARKFIELD; ALSO FELT AT SHANDON.
01/06/1935	04-25--?	35.90	120.45	D			F	IV AT PARKFIELD.
01/06/1935	04-40--?	35.98	120.48	C	4.0		F	IV AT PARKFIELD AND III AT SHANDON.
01/07/1935	-?-11--?	35.75	119.67	D	3.0			
01/23/1935	03-16--?	34.58	120.33	C	3.5		F	IV AT LOS ALAMOS.
01/27/1935	09-49--?	34.50	119.62	B	2.5			
02/18/1935	04-02--?	35.93	120.48	C	3.5			
02/19/1935	14-17--?	35.93	120.48	D	3.0			
02/28/1935	19-06--?	35.80	120.33	C	3.0			
03/03/1935	11-26--?	36.42	121.75	C	3.0			
03/06/1935	23-14--?	34.43	119.87	C	3.5		F	III AT SANTA BARBARA.
03/19/1935	03-59--?	34.55	120.78	B	4.0			OFF POINT ARGUELLO.
04/05/1935	10-13--?	35.93	120.48	C	3.5			
05/05/1935	12-58--?	34.58	119.68	C	2.5			
05/18/1935	04-36--?	34.58	120.33	B	3.5		F	IV AT LOS ALAMOS.
05/19/1935	03-44--?	34.58	120.33	C	3.0			
05/20/1935	23-44--?	34.58	120.33	C	3.0			
05/27/1935	16-08--?	35.37	120.97	C	3.0		F	III AT TEMPLETON.
06/10/1935	02-02--?	35.33	119.83	C	3.5			
06/18/1935	08-52--?	34.60	119.60	C	2.0			
06/23/1935	23-53--?	34.55	119.68	C	3.0			

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/30/1935	23-28--?	36.00	121.00	D	4.0		F	SE OF SALINAS; III AT HOLLISTER.
07/25/1935	04-16--?	35.80	120.33	C	3.0		F	V AT PARKFIELD.
07/28/1935	06--?--?	35.70	121.12	B	4.0		F	SAN SIMEON.
08/06/1935	19-05--?	34.62	119.62	C	3.0		F	SANTA BARBARA.
08/07/1935	22-30--?	34.55	120.78	C	3.5			
08/09/1935	17-14--?	36.17	120.98	C	3.5		F	PRIEST VALLEY.
08/31/1935	09-28--?	34.50	119.70	C	2.5			
10/18/1935	09-24--?	35.80	120.70	D	3.5		F	IV AT PARKFIELD - AFTERSHOCK.
10/22/1935	18-37--?	35.93	120.48	C	4.0		F	PARKFIELD.
10/25/1935	19-43--?	36.40	121.55	D			F	13 MI. W OF SOLEDAD; IV AT SAN BENITO.
10/26/1935	10-46--?	35.85	121.40	D			F	AFTERSHOCK.
12/22/1935	06-54--?	34.55	120.78	C	3.0			
02/03/1936	09-12--?	34.75	119.75	C	2.5			
02/21/1936	23-06--?	34.42	119.67	C	3.0			
02/22/1936	-?-18--?	34.42	119.67	C	3.0			
02/22/1936	-?-21--?	34.42	119.67	C	2.5			
02/22/1936	-?-23--?	34.42	119.67	C	3.0			
02/22/1936	04-55--?	34.42	119.67	C	3.0			
03/06/1936	03-45--?	35.90	120.40	D	3.0			
03/17/1936	01-55--?	36.50	120.92	C	4.0		F	IV AT CHUALAR, HOLLISTER, AND TRES PINOS.
03/18/1936	09-07--?	35.93	120.48	C	2.5			
03/27/1936	-?-58--?	34.55	120.78	C	3.0			
03/29/1936	09-26--?	34.50	119.62	C	2.5			
05/20/1936	17-22--?	35.93	120.48	C	3.0			
05/23/1936	04-41--?	36.17	120.92	C	4.0		F	IV AT KING CITY.
05/27/1936	19-55--?	36.50	121.17	C	4.5			SAN BENITO COUNTY.
06/24/1936	12-23--?	35.12	120.08	C	3.0		F	SAN LUIS OBISPO CO.; IV AT LOS ALAMOS.
07/13/1936	18-09--?	34.50	119.60	D	2.5			
07/22/1936	04-03--?	34.50	119.80	C	2.5			
07/30/1936	09-36--?	34.57	119.63	C	3.0			
09/07/1936	16-47--?	34.37	120.38	C	3.0			
09/09/1936	04-54--?	34.37	120.38	C	4.0		F	LOS ALAMOS.
09/10/1936	21-21--?	34.40	120.40	D	3.0			
09/12/1936	13-56--?	34.75	120.33	C	3.5			
09/15/1936	-?-09--?	34.50	120.50	D	2.5			
10/16/1936	15-30--?	34.83	120.58	C	4.0			NEAR CASMALIA.
10/16/1936	15-36--?	34.83	120.58	C	3.0			
10/17/1936	01-17--?	34.83	120.58	C	3.0			
10/19/1936	14-01--?	34.83	120.58	C	3.0			
11/01/1936	15-10--?	34.55	120.78	B	4.0			OFF POINT ARGUELLO.
11/02/1936	01-29--?	34.55	120.78	C	3.0			
11/05/1936	14-30--?	35.85	121.40	D			F	HOLLISTER.
11/08/1936	16-51--?	34.55	120.78	C	3.0			
11/08/1936	22-43--?	34.55	120.78	C	3.0			
11/18/1936	17-15--?	35.35	120.60	D			F	POZO, SAN LUIS OBISPO, AND SANTA MARGARITA.
11/18/1936	18-02--?	34.70	120.25	C	4.5		F	IV AT ARROYO GRANDE, ATASCADERO, BETTERAVIA, LOS ALAMOS OCEANO, POZO, SAN LUIS OBISPO, AND SANTA MARGARITA.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
11/22/1936	02-16--?	34.58	120.78	C	3.5			
11/25/1936	21-51--?	34.58	120.78	C	3.0			
12/23/1936	17-16--?	35.93	120.48	B	3.5			
12/26/1936	01-12--?	34.55	119.68	C	2.5			
01/12/1937	15-44--?	34.50	120.80	D	3.0			
01/28/1937	17-36--?	34.43	119.87	C	2.5			
02/16/1937	17-40--?	34.55	120.78	C	4.0			OFF POINT ARGUELLO.
02/17/1937	03-33--?	36.50	121.58	C	4.5		F	9 MI. SE OF PAICINE; FELT AT ANTELOPE, HOLLISTER, AND PANOCHE.
02/20/1937	09-58--?	35.93	120.48	C	4.0		F	PARKFIELD AND PASO ROBLES.
02/22/1937	18-10--?	36.17	121.53	C	4.0		F	KING CITY.
02/24/1937	13-37--?	34.50	119.70	C	2.0			
02/25/1937	03-20--?	34.50	119.70	C	2.0			
03/26/1937	21-35--?	34.60	119.70	C	3.5			
03/31/1937	17-43--?	34.50	119.70	C	3.0			
04/17/1937	08-30--?	34.60	119.70	C	2.5			
04/30/1937	08-16--?	34.50	119.70	D	2.5			
05/31/1937	15-33--?	36.50	120.70	C	3.0			
06/02/1937	09-32--?	34.40	119.70	C	2.5			
07/31/1937	14-18--?	34.22	119.55	C	3.0			
07/31/1937	15-14--?	34.22	119.55	C	2.5			
08/15/1937	19-01--?	36.50	120.70	D	3.0			
08/22/1937	01-56--?	35.00	121.00	D	3.5			
09/16/1937	02-48--?	35.93	120.48	B	3.5		F	NEAR PARKFIELD; FELT AT BRADLEY.
09/18/1937	13-29--?	36.50	121.50	D	4.0		F	9 MI. SE OF PAICINES; FELT AT CHUALAR, SALINAS, AND SPRECKLES.
09/22/1937	02-41--?	34.50	119.70	C	3.0			
09/29/1937	22-39--?	34.50	119.70	C	3.0			
10/13/1937	08-32--?	34.40	119.70	C	2.5			
11/01/1937	21-40--?	36.50	121.40	D			F	6 MI. N OF GONZALES.
11/03/1937	10--?--?	36.15	121.00	D			F	V AT SAN LUCAS; FELT ALSO AT KING CITY AND SAN ARDO.
11/22/1937	04-12--?	34.55	120.78	C	4.5		F	OFF POINT ARGUELLO; V AT BUELLTON, GOLETA, PISMO BEACH, POINT D SANTA MARIA, AND IV AT ARLIGHT, BETTERAVIA, BICKNELL, E, GAVIOTA, GUADALUPE, LOMPOC, LOS ALAMOS, LOS OLIVOS, SANTA URF.
11/22/1937	04-51--?	34.55	120.78	C	3.5			
11/28/1937	09-55--?	34.55	120.78	C	3.5			
12/03/1937	15-28--?	34.55	120.78	C	4.0		F	OFF POINT ARGUELLO; FELT AT GAVIOTA AND POINT CONCEPTION.
12/03/1937	21-13--?	34.55	120.78	C	3.5			
12/05/1937	01-36--?	36.00	121.00	D	3.5		F	19 MI. S OF LOS BANOS; V AT LOS BANOS.
12/05/1937	01-37--?	36.00	121.00	D	4.0			SAN BENITO COUNTY.
12/05/1937	02-05--?	36.00	121.00	D	3.0		F	19 MI. S OF LOS BANOS.
12/24/1937	11-57--?	34.50	120.80	D	4.0		F	OFF POINT ARGUELLO. FELT AT CASMALIA, LOS ALAMOS, POINT CONCEPTION.
12/25/1937	13-01--?	36.00	120.00	D	3.0			
01/01/1938	01-59--?	34.55	120.78	C	3.5			

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
01/18/1938	04-35--?	34.55	120.78	B	3.5			
01/24/1938	04-38--?	34.55	120.78	C	3.5			
01/25/1938	12-24--?	34.55	120.78	C	3.5			
02/01/1938	18-14--?	34.55	120.78	C	3.5			
02/20/1938	14--?--?	34.55	120.78	C	3.5			
02/21/1938	10-59--?	35.93	120.78	C	3.0			
03/04/1938	15-14--?	34.30	119.57	C	2.5			
03/04/1938	18-25--?	34.30	119.57	C	2.5			
04/12/1938	01-50--?	34.55	120.78	C	3.5			
05/10/1938	10-32--?	36.20	121.30	D	4.5		F	BIG SUR, HOLLISTER, KING CITY, PINNACLES, SALINAS, SOLEDAD, SOQUEL, AND TRES PINOS-6 SHOCKS FELT AT PINNACLES.
05/10/1938	10-41--?	36.20	121.30	D	4.0		F	SAN BENITO.
05/13/1938	19-34--?	36.20	121.30	D	4.0			MONTEREY COUNTY.
05/27/1938	22-03--?	36.20	120.00	D	3.5			
06/01/1938	05-17--?	34.55	119.68	D			F	SANTA BARBARA.
06/01/1938	06-17--?	34.55	119.68	D	3.0			
06/06/1938	02-55--?	34.50	119.67	C	3.0			
09/16/1938	06-11--?	36.40	121.20	D	4.0		F	PINNACLES.
09/27/1938	10-21--?	34.50	119.70	C	2.5			
09/27/1938	12-23--?	36.30	120.90	C	5.0		F	OVER AN AREA OF 9000 SQ. MI. OF WEST-CENTRAL CALIFORNIA, ALONG THE COAST AS FAR NORTH AS PESCADERO AND SOUTH TO SAN LUIS OBISPO. INLAND IT WAS FELT AT COALINGA, MENDOTA, AND STEVENSON, WITH A V AT BIG SUR, BRYSON, CHUALAR, GONZALES, GREENFIELD, HARMONY, HOLLISTER, JOLON, LOCKWOOD, PAICINES, PARAISO, PINNACLES, SAN ARDO, SAN BENITO, SAN LUCAS, SOLEDAD, AND SPRECKLES, AND IV AT BEN LOMOND, CAMBRIA, CARMEL, CASTROVILLE, DOS PALOS, GILROY, KING CITY, LOS BANOS, MENDOTA, MONTEREY, PASO ROBLES, PRIEST, SALINAS, SAN LUIS OBISPO, TRES PINOS, WATSONVILLE, ETC.
09/27/1938	16-20--?	36.45	121.25	D			F	PAICINES AND PINNACLES.
09/29/1938	12-12--?	34.55	120.78	C	4.0			OFF POINT ARGUELLO.
10/02/1938	18-45--?	34.33	119.58	C	4.0		F	SANTA BARBARA AND SUMMERLAND.
10/24/1938	13-40--?	36.45	121.25	D			F	HOLLISTER AND PINNACLES.
10/28/1938	10-07--?	35.80	120.33	C	3.5			
11/01/1938	22-46--?	35.12	120.08	C	3.0			
11/16/1938	13-39--?	35.80	120.33	C	3.0			
11/22/1938	15-30--?	35.93	120.48	B	4.5		F	NEAR PARKFIELD; FELT AT ATASCADERO, CAMBRIA, CRESTON, MORRO BAY, PARKFIELD, PASO ROBLES, SAN MIGUEL, AND SHANDON.
01/01/1939	-?-53--?	34.58	120.33	C	3.0			
01/21/1939	07-08--?	36.45	121.25	D			F	PINNACLES.
01/22/1939	15-52--?	34.40	119.70	C	2.5			
02/05/1939	03-30--?	35.65	120.65	D			F	PASO ROBLES.
02/09/1939	06-44--?	35.93	120.48	C	3.0		F	NEAR PARKFIELD.
02/12/1939	03-12--?	34.42	119.83	B	3.0		F	GOLETA AND SANTA BARBARA .
03/24/1939	02-49--?	34.55	120.78	C	3.5			

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/25/1939	03-45--?	36.45	121.25	D			F	PINNACLES.
03/30/1939	10-11--?	34.50	119.80	C	2.5			
05/02/1939	18-49--?	35.93	120.48	C	4.0		F	IV AT PARKFIELD.
05/03/1939	07-55--?	34.55	120.78	C	3.0			
05/03/1939	12-39--?	35.65	120.65	D			F	PASO ROBLES.
05/18/1939	23--?--?	35.80	120.33	C	3.0			
06/15/1939	21-12--?	34.50	119.70	C	2.5			
06/17/1939	04-30--?	34.75	120.25	D			F	LOS ALAMOS. REPORTS OF SEVERAL SHOCKS.
06/24/1939	12-55--?	35.85	120.85	D			F	BRADLEY.
06/24/1939	13-02--?	36.40	121.00	C	5.5		F	OVER AN AREA OF 10,000 SQ. MI. IN WEST-CENTRAL CALIFORNIA, ALONG THE COAST AS FAR NORTH AS HALF MOON BAY AND SOUTH TO ESTERO BAY. INLAND IT WAS FELT AT COALINGA, TRANQUILITY, AND VOLTA, WITH A VII AT HOLLISTER, VI AT KING CITY AND PAICINES, V AT CAYUCOS, SOLEDAD, AND SPRECKLES, AND IV AT PAICINES, V AT CAYUCOS, SOLEDAD, AND SPRECKLES, AND IV AT CAMBRIA, CARMEL, CASTROVILLE, CHUALAR, GILROY, GONZALES, LOCKWOOD, MILPITAS, MONTEREY, NIPOMO, PASO ROBLES, PINNACLES, SALINAS, SAN ARDO, SAN BENITO, SAN JUAN, SAN MIGUEL, SAN SIMEON, SANTA CRUZ, TRES PINOS, AND WATSONVILLE.
07/04/1939	10-49--?	36.40	121.00	C	4.0		F	HOLLISTER, PAICINES, AND SALINAS.
07/10/1939	18-33--?	36.40	121.25	D			F	PINNACLES.
07/24/1939	09-30--?	36.25	121.80	D			F	BIG SUR.
07/24/1939	13--?--?	36.00	121.15	D			F	JOLON.
09/06/1939	01-53-43	34.58	120.42	C	3.0			
09/07/1939	02-50-30	35.42	121.08	C	3.0		F	OFF SAN LUIS OBISPO CO.; FELT AT CAMBRIA.
09/08/1939	01-57--?	34.75	120.25	D			F	LOS ALAMOS.
09/08/1939	05--?--?	34.75	120.25	D			F	LOS ALAMOS.
09/12/1939	-?--?-47	34.25	119.75	C	3.0			
09/24/1939	11-57-40	36.40	121.00	D	3.5			
10/06/1939	04-39--?	35.80	121.50	D	3.5			
10/17/1939	19-21-41	34.55	120.78	C	3.5			
10/17/1939	20-42-43	34.55	120.78	C	4.0			OFF POINT ARGUELLO.
11/02/1939	14-02--?	34.40	120.50	D			F	POINT CONCEPTION LIGHT STATION.
11/04/1939	14-11-33	36.20	120.90	D	3.0		F	SALINAS AND SAN LUCAS.
12/14/1939	03-45-18	36.10	120.00	D	3.0			
12/25/1939	15-36-23	34.28	119.83	C	3.5			
12/28/1939	12-15-38	35.80	120.33	B	5.0		F	OVER AN AREA OF 15,000 SQ. MI. IN WEST-CENTRAL CALIFORNIA, ON THE COAST FROM SANTA CRUZ SOUTH TO POINT ARGUELLO, AND INLAND TO LOST HILLS AND FRESNO. V AT COALINGA, FRESNO, GREENFIELD, PRIEST, SAN ARDO, AND SAN LUCAS, AND IV AT APTOS, ATASCADERO, BIG SUR, CAMBRIA, CARMEL, CASTROVILLE, CAYUCOS, CHUALAR, GONZALES, HOLLISTER, KING CITY, MENDOTA, MONTEREY, MORRO BAY, PARKFIELD, PASO ROBLES, PINNACLES, SALINAS, SAN JUAN BAUTISTA, SAN LUIS OBISPO, SANTA CRUZ, SOLEDAD, TAFT, ETC.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/29/1939	04--?--?	36.40	121.25	D			F	PINNACLES.
12/30/1939	15-24-37	35.80	120.33	D	3.5		F	NEAR PARKFIELD. FELT AT SAN LUCAS.
02/27/1940	11-40-25	34.25	119.50	B	3.0			
05/21/1940	10-05-34	35.28	120.48	B	4.0		F	ATASCADERO, CAMBRIA, CAYUCOS, MORRO BAY, PASO ROBLES, PISMO BEACH, AND SAN LUIS OBISPO.
06/16/1940	09-25-04	34.55	120.78	C	4.0		F	OFF POINT ARGUELLO; FELT AT GUADALUPE AND LOS ALAMOS.
06/26/1940	08-56--?	36.08	120.32	C	3.5			
06/28/1940	04-06-42	34.55	120.78	C	3.0			
08/13/1940	22-07-29	36.23	120.32	B	4.0			(DEPT. OF WATER RESOURCES DATA.)
08/31/1940	08-52-46	34.55	120.78	B	3.5			
09/07/1940	10-36-30	36.50	121.50	D	3.5			
09/07/1940	10-38-36	36.50	121.50	D	3.5			
09/07/1940	13-02-06	36.50	121.50	D	4.5		F	CARMEL AND SALINAS.
10/20/1940	22-18-45	34.55	120.78	C	3.0			
11/10/1940	10-25-10	34.35	119.77	C	4.0		F	SANTA BARBARA CHANNEL; FELT AT GOLETA, PARADISE CAMP, AND SANTA BARBARA.
11/17/1940	21-23-43	35.00	119.50	C	3.0			
01/29/1941	08-54-01	34.48	119.53	B	3.0			
02/04/1941	03-19-12	34.55	119.68	C	3.0			
02/04/1941	03-42-09	34.55	119.68	C	3.0			
02/08/1941	15-58-50	34.55	119.68	C	3.5		F	SANTA BARBARA.
02/09/1941	23-49-18	34.50	119.70	C	2.0			
02/11/1941	06-43-30	34.27	119.57	B	3.5		F	SANTA BARBARA.
02/12/1941	20-10-24	34.40	119.70	C	3.0			
02/14/1941	22-19-06	34.40	119.70	C	2.5			
05/07/1941	16-17-34	34.55	120.78	C	3.5			
05/15/1941	03-29--?	36.15	120.35	D			F	COALINGA.
05/15/1941	06--?--?	36.15	120.35	D			F	COALINGA.
07/01/1941	07-50-57	34.33	119.58	A	6.0		F	SANTA BARBARA; FELT OVER AN AREA OF 20,000 SQ. MI. VIII AT CARPINTERIA AND SANTA BARBARA, VII AT GOLETA AND VENTURA, VI AT FILLMORE, KEYSTONE, LOS ALAMOS, OJAI, OXNARD, PORT HUENEME, SANTA PAULA, SUMMERLAND, AND WHEELER SPRINGS, AND V AT ACTON, ALTADENA, ARLIGHT, ARTESIA, ARVIN, BETTERAVIA, BUELLTON, BURBANK, CAMARILLO, CANOGA PARK, CASMALIA, CAYUCOS, CHATSWORTH, COMPTON, EL SEGUNDO, GAVIOTA, GLENDALE, HERMOSA BEACH, INGLEWOOD, LA CRESCENTA, LAGUNA BEACH, LANCASTER, LOMITA, LOMPOC, LONG BEACH, LOS ANGELES, LOS OLIVOS, MAYWOOD, MCKITTRICK, MONTALVO, MOORPARK, NEWBURY PARK, NEWPORT, NIPOMO, NORTH HOLLYWOOD, OCEANO, ORCUTT, PASADENA, PATTIWAY, IRU, POINT CONCEPTION, SANDBERG, SAN NICHOLAS ISLAND, SAN PEDRO, SANTA ANA, SANTA MARIA, SANTA MONICA, SANTA YNEZ, SIERRA MADRE, SIMI, STANTON, SUNLAND, SURF, TEHACHAPI, UPPER SESPE MOUNTAINS, VALYERMO, WHEELER RIDGE, AND WHITTIER.
07/01/1941	07-57--?	34.33	119.58	B	3.0			
07/01/1941	07-58--?	34.33	119.58	B	3.5			

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/01/1941	08-05--?	34.33	119.58	B	3.0			
07/01/1941	08-07--?	34.33	119.58	B	3.0			
07/01/1941	08-10--?	34.33	119.58	B	3.0			
07/01/1941	08-13--?	34.33	119.58	B	3.0			
07/01/1941	08-15--?	34.33	119.58	B	3.0			
07/01/1941	08-19--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57 (THIS DATE).
07/01/1941	08-21--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	08-25--?	34.33	119.58	B	3.5			
07/01/1941	08-30--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/01/1941	08-48--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	08-58--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	09-05--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	09-45--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	10-25--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	12-37--?	34.33	119.58	B	3.0			
07/01/1941	14-22--?	34.33	119.58	B	3.0			
07/01/1941	18-13--?	34.33	119.58	B	3.0			
07/01/1941	18-20--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	19-48--?	34.33	119.58	B	3.0			
07/01/1941	20-15--?	34.33	119.58	B	3.5			
07/01/1941	22-51--?	34.33	119.58	B	3.5			
07/01/1941	23-54--?	34.33	119.58	B	4.5		F	AFTERSHOCK OF 07-50-57; FELT AT FILLMORE, GAVIOTA, LOS ALAMOS, AND SANTA BARBARA.
07/02/1941	-?-17--?	34.33	119.58	B	3.0			
07/02/1941	04-33--?	34.33	119.58	B	3.5			
07/02/1941	08-45--?	34.33	119.58	B	3.5			
07/02/1941	11-41--?	34.33	119.58	B	3.0			
07/02/1941	22-19--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/03/1941	-?-25--?	34.33	119.58	B	3.5			
07/03/1941	19-26--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/07/1941	01-06--?	34.33	119.58	B	3.0			
07/07/1941	06-25--?	34.33	119.58	B	3.5			
07/08/1941	19-37--?	34.33	119.58	B	3.0			
07/12/1941	16-18--?	34.33	119.58	B	4.5		F	AFTERSHOCK OF 07-50-57; FELT AT FILLMORE, GLENDALE, MONTROSE, SATICOY, SAUGUS, AND WHEELER SPRINGS.
07/12/1941	16-41--?	34.33	119.58	B	3.0			
07/12/1941	21-07--?	34.33	119.58	B	3.0			
07/12/1941	21-12--?	34.33	119.58	B	3.0			
07/13/1941	06-11--?	34.33	119.58	B	3.5			
07/16/1941	23-10--?	34.33	119.58	B	3.0			
07/17/1941	18-31--?	34.33	119.58	B	3.0			
07/27/1941	12-44--?	34.33	119.58	B	3.0			
07/31/1941	13-23--?	34.33	119.58	B	3.0			
08/02/1941	12-31-19	34.33	119.58	C	3.0			
08/09/1941	05-05-24	34.33	119.58	C	3.5			

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
08/12/1941	22-35-24	34.33	119.58	C	3.5			
08/19/1941	10-20-25	34.33	119.58	C	3.0			
08/25/1941	06-58-22	34.33	119.58	C	3.0			
08/27/1941	17-11-02	34.33	119.58	C	3.0			
08/29/1941	08-43-24	34.60	120.30	C	3.0			
09/08/1941	03-12-45	34.33	119.58	B	4.5		F	AFTERSHOCK OF 07/01/41, 07-50-57. V AT GOLETA AND SANTA BARBARA; FELT STRONGLY AT LOS ALAMOS AND SUMMERLAND.
09/08/1941	03-14-23	34.33	119.58	B	4.0		F	TWIN SHOCK OF 03-12-45; SAME "FELT" REPORT.
09/08/1941	04-45-16	34.33	119.58	B	3.5		F	SANTA BARBARA.
09/09/1941	03-23-17	34.33	119.58	B	3.5		F	SANTA BARBARA.
09/09/1941	13-44-46	34.33	119.58	B	3.0			
09/14/1941	01-45-18	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07/01/41, 07-50-57.
09/14/1941	02-20-42	34.33	119.58	B	3.0			
09/15/1941	01-37-02	34.33	119.58	B	4.0		F	GOLETA, SANTA BARBARA, AND SUMMERLAND.
09/15/1941	01-55-18	34.33	119.58	B	3.0			
09/15/1941	02-49-06	34.33	119.58	B	3.5			
09/16/1941	07-27--?	34.33	119.58	B	3.5			
09/25/1941	05-12-56	34.33	119.58	B	4.0		F	GOLETA AND SANTA BARBARA.
10/07/1941	12-05-42	34.33	119.58		3.0			
10/19/1941	23-22-19	34.33	119.58	B	3.0			
11/05/1941	16-36--?	35.00	121.00	D	3.5		F	OFF POINT CONCEPTION; FELT AT SAN SIMEON.
11/17/1941	17-30-27	34.33	119.58	C	3.0			
11/18/1941	18-08-10	34.33	119.58	C	4.0		F	CARPINTERIA AND SANTA BARBARA.
11/21/1941	16-56-03	34.33	119.58	C	4.0		F	GOLETA AND SANTA BARBARA.
11/25/1941	20-01-48	34.33	119.58	C	3.0			
11/28/1941	06-33--?	35.00	120.00	D	3.5			
12/08/1941	-?-29-42	36.00	121.00	D	3.5			
12/22/1941	-?-54-09	35.93	120.48	C	4.0		F	NEAR PARKFIELD-NOT RECORDED ON BERKELEY NETWORK.
01/06/1942	09-20--?	36.15	120.65	D			F	PRIEST VALLEY-RECORDED AT TINEMAHA.
01/06/1942	09-23--?	36.15	120.65	D			F	PRIEST VALLEY-RECORDED AT TINEMAHA.
01/08/1942	18-21-05	34.13	119.58	C	2.5			
01/18/1942	11-35--?	36.40	121.25	D			F	PINNACLES.
01/18/1942	16-50--?	36.40	121.25	D			F	PINNACLES.
02/19/1942	18-33--?	36.40	121.25	D			F	PINNACLES.
03/09/1942	05-57-42	34.30	119.60		3.0			
03/25/1942	-?--?--?	36.40	121.25	D			F	PINNACLES; LIGHT SHOCK.
04/19/1942	04-02-47	34.30	119.60	D	3.0			
04/22/1942	05-32-52	35.30	119.50	D	3.0			
05/08/1942	17-19-13	34.33	119.58	C	3.0			
06/06/1942	06-42-11	34.35	119.85	C	3.0		F	GOLETA.
06/29/1942	21-07-30	35.60	120.80	D	4.0		F	IV AT CAMBRIA AND SAN LUIS OBISPO.
07/19/1942	10-42-07	36.40	121.10	D	1.6			SW OF LLANADA.
09/15/1942	10-36-33	36.13	122.18	B	3.0			SW OF KING CITY.
10/04/1942	10--?--?	34.60	120.00	D			F	IV AT SANTA YNEZ PEAK.
10/11/1942	23-48-23	36.48	121.40	C	1.9			FORESHOCK OF QUAKE ON OCTOBER 15 AT 13-53-56.
10/15/1942	13-53-56	36.48	121.40	B	4.3		F	IV AT BIG SUR, GONZALES, GREENFIELD, HOLLISTER, SALINAS, AND SOLEDAD.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
10/18/1942	08--?--?	36.00	121.00	D			F	CAMBRIA.
10/18/1942	12-01-42	36.00	121.00	D			F	V AT CAMBRIA.
10/19/1942	10-23--?	34.50	119.65	D			F	V AT SANTA BARBARA.
10/20/1942	10-25--?	36.00	121.00	D			F	V AT CAMBRIA.
10/26/1942	01-09-01	36.40	121.60	D	1.8			DEPTH ABOUT 12 KM.
12/02/1942	11-46--?	34.33	119.58	C	3.5		F	V AT SANTA BARBARA.
12/06/1942	16-57-49	35.93	120.48	C	3.5			
01/24/1943	06-55-57	34.33	119.58	C	3.0			
03/16/1943	09-27-47	34.28	119.60	C	3.0			
04/01/1943	13-39-66	34.68	121.75	B	3.1			OFF COAST, WEST OF POINT ARGUELLO.
06/29/1943	02-50-53	36.50	121.10	D	3.1			SW OF LLANADA.
07/05/1943	16-30-29	36.38	121.83	C	3.9			SOUTH OF SALINAS.
07/15/1943	-?-44-42	36.00	120.15	D			F	NEAR AVENAL.
08/07/1943	16-59-47	34.28	119.57	C	3.5			
08/12/1943	15-56-33	34.75	121.15	C	3.5			
08/27/1943	08-16-53	34.43	119.87	C	3.5		F	IV AT SANTA BARBARA.
09/13/1943	12-40--?	35.65	120.65	D			F	PASO ROBLES, POSSIBLY GUN FIRE.
09/18/1943	17-07-16	34.37	119.58	C	3.0			
10/22/1943	12--?--?	36.00	120.90	D			F	SAN ARDO; 2 SHOCKS.
10/26/1943	22-10--?	34.75	120.25	D			F	LOS ALAMOS.
10/31/1943	17-54-06	35.80	120.40	D	3.5			
10/31/1943	20--?--?	36.40	121.00	D			F	LONOAK.
11/08/1943	11-33-46	36.00	119.92	C	3.0		F	KETTLEMAN HILLS; FELT AT AVENAL.
11/30/1943	21-57-18	36.30	120.50	D	4.0			NEAR COALINGA.
12/01/1943	04-51--?	36.50	121.10	D			F	SAN BENITO.
01/04/1944	18-06-40	34.10	120.40	D	3.3			
02/18/1944	16-29-37	34.10	119.52	C	2.1			
02/21/1944	13--?-11	36.17	120.93	C	3.8			WEST OF PRIEST.
03/06/1944	21-32-16	36.40	121.25	C	3.4			NE OF PARAISO.
04/03/1944	02-33--?	34.50	121.40	D	4.0			OFF POINT ARGUELLO.
04/12/1944	15-33-10	34.27	119.52	C	4.0		F	OFF CARPINTERIA; FELT EAST OF SANTA BARBARA.
06/13/1944	08-27-32	34.67	120.50	C	4.6		F	NEAR LOMPOC; VI AT LOS ALAMOS AND IV AT SANTA MARIA.
06/13/1944	08-46-43	34.67	120.50	C	4.0		F	AFTERSHOCK OF 08-27-32.
06/13/1944	11-07-24	34.67	120.50	C	4.4		F	AFTERSHOCK OF 08-27-32.
07/11/1944	22-33--?	36.50	121.10	D			F	SAN BENITO.
07/15/1944	19-22-37	34.37	119.62	C	3.1			
09/04/1944	02-47-46	35.00	120.00	D	3.4		F	LOS ALAMOS.
09/04/1944	05--?--?	35.00	120.00	D			F	LOS ALAMOS.
09/15/1944	14-12-42	34.70	120.20	D	2.6		F	KETTLEMAN HILLS REGION; FELT AT PARKFIELD.
09/18/1944	01-30--?	35.00	120.00	D	3.5			
11/04/1944	08-12-01	36.33	120.08	C	3.4			
11/08/1944	16-12-36	34.33	119.72	C	3.1			
11/28/1944	10-36--?	35.80	120.00	D	3.3			
11/30/1944	18-53-15	34.72	120.42	C	4.1		F	NEAR LOS ALAMOS; FELT AT LOS ALAMOS AND LOS OLIVOS.
12/02/1944	15-09-12	35.80	120.00	D	3.2			
01/27/1945	17-50-31	34.75	120.67	C	3.9			
02/25/1945	20-18-38	36.00	120.48	C	3.6			

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
04/15/1945	22-59-57	34.13	119.83	C	3.1			
06/11/1945	03-54-52	34.50	120.80	D	3.2			
07/11/1945	16-13--?	35.67	121.25	D	4.0		F	NEAR SAN SIMEON; IV AT CAMBRIA.
07/28/1945	02-33-48	34.70	120.10	D	4.2		F	EAST OF SANTA MARIA; IV AT LOS ALAMOS.
09/04/1945	12-38-31	34.32	119.63	C	3.2			
09/07/1945	11-34-20	35.83	120.75	C	4.2		F	NEAR BRADLEY; IV AT CAMBRIA, PARKFIELD, PASO ROBLES, AND SAN MIGUEL.
11/04/1945	-?-46-34	36.38	121.28	C	3.3			NEAR SOLEDAD.
02/09/1946	02-55-28	34.33	119.92	C	2.5			
02/10/1946	11-01-19	36.50	121.00	D	4.2		F	OVER AN AREA OF 2000 SQ. MI. IN WEST CENTRAL CALIFORNIA. V AT SAN BENITO, AND IV AT BIG SUR, CHUALAR, GREENFIELD, HOLLISTER, LONOAK, SAN LUCAS, SAN MIGUEL, SANTA CRUZ, AND SOLEDAD.
02/15/1946	12-07-00	35.90	121.45	D			F	PARKFIELD; LIGHT SHOCK.
04/19/1946	12-50--?	34.00	120.40	D			F	SANTA MARIA.
07/08/1946	19-59-44	34.83	120.53	C	3.2			
08/06/1946	04-55-07	34.95	120.18	C	2.8		F	E OF SANTA MARIA; FELT AT LOS ALAMOS.
09/02/1946	10-09-47	34.18	119.62	C	3.0			
09/09/1946	11-20--?	34.90	120.40	D			F	SANTA MARIA.
09/19/1946	06-35-44	35.83	119.67	C	3.2			
10/24/1946	18-26-50	34.37	119.62	C	2.7			
11/22/1946	09-47-59	34.83	120.68	D	3.0			
11/27/1946	14-44-51	35.50	120.92	C	4.3		F	NEAR CAYUCOS; V AT MORRO BAY AND SANTA MARGARITA; ALSO FELT ATASCADERO, LOS ALAMOS, PISMO BEACH, AND SAN LUIS OBISPO.
12/13/1946	-?-40-01	34.17	119.53	C	3.5			
01/06/1947	21-05-47	35.85	120.47	C	3.6			
01/13/1947	19-38-31	34.32	119.65	C	2.2			
01/14/1947	20-49-27	34.23	119.65	C	2.7			
01/18/1947	12--?-42	34.20	121.50	D	3.3			
01/19/1947	19-32--?	35.60	120.30	D	3.1		F	PASO ROBLES.
02/05/1947	06-14--?	38.23	120.65	B	5.0		F	VI AT LONOAK, V AT COALINGA, IDRIA, AND KING CITY, AND IV AT BIG SUR, HURON, PARKFIELD, SAN ARDO, AND WESTHAVEN.NEAR COALINGA - AFTERSHOCK OF 2/5/47 OF 06-14--?.
02/25/1947	11-45-18	36.20	120.50	D	4.2			
03/23/1947	16-04-51	35.15	121.30	D	3.7			
03/27/1947	09-16-46	35.00	121.00	D	4.2		F	OFF COAST; V AT LOMPOC.
04/29/1947	07-44--?	34.33	119.55	C	3.2			
06/25/1947	18-39-53	34.25	119.50	C	3.1		F	NEAR CARPINTERIA.
06/25/1947	13-41-21	34.25	119.50	C	3.6		F	NEAR CARPINTERIA.
06/25/1947	18-48-26	34.25	119.50	C	2.5			
06/25/1947	20-55-16	34.25	119.50	C	3.2		F	NEAR CARPINTERIA.
06/25/1947	20-55-54	34.25	119.50	C	3.8			
07/13/1947	05-35--?	36.08	121.10	D	3.4			SOUTH OF KING CITY.
07/14/1947	05-40-06	35.92	119.92	C	4.0		F	KETTLEMAN HILLS; IV AT KETTLEMAN CITY.
10/6/1947	18-39--?	36.50	121.23	A	3.2			EAST OF GONZALES.
12/14/1947	05-42--?	36.45	121.08	B	3.4			SW OF LLANADA.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/16/1947	09-21-03	36.25	120.77	C	3.6		F	IV AT SAN LUCAS.
12/18/1947	19-30-06	36.12	120.90	D			F	IV AT PARKFIELD.
12/25/1947	06-05--?	35.60	121.10	D			F	CAMBRIA.
12/25/1947	06-20--?	35.60	121.10	D			F	CAMBRIA.
01/11/1948	05-37-28	36.43	121.48	B	4.3		F	IV AT HOLLISTER.
02/01/1948	17--?-54	34.42	119.92	C	3.0			
02/15/1948	08-04-06	35.88	120.37	A	3.4			EAST OF PARKFIELD.
03/07/1948	07-46-22	36.10	120.40	D	3.0			NEAR COALINGA.
03/10/1948	23-24-34	34.43	119.73	C	2.6			
03/18/1948	09-35-05	34.40	119.60	C	2.8			
03/29/1948	02-40--?	35.85	121.40	D			F	IV AT HOLLISTER.
04/23/1948	15-23-43	34.10	120.93	C	3.7			
05/05/1948	06-47-06	34.45	119.72	B	2.7			
05/07/1948	12--?-32	36.20	121.90	D	3.0			WEST OF PRIEST.
05/09/1948	11-10--?	34.75	120.25	D			F	V AT LOS ALAMOS.
07/14/1948	11-05-37	34.67	120.92	C	3.2			
07/17/1948	05-26-31	34.55	120.05	C	3.4			
07/28/1948	01-30-57	36.05	120.53	C	3.1			SE OF PRIEST.
07/29/1948	13-16-23	35.12	120.47	C	3.4			
08/04/1948	10-22-57	35.92	120.33	C	3.6			
09/03/1948	23-42-26	34.33	119.53	C	3.9		F	SANTA BARBARA.
09/17/1948	15-41-01	34.40	119.62	C	3.1			
10/27/1948	03-05--?	34.75	120.25	D			F	IV AT LOS ALAMOS.
10/29/1948	03-04-59	34.10	120.40	D	3.4		F	V AT ARLIGHT AND POINT ARGUELLO LIGHT STATION.
11/02/1948	19-06-45	34.37	119.58	C	2.9			
12/04/1948	06-44-20	34.43	119.72	C	2.8			
12/04/1948	23-32-51	34.42	119.50	C	2.7			
12/20/1948	04-42-46	35.80	121.50	C	4.5		F	OFF COAST, NEAR PIEDRAS BLANCAS POINT; III AT SAN SIMEON.
12/31/1948	14-35-46	35.67	121.40	B	4.6		F	ALONG THE COAST FROM LOMPOC TO MOSS LANDING; VI AT SAN SIMEON AND V AT CAYUCOS, CRESTON, MOSS LANDING, AND PIEDRAS BLANCAS LIGHT STATION.
01/25/1949	04-29--?	34.90	120.40	D			F	V AT ORCUTT AND SANTA MARIA.
03/27/1949	06-31-16	34.25	119.62	C	2.6			
04/06/1949	14-07--?	35.00	120.00		2.6			
04/08/1949	13-17-07	34.60	120.35	C	3.2		F	IV AT LOS ALAMOS.
04/14/1949	01-46-12	34.28	119.52	C	2.6			
04/23/1949	09-18-09	36.38	121.37	C	3.7			NORTH OF PARAISO.
05/06/1949	04-23-46	34.50	121.00	C	3.4			
05/10/1949	06-20--?	35.90	120.40	D			F	SANTA MARIA - SLIGHT.
05/10/1949	11--?--?	35.90	120.40	D			F	SANTA MARIA - SLIGHT.
05/16/1949	03-01-03	34.72	120.02	C	3.2			
05/17/1949	23-57-55	35.63	121.15	D	4.1		F	IV AT SAN SIMEON.
06/27/1949	10-35-31	35.80	121.10	D	4.5		F	V AT SAN ARDO AND SAN MIGUEL; ALSO FELT AT PASO ROBLES, SAN LUIS OBISPO, AND SANTA MARGARITA.
07/21/1949	16-50--?	36.15	120.35	D			F	IV AT COALINGA.
07/21/1949	17-01--?	36.15	120.35	D			F	IV AT COALINGA.
07/24/1949	03-04-05	36.00	120.00	D	2.3			SE. KINGS C0. AFTER SHOCK AT 06-26--?, MAG. 2.0.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/27/1949	18-21-35	34.53	120.37	C	3.6			
08/01/1949	-?-07-24	36.90	121.20	D	3.0			SOUTH OF KING CITY.
08/07/1949	01-38-43	36.50	121.50	D	2.3			NO. MONTEREY CO.
08/10/1949	09-17-39	36.50	121.00	C	2.6			CENTRAL SAN BENITO CO.
08/22/1949	03--?--?	36.00	120.00	D			F	KETTLEMAN HILLS. FIFTH SHOCK IN 2 WEEKS.
08/26/1949	16-52-32	34.50	120.50	D	4.2		F	NEAR POINT CONCEPTION. VI AT ARLIGHT AND SURF. IV AT GUADALUPE, LOMPOC, AND LOS ALAMOS.
08/27/1949	14-15--?	34.50	120.50	D			F	ARLIGHT. SLIGHT SHOCK.
08/27/1949	14-51-46	34.50	120.50	D	4.9		F	NEAR POINT CONCEPTION. VI AT ARLIGHT, LOMPOC, AND SUDDEN. V AT COSMALIA, LOS ALAMOS, NIPOMO, SANTA BARBARA, AND SURF.
08/29/1949	12-07-20	36.00	120.10	D	3.0		F	IV IN AVENAL AND KETTLEMAN CITY.
10/28/1949	08-07-02	36.80	120.90	C	2.6			NW OF PRIEST.
11/17/1949	05-06-06	34.80	120.70	D	2.8		F	IV AT SANTA MARIA.
12/28/1949	09-17-12	36.20	120.70	D	2.6			NEAR PRIEST.
02/19/1950	08-29-44	34.50	120.70	D	3.5			
03/09/1950	23-43-19	36.35	121.22	C	3.2		F	NORTH OF KING CITY; V AT ROBLES DEL RIO.
03/22/1950	01-31-57	35.97	120.63	C	3.7			
03/29/1950	12-43-20	35.97	120.88	D	3.5			
04/15/1950	11-56-32	35.75	119.62	C	4.6		F	NE OF LOST HILLS; V AT ASH MOUNTAIN, (SEQUOIA NATIONAL PARK), KERNVILLE, AND SHAFTER, AND IV AT BUTTONWILLOW, JAWBONE AQUEDUCT STATION, LOST HILLS, THREE RIVERS, AND VISALIA.
04/21/1950	13-17-29	34.38	119.58	B	3.0		F	IV AT SANTA BARBARA.
04/26/1950	07-23-29	35.20	120.60	C	3.5		F	V AT SANTA MARIA; ALSO FELT AT ORCUTT.
04/26/1950	07-38--?	35.20	120.60	D			F	SANTA MARIA.
05/21/1950	18-59-03	34.57	119.63	C	2.6			
05/21/1950	19-26-48	35.88	119.73	C	3.4			
05/24/1950	01-46-57	36.43	120.77	C	2.9			SE OF LLANADA.
07/13/1950	15-01-47	34.33	119.50	C	2.8		F	OFF CARPINTERIA; V AT MONTECITO; ALSO FELT AT SANTA BARBARA AND NEARBY AREAS.
08/01/1950	21-08-43	36.20	122.23	B	2.0			OFF COAST, WEST OF BIG SUR.
08/02/1950	06-50-48	34.67	120.63	C	3.3			
08/23/1950	09-10--?	34.40	119.50	D			F	IV AT RINCON POINT; FELT AT CARPINTERIA.
09/24/1950	04-45--?	34.50	120.50	D			F	III AT ARLIGHT.
09/24/1950	12-23--?	34.22	119.58	C	3.3			
09/24/1950	21-51-44	36.20	120.50	D	2.9			EAST OF PRIEST.
10/20/1950	08-23-25	36.33	121.07	C	2.7			SOUTH OF KING CITY.
11/21/1950	04-30--?	30.90	120.40	D			F	SANTA MARIA.
03/02/1951	02-13-44	36.10	120.60	D	3.1			SE OF PRIEST
03/04/1951	13-32--?	34.90	120.40	D			F	IV AT SANTA MARIA; 2 SHOCKS.
03/05/1951	09-50--?	34.90	120.40	D			F	IV AT SANTA MARIA.
03/10/1951	05-35--?	34.50	120.50	D			F	IV AT ARLIGHT.
03/15/1951	13-50-43	35.02	120.48	C	3.8		F	IV AT LOS ALAMOS.
03/26/1951	06-07-34	34.62	119.50	C	3.5		F	IV AT OJAI AND SUMMERLAND; FELT AT VENTURA.
05/04/1951	03-28-36	36.20	120.20	D	3.1			FORESHOCK OF QUAKE AT 20-08-10.
05/04/1951	20-08-10	36.20	120.20	D	3.2			EAST OF COALINGA.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
05/06/1951	03-18-03	36.40	120.40	D	2.8			NORTH OF COALINGA.
05/25/1951	05-11-18	36.30	120.30	D	3.1			NORTH OF COALINGA.
05/29/1951	05-08-24	35.08	119.65	C	3.2		F	ELKHORN HILLS; IV IN CUYAMA VALLEY.
05/31/1951	06-28-42	36.30	120.20	D	2.7			NE OF COALINGA.
06/16/1951	19-01-17	34.40	120.08	C	3.3			
06/19/1951	06-13-47	35.97	120.42	C	3.6			SOUTH OF COALINGA.
07/01/1951	-?-13-19	36.20	120.95	B	3.2			EAST OF KING CITY.
07/07/1951	05-53-33	34.75	120.75	C	3.5			
08/02/1951	05-09-25	36.35	121.27	B	3.9		F	NEAR GREENFIELD; IV AT BIG SUR, AT 7 MI. S OF HOLLISTER, AND ROBLES DEL RIO.
08/08/1951	19-42--?	34.80	120.40	D			F	IV AT ORCUTT.
08/09/1951	09-20-48	36.15	121.75	C	2.2			NEAR BIG SUR.
08/25/1951	01-04-10	36.47	121.15	B	3.1			SW OF LLANADA.
08/28/1951	22-12-27	34.60	121.00	D	3.5		F	OFF POINT ARGUELLO; III AT LOS ALAMOS.
09/18/1951	02-30--?	36.25	121.80	D			F	IV AT BIG SUR.
09/19/1951	22-50--?	36.25	121.80	D			F	IV AT BIG SUR.
10/03/1951	13-44-33	35.92	120.52	C	3.8			
10/26/1951	16-25-40	34.42	119.73	C	3.0			
11/17/1951	03-19-48	34.70	120.50	D	2.5		F	NEAR LOMPOC; III AT LOS ALAMOS.
11/25/1951	23-15-39	35.33	119.50	B	3.8			
12/20/1951	04-13-06	36.00	120.05	C	3.7			
01/24/1952	-?-32-38	34.18	119.88	C	2.7			
01/30/1952	11-05-33	36.30	121.13	C	2.7			NEAR KING CITY.
01/31/1952	20-09-02	34.18	119.53	C	2.6			
01/31/1952	21-33-12	36.40	121.40	C	3.6			SOUTHEAST CF SOLEDAD.
02/09/1952	22-26-39	34.07	120.75	C	3.6			
03/25/1952	09-18-50	34.18	120.95	C	3.6			
04/02/1952	05-21-10	36.45	121.25	B	3.1			NEAR SOLEDAD.
05/07/1952	05-45--?	34.40	119.60	D			F	IV AT MONTECITO AND SUMMERLAND.
06/18/1952	04--?--?	34.60	120.65	D			F	IV AT POINT ARGUELLO LIFEBOAT STATION.
07/01/1952	15-29-24	34.30	119.80	D	3.1			
07/15/1952	06-07-55	36.42	121.00	C	2.5			ABOUT 15 MI. NE OF KING CITY.
07/27/1952	18-15-14	34.18	119.70	C	3.1			
07/27/1952	20-20-35	34.22	119.67	C	3.2			
07/27/1952	20-30-05	34.20	119.67	B	3.5			
08/07/1952	19-l6-12	34.33	120.68	C	3.6		F	OFF POINT CONCEPTION; IV AT LOS ALAMOS.
08/11/1952	21-42-29	34.17	119.67	C	3.1			
08/23/1952	20-10--?	34.85	119.50	D			F	IV AT VENTUCOPA - SECOND SHOCK AT 21-20--?.
08/30/1952	14-58-11	34.35	119.62	B	3.3			
09/01/1952	12-03--?	34.30	119.60	D	3.0			
09/12/1952	21--?-15	34.25	119.70	C	3.0			
09/14/1952	11-46-06	35.90	120.30	D	3.3			
10/09/1952	14-46-02	34.20	122.20	D	4.6			(DEPT. OF WATER RESOURCES DATA)

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
11/22/1952	07-46-37	35.73	121.20	B	6.0		F	6 MI. NORTH OF SAN SIMEON, NEAR BRYSON; FELT OVER AN AREA OF 20,000 SQ. MI. VII AT BRADLEY AND BRYSON, VI AT ARROYO GRANDE, ATASCADERO, CAMBRIA, CAMP COOKE, CARMEL VALLEY, CAYUCOS, CHUALAR, CRESTON, GORDA STATION, GUADALUPE, HARMONY, HEARST RANCH, KING CITY, LOCKWOOD, LONOAK, MORRO BAY, OCEANO, PARKFIELD, PASO ROBLES, PISMO BEACH, SALINAS, SAN ARDO, SAN LUIS OBISPO, SAN SIMEON, SANTA MARGARITA, AND TEMPLETON, AND V AT AVENAL, BEN LOMOND, BIG SUR, BUELLTON, BUTTONWILLOW, CARUTHERS, CASMALIA, CHOLAME, COALINGA, CORCORAN, DOS PALOS, HOLLISTER, HUASNA, KETTLEMAN CITY, LOMPOC, LOST HILLS, LUCIA, MARICOPA, MONTEREY, MOSS LANDING, NIPOMO, ORCUTT, PAICINES, RIVERDALE, SAN MIGUEL, SANTA CRUZ, SANTA MARIA, SHAFTER, STRATFORD, SUDDEN, AND SURF.
11/22/1952	08-02-40	35.73	121.20	B	3.2			SAN SIMEON AFTERSHOCK.
11/22/1952	08-29-47	35.73	121.20	B	3.1			SAN SIMEON AFTERSHOCK.
11/22/1952	08-53-04	35.73	121.20	B	3.4		F	SAN SIMEON AFTERSHOCK; IV AT ARVIN, CALIENTE, JOLON, LOST HILLS, MALIBU, MARICOPA, MCFARLAND, MIRACLE HOT SPRINGS, MORGAN HILL, NIPOMO, PISMO BEACH, AND SHAFTER.
11/22/1952	11-08-44	35.73	121.20	B	3.1			SAN SIMEON AFTERSHOCK.
11/22/1952	11-45-31	35.73	121.20	B	3.1			SAN SIMEON AFTERSHOCK.
11/22/1952	12-34-44	35.73	121.20	B	3.0			SAN SIMEON AFTERSHOCK.
11/22/1952	13-37-31	35.73	121.20	B	4.0		F	SAN SIMEON AFTERSHOCK; V AT CALIENTE, MIRACLE HOT SPRINGS, AND WHEELER SPRINGS.
11/22/1952	19-25-21	35.73	121.20	B	3.9			SAN SIMEON AFTERSHOCK.
11/22/1952	19-36-27	35.70	121.20	D	3.1			SAN SIMEON AFTERSHOCK.
11/22/1952	23-39-20	35.70	121.20	D	3.1			SAN SIMEON AFTERSHOCK.
11/23/1952	09-22-35	36.00	120.90	D	3.2			20 MI. SE OF KING CITY.
11/23/1952	18-40-19	35.67	121.17	C	4.2			SAN SIMEON AFTERSHOCK.
11/25/1952	19-17-54	36.20	120.00	D	3.2			
11/25/1952	20-14-45	35.73	121.20	C	3.6			SAN SIMEON AFTERSHOCK.
11/25/1952	21-59-17	35.73	121.20	C	4.4			SAN SIMEON AFTERSHOCK.
11/26/1952	13-32-09	35.73	121.20	C	3.5			SAN SIMEON AFTERSHOCK.
11/27/1952	17-37-05	35.70	121.20	D	3.3			SAN SIMEON AFTERSHOCK.
11/28/1952	10-22-33	35.90	121.20	D	3.0			SAN SIMEON AFTERSHOCK.
11/29/1952	16--?--?	36.00	121.15	D			F	IV AT JOLON - TIME MAY BE 04--?--? ON 11/30/1952.
11/29/1952	23-15-58	35.70	121.20	D	3.5			SAN SIMEON AFTERSHOCK.
12/05/1952	01-05-57	36.50	120.70	D	3.0			14 MI. SE OF LLANADA.
12/06/1952	23-50--?	35.66	120.65	D			F	IV AT PASO ROBLES; FELT AT ADELAIDA.
12/12/1952	-?-27-07	36.40	120.97	B	3.0		F	17 MI. NE OF KING CITY; III AT LONOAK.
12/25/1952	16-44-10	34.40	121.40	D	3.6			
01/12/1953	13-05-18	35.80	121.10	D	3.2			14 MI. NE OF SAN SIMEON.
01/24/1953	-?--?--?	35.90	121.00	D			F	TEN SHOCKS REPORTED FELT FROM 1/24 TO 1/31 AT BRYSON (E. WEFERLING RANCH).
01/29/1953	20-31-19	35.80	121.10	D	3.1			14 MI. NE OF SAN SIMEON.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
02/03/1953	14-50-18	35.47	120.75	C	4.1		F	12 MI. NNW OF SAN LUIS OBISPO; V AT ATASCADERO, BRYSON, CRESTON, MORRO BAY, SANTA MARGARITA, AND IV AT CAYUCOS, PASO ROBLES, SAN LUIS OBISPO, AND TEMPLETON.
02/05/1953	02-54-12	35.90	121.00	D	2.8		F	IV AT BRYSON (E. WEFERLING RANCH).
02/15/1953	15-30--?	35.90	121.00	D			F	BRYSON (E. WEFERLING RANCH).
02/17/1953	08-06--?	35.90	121.00	D			F	III AT BRYSON (PLEYTO SCHOOL) - SEVERAL MILD SHOCKS REPORTED FELT DAILY SINCE SHOCK OF 11/21/52, 23-46-38 (NOT LISTED).
02/18/1953	14-10--?	35.90	121.00	D			F	BRYSON (E. WEFERLING RANCH) - MILD.
03/01/1953	18-53--?	35.90	121.00	D			F	V AT BRYSON.
03/04/1953	03-40--?	35.90	121.00	D			F	BRYSON (PLEYTO SCHOOL) - LIGHT.
03/15/1953	21--?-32	34.87	121.53	C	3.7			
03/18/1953	05-03--?	35.90	121.00	D			F	III AT BRYSON (PLEYTO SCHOOL).
03/29/1953	17-19-48	35.90	120.20	D	3.7			
04/08/1953	-?-59-20	34.80	120.60	D	3.6		F	NEAR CASMALIA; IV AT LOS ALAMOS.
04/15/1953	-?-29-10	35.83	121.07	C	3.1		F	14 MI. NNE OF SAN SIMEON; IV AT BRYSON.
04/15/1953	05-30--?	35.90	121.00	D			F	BRYSON - LIGHT.
04/29/1953	05-26-53	36.00	121.15	C	3.5		F	14 MI. S OF KING CITY - USCGS GIVES TIME AS 05-26-52, LOCATION AS N35.8 121.2W, REPORT AS NEAR BRYSON; V AT PLEYTO SCHOOL. 22 MI. NE OF KING CITY.
05/01/1953	22-16-51	36.40	120.80	D	3.0			
05/08/1953	08-15--?	34.65	120.45	D			F	III AT LOMPOC.
05/14/1953	03-36--?	36.00	120.00	D	3.3			
05/14/1953	09-36-09	35.75	121.08	B	3.7		F	9 MI. NE OF SAN SIMEON - USCGS GIVES N35.52 121.28W, OFF CAMBRIA; V AT BRYSON.
05/15/1983	07-15--?	35.90	121.00	D			F	IV AT BRYSON (PLEYTO SCHOOL).
05/28/1953	03-51-13	35.88	120.50	B	4.3		F	20 MI. SW OF COALINGA; IV AT PASO ROBLES AND III AT SAN MIGUEL.
05/28/1953	07-58-33	35.88	120.50	C	3.5		F	AFTERSHOCK OF 03-51-13; FELT AT SAN MIGUEL.
05/29/1953	10-20-16	35.90	121.20	D	2.9			20 MI. SOUTH OF KING CITY.
05/31/1953	23-51-17	36.10	120.40	D	3.2			NEAR COALINGA.
06/04/1953	11-40--?	35.50	120.50	D			F	V AT CRESTON - PROBABLY A BLAST.
06/06/1953	20-26-33	36.00	120.30	D	2.9			10 MI. SOUTH OF COALINGA.
06/19/1953	11-24-50	36.30	120.70	D	2.8			20 MI. EAST OF KING CITY.
06/22/1953	15-22-35	35.93	120.38	C	4.3		F	15 MI. WSW OF COALINGA; FELT AT COALINGA AND PASO ROBLES.
07/01/1953	22-17-20	34.60	121.35	D	3.2		F	OFF POINT ARGUELLO; IV AT POINT ARGUELLO LIGHT STATION.
08/14/1953	01-40-06	36.30	120.30	D	2.9			8 MI. NORTH OF COALINGA.
08/14/1953	09-22-50	36.50	121.20	D	2.3			20 MI. NORTH OF KING CITY.
09/02/1953	09-41-20	35.90	120.80	D	3.0			30 MI. SE OF KING CITY.
09/03/1953	11--?--?	35.50	120.50	D			F	CRESTON.
09/04/1953	03-54-25	35.90	120.32	C	3.5		F	15 MI. SOUTH OF COALINGA; IV AT CRESTON AND PASO ROBLES.
09/22/1953	07-36-58	36.40	121.20	D	3.8			NORTH OF KING CITY.
09/23/1953	06-21-51	35.70	121.10	D	3.5		F	NEAR SAN SIMEON; V AT BRYSON.
10/01/1953	03-56-15	36.25	121.83	C	3.4		F	25 MI. S OF MONTEREY; IV AT BIG SUR.
10/16/1953	03-45-35	35.95	120.53	C	3.4			SOUTHWEST OF COALINGA.
10/21/1953	16-02-38	34.32	119.70	B	4.0		F	OFF SANTA BARBARA; V AT SANTA BARBARA AND VICINTIY, AND IV AT GOLETA AND LOS PRIETOS RANGER STATION.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
10/24/1953	13-24-30	35.90	121.10	D	3.6			SOUTH OF KING CITY.
10/25/1953	08-43-25	36.50	121.50	D	3.2			NORTHWEST OF KING CITY.
11/02/1953	-?-52-06	36.40	121.30	D	3.4			NORTHWEST OF KING CITY.
01/04/1954	23-03-11	36.12	120.63	B	3.2			14 MI. WEST OF COALINGA.
01/05/1954	-?-23-23	35.93	120.00	C	3.0			SOUTHEAST OF COALINGA.
01/15/1954	22-02-18	36.50	121.23	C	2.6			NORTH OF KING CITY.
01/24/1954	19-06-45	35.78	121.08	C	3.1			30 MI. SOUTH OF KING CITY.
01/26/1954	09-43-22	34.50	120.33	C	3.8		F	W OF LAS CRUCES; III AT SANTA YNEZ.
03/09/1954	19-55-30	35.90	120.50	D	3.6		F	16 MI. SSE OF COALINGA; FELT NEAR PARKFIELD.
03/15/1954	22-43-50	35.00	120.70	D	3.4			
03/18/1954	12-07-53	35.40	120.90	D	3.0			NORTHWEST OF SAN LUIS OBISPO.
04/01/1954	12-04-38	36.05	120.20	C	3.3			6 MI. SOUTHEAST OF COALINGA.
04/09/1954	07-38-23	35.78	121.08	D	3.1			10 MI. NORTHEAST OF SAN SIMEON.
04/09/1954	14-58--?	35.90	121.00	D			F	IV AT BRYSON (PLEYTO SCHOOL); SECOND SHOCK REPORTED FELT AT 23-40--?.
04/20/1954	09-32-18	36.63	121.03	D	2.6			12 MI. NORTHEAST OF KING CITY.
05/10/1954	14-24-28	36.08	120.80	C	3.1		F	NE OF SAN ARDO - SLIGHT AT KING CITY.
06/04/1954	11-58-38	36.45	121.13	C	3.5			16 MI. SOUTHWEST OF LLANADA.
07/05/1954	07-25-39	36.20	121.80	D	3.2			30 MI. SOUTH OF MONTEREY.
08/13/1954	13-36-44	34.25	120.50	C	3.2			
08/13/1954	13-44-23	34.25	120.50	C	3.2			
08/19/1954	11-45-08	34.25	120.50	C	3.2			
08/21/1954	22-50-49	35.47	121.33	B	3.3			40 MI. SOUTH OF HOLLISTER.
08/22/1954	08-34-40	34.33	120.67	C	3.8			
08/22/1954	12-36-07	34.33	120.67	C	3.8			
12/22/1954	21-12-24	36.00	121.00	D	3.7		F	SE OF KING CITY; III AT KING CITY.
12/22/1954	21-12-28	36.00	120.60	D	3.8			
01/07/1955	14-50-22	34.40	119.60	D	3.0			
01/18/1955	13-30--?	36.20	121.85	D			F	IV REPORTED FELT AT BIG SUR.
02/05/1955	07-10-19	35.80	121.40	C	3.3			WEST OF SAN SIMEON.
02/27/1955	03-17-51	36.25	120.83	C	2.9		F	EAST OF KING CITY; IV IN PRIEST VALLEY.
03/02/1955	03-30--?	36.00	120.70	D			F	IV REPORTED FELT IN INDIAN VALLEY.
03/02/1955	15-59-01	36.00	120.93	B	4.8		F	18 MI. SE OF KING CITY; FELT OVER 7000 SQ. MI. OF W CENTRAL CALIF. USCGS MAG. 5.1. VI AT ADELAIDA, BRYSON, INDIAN VALLEY, SAN ARDO, SAN LUCAS, AND TEMPLETON.
03/02/1955	20-02-53	36.00	120.93	B	3.7			AFTERSHOCK OF QUAKE AT 15-59-01.
03/05/1955	08-46-36	36.10	121.10	D	2.0			SOUTH OF KING CITY.
03/06/1955	10-47-32	35.92	120.90	D	3.2			SOUTHEAST OF KING CITY.
04/04/1955	20-56-56	36.08	121.00	C	3.2			SOUTHEAST OF KING CITY.
04/27/1955	09-28-08	35.90	121.20	D	2.8			SOUTHWEST OF KING CITY.
05/14/1955	20--?--?	36.35	121.85	D			F	IV REPORTED FELT AT BIG SUR AND SANTA CRUZ.
05/16/1955	18-22-52	35.92	120.58	C	3.0			SOUTHWEST OF COALINGA.
05/30/1955	09-38-29	36.25	121.25	C	3.0			WEST OF KING CITY.
05/31/1955	01-45-53	36.40	121.25	B	3.0			NORTH OF KING CITY.
06/13/1955	14-55-12	36.30	121.30	C	2.5			NORTH OF KING CITY.
06/19/1955	05-36-33	35.62	121.10	C	2.5			SOUTHEAST OF SAN SIMEON.
07/06/1955	11-29-18	36.50	121.42	C	3.4			SOUTH OF HOLLISTER.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/06/1955	13-18-53	36.50	121.50	D	2.7			SOUTH OF HOLLISTER.
07/28/1955	12-07-52	36.50	121.40	D	2.6			SOUTH OF HOLLISTER.
09/21/1955	18-06-52	36.50	121.00	D	3.3			NORTH OF KING CITY.
10/22/1955	07-04-18	36.22	120.33	C	4.2		F	V AT AND 14 MI. NW OF COALINGA.
11/02/1955	19-40-06	36.00	120.92	A	5.2		F	55 MI. NNW OF SAN LUIS OBISPO; FELT OVER 7000 SQ. MI. OF COASTAL W CENTRAL CALIF. VI AT ADELAIDA RD. (14 MI. W OF PASO ROBLES), BRYSON, KING CITY, PASO ROBLES, SAN ARDO, SAN LUCAS, AND SAN MIGUEL.
11/18/1955	09-03-30	35.90	120.50	D	2.9			SOUTHWEST OF COALINGA.
11/19/1955	07-20--?	34.50	119.65	D			F	REPORTED FELT AT SANTA BARBARA.
11/19/1955	10-59-41	36.03	120.90	C	3.3			SOUTHEAST OF KING CITY.
11/21/1955	21-14-18	36.10	119.90	D	3.5			
12/11/1955	20-10-38	36.27	120.72	C	3.5			NORTHWEST OF COALINGA.
12/16/1955	14-43-11	36.03	120.87	C	3.8		F	SOUTHWEST OF KING CITY; FELT AT ATASCADERO, PASO ROBLES, AND SAN MIGUEL.
12/29/1955	13-33-17	36.45	121.25	C	3.4			NORTH OF KING CITY.
02/14/1956	22-15-08	36.50	121.10	D	2.8			SOUTHWEST OF LLANADA.
03/15/1956	15-26-11	36.50	121.20	D	2.6			SOUTHEAST OF HOLLISTER.
04/03/1956	09-26-02	36.45	121.23	B	2.7			SOUTH OF HOLLISTER.
04/10/1956	11-24-21	36.43	121.48	C	2.9			SOUTHEAST OF MONTEREY.
04/10/1956	20-53-21	36.30	121.00	D	2.9			NORTHEAST OF KING CITY.
05/01/1956	15-06-33	36.50	121.00	D	2.5			SOUTH OF HOLLISTER.
05/04/1956	08-16-14	35.75	121.07	B	3.1			NORTHEAST OF SAN SIMEON.
05/04/1956	08-16-16	35.95	120.93	D	3.5			
05/15/1956	10-45--?	34.90	120.40	D			F	REPORTED FELT AT SANTA MARIA.
06/11/1956	-?-48-37	36.00	120.97	C	3.2			SOUTHEAST OF KING CITY.
06/15/1956	23-42-03	36.30	121.80	D	2.8			SOUTH OF MONTEREY.
07/09/1956	23-15--?	35.10	120.50	D			F	III REPORTED FELT NEAR HUASNA.
07/23/1956	08-03-48	36.30	121.30	D	4.7		F	NW OF KING CITY; FELT OVER 4000 SQ. MI. OF COASTAL CENTRAL CALIF. V AT BIG SUR, CHUALAR, GONZALES, GREENFIELD, 7.5 MI. S OF HOLLISTER, KING CITY, PASO ROBLES, SAN BENITO, AND SAN JUAN BAUTISTA.
07/23/1956	08-20-37	36.50	121.40	D	3.1			AFTERSHOCK OF QUAKE AT 08-03-48.
07/31/1956	-?-40-43	34.15	119.60	C	3.2			
07/31/1956	17-25--?	35.10	120.50	D			F	IV REPORTED FELT AT HUASNA.
08/09/1956	-?-08-49	34.37	119.80	B	4.0		F	OFF SANTA BARBARA; IV AT LOS PRIETOS RANGER STATION.
08/10/1956	23-24-03	35.90	121.30	D	3.0			SOUTHWEST OF KING CITY.
08/20/1956	05-10-33	36.48	121.48	B	3.2		F	NEAR GONZALES; IV AT PINNACLES NATIONAL MONUMENT.
09/15/1956	-?-34-37	36.30	120.30	D	2.7			NORTH OF COALINGA.
10/10/1956	20-02-24	34.70	121.00	D	3.8			
11/12/1956	10-13--?	36.30	120.10	C	3.3			
11/16/1956	03-23-09	35.95	120.47	B	5.0		F	SW OF COALINGA; FELT OVER 8000 SQ. MI. FROM HOLY CITY TO BETTERAVIA TO FIREBAUGH. VI AT KING CITY, MEE RANCH (LONOAK), AND SAN LUCAS.
11/19/1956	13-53-53	35.98	120.57	C	3.3		F	SOUTHWEST OF COALINGA; III AT ADELAIDA (15 MI. WEST OF PASO ROBLES).
11/20/1956	03-42-44	34.70	120.50	C	3.6		F	IV AT LOS ALAMOS; III FELT AT 07-42--?, 11/21/1956.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/11/1956	10-56-53	35.88	120.47	C	4.1			NEAR PARKFIELD.
12/28/1956	13-39-37	35.90	121.10	D	2.6			NORTHEAST OF SAN SIMEON.
01/01/1957	09-25--?	35.50	120.65	D			F	REPORTED FELT AT ATASCADERO.
01/29/1957	21-19-53	35.87	122.12	C	4.9		F	OFF COAST NW OF SAN SIMEON; FELT OVER 5000 SQ. MI. OF COASTAL CENTRAL CALIF. V AT BIG SUR, CAMBRIA, CARMEL VALLEY, HARMONY, KING CITY, LUCIA, MARINA, AND SEASIDE, AND IV GENERALLY FROM MOSS LANDING TO 20 MI. W OF COALINGA TO SAN LUIS OBISPO.
02/03/1957	07-57-12	34.50	121.20	C	3.9			
02/08/1957	04-45-38	36.50	121.20	D	2.8			NORTH OF KING CITY.
02/08/1957	21-20--?	36.50	122.00	D			F	SHARP SHOCK FELT MONTEREY PEN. (BSSA).
02/09/1957	08-10--?	35.50	120.65	D			F	IV REPORTED FELT AT ATASCADERO.
02/14/1957	-?-31-30	35.10	119.80	D	2.4			
02/14/1957	10-30-27	36.00	120.60	C	3.6			
02/16/1957	11-43-50	34.30	119.53	C	3.5			
03/09/1957	14-38-28	34.70	119.60	C	2.9			
03/09/1957	14-59-21	34.70	119.60	C	2.4			
04/05/1957	-?-40--?	34.75	120.25	D			F	IV REPORTED FELT AT LOS ALAMOS.
06/21/1957	20-46-42	35.10	120.90	D	3.7		F	OFF COAST; FELT AT SAN LUIS OBISPO AND MORRO BAY.
07/02/1957	09-18-22	34.37	119.88	B	3.4		F	W OF SANTA BARBARA; FELT AT SANTA BARBARA.
07/02/1957	12-59-05	34.37	119.88	B	3.3			
07/02/1957	13-58-28	34.37	119.88	B	3.2			
07/21/1957	01-29-20	36.43	121.22	B	3.1			NORTH OF KING CITY.
08/03/1957	09-31-22	36.25	120.88	C	2.5			EAST OF KING CITY.
08/18/1957	03-05-25	34.47	120.13	C	3.4			
08/18/1957	11-08-23	34.47	120.13	C			F	N OF GAVIOTA; FELT AT CACHUMA RESERVOIR.
08/21/1957	07-36-54	36.47	121.52	C	3.6			NORTHWEST OF KING CITY.
08/28/1957	01-13-57	34.58	121.00	C	3.5			
09/12/1957	21-36--?	35.50	121.00	D			F	II FELT AT P G AND E PLANT, MORRO BAY.
09/21/1957	06-54-26	36.40	121.10	D	2.8			NORTH OF KING CITY.
09/21/1957	15-32--?	35.50	121.00	D			F	II FELT AT P G AND E PLANT, MORRO BAY.
09/25/1957	23-33-31	36.50	121.50	D	2.7			SOUTH OF HOLLISTER.
10/01/1957	12-55-57	36.47	121.23	C	3.3			SOUTHWEST OF LLANADA.
10/05/1957	14-42--?	34.75	120.25	D			F	IV REPORTED FELT AT LOS ALAMOS.
10/19/1957	-?-04-38	36.10	120.87	B	3.3			SOUTHEAST OF KING CITY.
10/28/1957	11-41-02	34.33	120.00	C	2.8			
11/05/1957	23-50-52	34.72	120.33	C	3.4			
11/18/1957	01-11-42	36.38	121.23	C	3.1			NORTHWEST OF KING CITY.
11/18/1957	07-26-32	36.50	121.70	D	3.3			SOUTHEAST OF MONTEREY.
12/31/1957	22-32-55	36.40	121.00	D	2.9			NORTHEAST OF KING CITY.
01/07/1958	17-13-16	35.70	120.80	D	3.0			NORTH OF SAN LUIS OBISPO.
01/18/1958	08-12--?	35.55	120.65	D			F	REPORTED FELT AT PASO ROBLES.
01/21/1958	21-22-08	36.40	120.50	D	2.9			NORTHWEST OF COALINGA.
01/23/1958	07-06-46	34.38	119.58	B	2.6		F	E OF SANTA BARBARA; IV AT SANTA BARBARA.
01/28/1958	07-12-54	36.50	121.10	D	2.2			SOUTHWEST OF LLANADA.
03/26/1958	13-12-30	36.20	120.30	D	2.4			NEAR COALINGA.
03/27/1958	20-26-14	35.90	121.50	D	2.8			NORTHWEST OF SAN SIMEON.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/31/1958	17-38-23	36.50	121.10	D	2.7			SOUTHWEST OF LLANADA.
04/10/1958	08-32-33	36.45	121.12	C	2.9			SOUTHWEST OF LLANADA.
06/05/1958	17-12-50	36.40	121.10	D	3.1			NORTH OF KING CITY.
06/15/1958	07-02-33	36.50	121.38	C	2.9			FORESHOCK OF QUAKE AT 07-05-34.
06/18/1958	07-05-34	36.50	121.38	C	3.3			SOUTH OF HOLLISTER.
06/21/1958	01-03-31	36.40	120.40	D	2.1			SOUTHWEST OF FRESNO.
07/02/1958	17-56-26	36.50	121.30	D	2.8			SOUTHWEST OF LLANADA.
08/08/1958	18-43-01	36.30	121.20	D	2.7			FORESHOCK OF 13-43-15 - RECORDS MIXED.
08/08/1958	13-43-15	36.30	121.20	D	3.9		F	NORTHWEST OF KING CITY; IV AT BIG SUR.
08/18/1958	05-30-42	35.80	121.30	D	3.4			NEAR SAN SIMEON.
09/01/1958	11-31-42	36.10	120.80	D	3.2			SOUTHEAST OF KING CITY.
09/21/1958	07-24-55	36.35	121.12	C	4.0		F	NORTH OF KING CITY; VI AT SAN BENITO; ALSO FELT AT SOLEDAD.
09/21/1958	14-23-01	36.50	121.05	C	2.7			SOUTHWEST OF LLANADA.
10/03/1958	04-25-51	34.37	119.50	B	3.7		F	FROM CARPINTERIA TO GOLETA.
10/10/1958	13-05-16	35.93	120.50	B	4.5		F	SOUTHWEST OF COALINGA; FELT OVER AN AREA OF APPROXIMATELY 3500 SQ. MI. OF THE SOUTHWEST-CENTRAL REGION OF CALIFORNIA - APPEARS TO HAVE BEEN FELT MORE STRONGLY AT PARKFIELD THAN ELSEWHERE; V AT ADELAIDA, CAMP ROBERTS, COALINGA, HARMONY, LONE PINE INN, OILFIELD, PARKFIELD, PASO ROBLES, AND SAN ARDO.
10/15/1958	16-16-44	35.50	121.20	D	3.2			NEAR SAN SIMEON.
11/06/1958	20-11-57	36.08	120.88	C	3.1			SOUTHEAST OF KING CITY.
11/16/1958	09-34-04	34.50	119.83	C	4.0		F	NW OF SANTA BARBARA; FELT OVER 600 SQ. MI. FROM SANTA YNEZ TO VENTURA; V AT CARPINTERIA, GOLETA, AND SANTA BARBARA.
11/27/1958	06-04-26	36.37	121.15	C	3.9		F	WEST OF LLANADA; FELT SLIGHTLY AT CARMEL.
11/27/1958	13-39-01	36.20	120.80	D	3.1			EAST OF KING CITY.
12/15/1958	14-58-49	36.20	120.40	D	3.0			NEAR COALINGA.
12/15/1958	15-24-01	36.20	120.40	D	3.0		F	NEAR COALINGA; IV AT COALINGA.
12/30/1958	01-34-15	35.92	119.80	C	3.2			
01/11/1959	05-18-26	36.20	120.80	D	2.5			WEST OF COALINGA.
02/07/1959	05-51-02	36.10	120.00	D	3.0			SOUTHEAST OF KING CITY.
02/27/1959	21-35-01	36.25	120.75	C	3.1			SOUTHEAST OF LLANADA.
03/13/1959	02-44-27	35.80	120.30	D	2.5			SOUTH OF COALINGA.
03/14/1959	02-43-41	35.70	121.30	D	3.6			WEST OF SAN SIMEON.
03/20/1959	05-12-09	36.48	121.17	B	2.9			SOUTHWEST OF LLANADA.
03/25/1959	05-34-17	34.25	119.58	C	2.5		F	SANTA BARBARA CHANNEL; IV AT CARPINTERIA.
04/08/1959	07-41-57	36.37	121.20	B	3.4			NORTH OF KING CITY.
04/09/1959	14-03-11	36.38	121.15	C	2.5			NORTH OF KING CITY.
04/21/1959	09-36-23	36.40	120.40	D	3.0			NORTH OF COALINGA.
04/21/1959	12-31-10	36.10	121.10	D	2.2			NEAR KING CITY.
04/22/1959	19-04-25	36.20	120.90	D	2.6			NEAR KING CITY.
05/13/1959	14-28-10	36.48	121.03	C	2.6			SOUTHWEST OF LLANADA.
05/14/1959	01-34-09	36.50	121.20	D	2.4			SOUTHWEST OF LLANADA.
05/20/1959	10-15-55	36.30	120.40	D	2.6			NORTHWEST OF COALINGA.
06/01/1959	03-47-24	36.50	121.23	C	2.4			SOUTHWEST OF LLANADA.
06/20/1959	15-01-17	36.50	121.30	D	2.9			SOUTH OF VINEYARD.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/21/1959	09-24-07	34.32	119.67	B	3.3			
07/18/1959	01-11-47	36.50	121.30	D	2.5			SOUTH OF VINEYARD.
08/05/1959	03--?-34	35.95	120.48	C	3.5		F	SOUTHEAST OF COALINGA (NEAR PARKFIELD; FELT STRONGEST AT PARKFIELD; IV FELT AT PASO ROBLES).
09/05/1959	05-45-34	36.50	121.70	D	3.8			SOUTHWEST OF VINEYARD.
10/01/1959	04-35-35	34.43	120.57	B	4.5		F	OFF POINT CONCEPTION; VI AT GAVIOTA PASS AND V AT GAVIOTA, GOLETA, AND LOMPOC.
10/01/1959	05-52-55	34.20	119.50	C	3.2			
10/11/1959	02-03-09	36.45	121.12	C	4.1		F	SOUTHWEST OF LLANADA; FELT AT SALINAS.
10/24/1959	23-12-54	36.47	121.40	C	3.2			SOUTH OF HOLLISTER.
10/25/1959	03-33-13	36.50	121.20	D	2.4			SOUTHEAST OF VINEYARD.
10/25/1959	03-34-02	36.50	121.32	C	3.0			SOUTH OF VINEYARD.
10/26/1959	09-56-01	36.40	121.10	D	3.0			SOUTHEAST OF VINEYARD.
11/25/1959	09-28-22	35.20	121.20	D	3.5			SOUTH OF KING CITY.
11/26/1959	07-02-05	36.40	121.40	D	2.7			SOUTH OF VINEYARD.
12/11/1959	05-55-26	35.60	120.60	D	3.5			SOUTHEAST OF VINEYARD.
12/25/1959	20-38-28	36.00	120.60	D	3.1			SOUTHEAST OF VINEYARD.
12/29/1959	14-53-08	35.75	120.30	C	3.5		F	NEAR CHOLAME; FELT AT PASO ROBLES.
01/02/1960	22-51-48	35.40	121.20	D	4.0			NW OF SAN LUIS OBISPO.
01/04/1960	12-18-20	36.20	120.70	D	3.2			WEST OF COALINGA.
02/14/1960	08-34-30	35.80	121.70	D	2.8			WEST OF SAN SIMEON.
02/25/1960	06-34-31	36.50	121.20	D	2.7			SOUTHWEST OF LLANADA.
02/28/1960	02-55-32	34.33	119.95	C	3.1			
03/21/1960	20-46-39	36.50	120.73	C	2.5			SOUTHEAST OF LLANADA.
03/26/1960	21-39-21	36.22	121.00	C	2.7			EAST OF KING CITY.
03/29/1960	11-46-42	36.50	121.10	C	2.4			SOUTHEAST OF VINEYARD.
03/31/1960	08-35-09	36.40	121.20	D	2.6			SOUTHEAST OF VINEYARD.
04/02/1960	13-02-10	35.97	120.33	C	2.7			SOUTH OF COALINGA.
04/02/1960	19-01-12	36.20	120.60	D	3.4			WEST OF COALINGA.
04/09/1960	08-01-14	36.50	121.13	B	3.6			SOUTHEAST OF HOLLISTER.
05/04/1960	09-44-32	36.42	120.72	C	3.4			SOUTHEAST OF LLANADA.
05/15/1960	06-07-23	36.43	121.27	C	2.5			SOUTH OF VINEYARD.
06/11/1960	17-39-48	36.30	120.90	D	3.7			SOUTHEAST OF VINEYARD, DIABLO RANGE.
06/19/1960	19-51-20	36.20	121.90	D	2.6			SOUTHWEST OF BIG SUR.
06/24/1960	18-13-12	36.45	121.22	B	3.5			SOUTHEAST OF VINEYARD.
07/14/1960	03-22-23	35.60	120.40	D	3.0			NORTHEAST OF SAN LUIS OBISPO.
07/20/1960	-?-59-36	35.80	119.80	D	2.8			NORTHEAST OF SAN LUIS OBISPO.
07/30/1960	02-16-29	36.43	120.28	C	2.5			SOUTHWEST OF FRESNO.
08/09/1960	08-59-47	36.20	120.20	D	3.2			EAST OF COALINGA.
08/10/1960	03-03-50	36.47	121.40	D	3.2			SOUTH OF VINEYARD.
08/26/1960	08-57-24	38.33	121.13	C	3.0			SOUTHEAST OF VINEYARD.
09/10/1960	01-18-22	36.47	121.05	C	2.7			SOUTHEAST OF HOLLISTER.
09/10/1960	20-49-12	36.45	121.28	D	2.8			SOUTHWEST OF LLANADA.
10/08/1960	-?-02-29	36.50	121.67	C	3.0			SOUTHWEST OF VINEYARD.
11/03/1960	07-13-40	36.43	121.07	C	2.7			SOUTH-SOUTHWEST OF LLANADA.
11/18/1960	04-36-44	36.38	121.20	C	3.0			NORTH-NORTHWEST OF KING CITY.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/01/1960	14-23-49	34.33	119.85	B	3.2			OFF SANTA BARBARA.
12/15/1960	08-28-08	36.40	121.30	D	3.0			SOUTH OF HOLLISTER.
12/27/1960	03-57-55	36.00	121.10	D	3.3			SOUTH OF KING CITY.
01/06/1961	20-46-36	35.80	120.20	D	3.4			SE OF PARKFIELD.
02/02/1961	12-31--?	36.35	121.20	D	2.2			NORTHWEST OF KING CITY.
02/21/1961	15-46-58	34.37	119.53	C	2.8			SE OF SANTA BARBARA.
03/14/1961	04-15--?	36.40	121.20	D	3.4			SOUTHEAST OF VINEYARD.
03/29/1961	16--?-11	36.50	121.50	D	2.5			SOUTH OF VINEYARD.
04/07/1961	12-21-19	36.20	120.40	D	2.9			NORTH OF COALINGA.
04/08/1961	04-55-26	36.00	121.20	D	2.7			SOUTH OF KING CITY.
04/08/1961	09-29-47	36.10	120.43	B	3.4			NEAR COALINGA.
04/08/1961	12-52-16	36.12	120.43	C	2.7			AFTERSHOCK OF QUAKE AT 09-29-47.
04/11/1961	09-08-11	36.00	120.10	D	2.7			SOUTHEAST OF KING CITY.
04/12/1961	04-59-08	35.92	120.50	C	2.6			SOUTHWEST OF COALINGA.
04/19/1961	18-16-35	36.40	121.58	C	3.3			SOUTHEAST OF MONTEREY.
05/25/1961	14-19-05	36.33	121.00	C	3.4			NORTHEAST OF KING CITY.
05/25/1961	14-19-35	36.33	121.00	B	3.4			NORTHEAST OF KING CITY.
06/01/1961	06-47-20	36.33	121.32	B	2.7			NORTHWEST OF KING CITY.
06/01/1961	14-11-30	36.45	121.20	C	2.6			NORTH OF KING CITY.
06/18/1961	12-50-59	36.18	120.83	C	2.1			EAST OF KING CITY.
06/25/1961	13-15-26	36.48	121.35	C	3.6		F	SOUTH OF HOLLISTER; FELT IN HOLLISTER AREA. INTENSITY IV 7.5 MI. SOUTH OF HOLLISTER AT HARRIS RANCH.
06/26/1961	11-30-22	35.77	122.00	C	2.5			OFF SAN SIMEON COAST.
07/22/1961	18-01-55	36.40	121.20	C	4.0		F	NORTHEAST OF PARAISO; FELT AT PINNACLES NATIONAL MONUMENT (ABOUT 25 MI. SOUTHEAST OF HOLLISTER).
07/31/1961	-?-07-09	35.82	120.37	C	4.7		F	SAN LUIS OBISPO; FELT OVER AN AREA OF 5000 SQ. MI. OF WEST CENTRAL CALIFORNIA. INTENSITY V AT ATASCADERO, CHOLAME, CRESTON, PARKFIELD, SAN LUIS OBISPO, AND TEMPLETON.
08/01/1961	06-12-54	36.43	120.85	C	3.1			SOUTH OF LLANADA.
08/17/1961	17-14-45	36.33	120.95	B	3.1			NORTHEAST OF KING CITY.
09/14/1961	15-12-20	34.32	119.63	C	2.7			
09/14/1961	15-14-38	34.32	119.63	C	2.8			
09/27/1961	02-02-06	36.33	121.25	C	2.7			EAST OF PARAISO.
09/29/1961	15-39-58	36.33	120.88	B	2.4			SOUTH OF LLANADA.
10/12/1961	06-31-11	35.80	121.30	D	2.3			NORTH OF SAN SIMEON.
10/29/1961	11-47-33	36.33	120.92	C	2.0			EAST OF PARAISO.
11/05/1961	10-43-57	36.03	120.10	D	2.0			EAST OF LLANADA.
11/29/1961	04-49-03	35.15	120.13	C	3.0			SOUTHEAST OF KING CITY.
12/06/1961	03-27-30	36.43	121.85	B	2.4			SOUTH OF MONTEREY.
12/14/1961	07-28-44	36.48	121.08	B	2.1			SOUTHWEST OF LLANADA.
01/04/1962	03-56-10	36.40	121.40	C	3.0	11		NORTHWEST OF KING CITY.
01/31/1962	08-33-15	34.88	120.68	C	3.6			
02/01/1962	06-37-57	34.88	120.68	C	4.5		F	WEST OF GUADALUPE; FELT OVER AN AREA OF 3000 SQ. MI. V AT ARROYO GRANDE, AVILA BEACH, CASMALIA, GROVER CITY, GUADALUPE, HALCYON, OCEANO, POINT ARGUELLO, AND SHELL
02/01/1962	07-58-12	34.38	120.68	C	3.7			
02/04/1962	11-43-34.1	36.42	121.27	C	3.2	12		

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
02/07/1962	13--?-70	34.30	122.10	D	3.9			
03/05/1962	07-44-01	34.60	121.60		4.5		F	OFF COAST NEAR LOMPOC; V AT MORRO BAY AND PISMO BEACH.
03/06/1962	03-40-22	34.60	121.60	D	3.6			
03/10/1962	08-07-21	34.60	121.60	D	4.2			OFF COAST NEAR LOMPOC.
03/10/1962	13-40-48	34.60	121.60	D	4.0			OFF COAST NEAR LOMPOC.
03/10/1962	15-24-21	34.60	121.60	D	3.5			
03/12/1962	21-32-09	34.60	121.60	D	3.9			
03/23/1962	22-10-18	34.28	120.20	C	2.9			
03/24/1962	03-38-41.8	36.20	119.78	B	3.4	19		SOUTH OF FRESNO.
04/02/1962	03-06-03.2	36.25	120.10	B	3.7	16	F	EAST OF COALINGA; V IN TEHACHAPI.
04/15/1962	08-41-02.3	36.42	120.62	B	4.7	23	F	SOUTHEAST OF LLANADA; V AT IDRIA.
05/04/1962	20-52-32	35.27	119.55	B	2.8			
05/05/1962	-?-55-20	34.20	121.50	D	3.3			
09/03/1962	17-53-33.1	36.47	121.07	C	2.6	8	F	SOUTHWEST OF LLANADA; FELT IN HOLLISTER.
09/11/1962	01-34-31	36.03	121.23	B	3.3	16		SOUTHWEST OF KING CITY.
09/16/1962	18-12-35	34.48	119.68	B	4.0		F	NEAR SANTA BARBARA; V AT LOS PRIETOS.
09/16/1962	18-17-09	34.48	119.68	C	2.2			
09/16/1962	18-31-17	34.52	119.77	B	2.9			
09/21/1962	05-07-18	34.47	119.58	B	3.0			
09/29/1962	19-47-32	34.47	119.70	B	2.9			
10/13/1962	17-49-39.5	36.35	120.42	B	3.7	17		NORTHEAST OF PRIEST.
12/15/1962	-?-40-20.9	36.47	120.63	B	2.9	13		NORTH OF PRIEST.
01/09/1963	06-04-25.7	35.98	120.35	B	3.2	14	F	SE OF PRIEST; III AT WHEELER RIDGE.
02/09/1963	02-52-14.5	35.98	121.69	C	2.8	8		OFF COAST S OF BIG SUR.
02/12/1963	03-44-30.9	36.50	121.32	B	2.6	10		S OF VINEYARD.
02/22/1963	15-56-21.9	35.11	121.44	C	3.3	15		OFF COAST, SW OF MORRO BAY.
02/22/1963	15-56-36.0	35.67	120.83	D	3.6			
04/04/1963	01--?-58	35.80	121.50		2.5	6		NW OF SAN SIMEON.
04/10/1963	01-38-56.8	36.42	121.05		2.9	11		SW OF LLANADA.
04/11/1963	14-02-31.8	36.20	120.87		2.9	13		NW OF PRIEST.
04/20/1963	16-37-33.0	36.38	120.96		3.0	14		SOUTH OF LLANADA.
05/10/1963	10-17-57.1	36.37	120.98		2.5	9		SOUTH OF LLANADA.
06/01/1963	05-19--0.2	34.33	119.54	B	2.0			
07/02/1963	12--?-24.9	34.86	119.80	C	2.0			
07/04/1963	03-20-41.0	34.77	120.02	C	3.2			
07/06/1963	23-32-30.4	34.78	120.63	B	3.3			
08/15/1963	21-02-32.2	35.97	121.02		3.6	15		NEAR JOLON; FORESHOCK OF FOLLOWING--
08/15/1963	21-21-32.1	35.91	121.06		3.9	18	F	NEAR JOLON; FELT AT HARRIS RANCH.
08/16/1963	08-12-13.6	36.06	121.01		3.2	10		NEAR JOLON; AFTERSHOCK OF PRECEDING.
09/06/1963	03-54-34	36.22	121.48		2.6	8		WEST OF PARAISO.
11/01/1963	14-05-56.0	35.56	120.23		3.4	9		EAST OF ATASCADERO.
11/01/1963	14-06--0.4	35.75	120.47	C	3.2			
11/18/1963	07-31-38.5	36.22	120.30	C	3.5		F	IV 15 MI. NE OF SAN MIGUEL.
11/18/1963	10-54-45.4	36.38	120.32		2.7	11		NE OF COALINGA.
11/19/1963	03-33-09.2	36.42	121.03		2.9	10		SW OF LLANADA.
12/12/1963	17-10-48.5	34.98	119.51	C	3.1			
02/10/1964	05-47-25.0	35.75	120.94		3.9	19		NE OF PASO ROBLES.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/20/1964	13-15-51.0	36.40	121.03		2.6	7		SW OF LLANADA.
04/28/1964	15-01-48.3	36.23	121.08		2.8	7		NEAR KING CITY.
05/07/1964	17-53-58.3	36.43	120.54		2.5	5		N OF PRIEST.
06/06/1964	11-47-39.0	34.63	121.40	D	4.3			
06/20/1964	09-21-51.4	34.13	120.67	C	3.1			
07/24/1964	07-09-35.9	36.47	121.18		2.9	15		NE OF PARAISO.
08/30/1964	03-41-10.4	36.29	121.94		2.9	7		OFF COAST NW OF POINT SUR.
09/12/1964	01-45-53.5	36.08	120.49		3.1	10		SE OF PRIEST.
10/17/1964	23-43-22.6	36.21	120.92		3.3	14		NW OF PRIEST.
11/08/1964	01-19-19.0	36.00	120.00		4.0	15		E OF AVENAL.
11/08/1964	13-45-51.1	36.34	121.32		3.1	16		NEAR PARAISO.
11/18/1964	01-47-34.0	35.98	121.13		2.7	8		SW OF KING CITY.
11/25/1964	12-49-41.8	36.21	120.78		2.8	15		NW OF PRIEST.
12/05/1964	13-55-57.5	36.02	121.08		2.6	12		W OF SAN ARDO.
12/11/1964	03-35-38.8	34.24	119.76	B	3.5			
12/25/1964	11-21-13.2	35.97	121.18		2.6	5		N OF LAKE NACIMIENTO.
12/27/1964	18-58-59.4	36.46	121.06		2.6	7		SW OF LLANDA.
01/13/1965	04-20-48.2	36.45	120.58		2.6	9		NE OF PRIEST.
01/26/1965	08-34-30.7	35.72	120.54		3.0	12		SE OF PRIEST.
01/26/1965	08-36-36.6	35.92	120.27	C	3.1			
01/26/1965	08-38-16.4	36.04	120.26	C	3.1			
02/21/1965	18-39-18.3	35.67	120.43		3.1	12		E OF PASO ROBLES.
03/28/1965	02-32-21.0	36.20	120.40		3.5			(USCGS)
04/06/1965	20-49-24.4	35.95	121.46		2.5	7		N OF SAN SIMEON.
04/08/1965	01-05-40.6	36.03	121.40		3.0	10		N OF SAN SIMEON.
04/09/1965	12-50-19.3	36.03	120.64		3.0	10		S OF PRIEST.
04/18/1965	03-58-52.4	36.50	121.23		2.7	7		NEAR PINNACLES NATIONAL MONUMENT
04/24/1965	07-29-47.1	34.91	120.14	C	3.6			
05/12/1965	17-55-08.7	35.49	121.17		3.0	6		SW OF SAN SIMEON.
06/07/1965	15-06-47.6	36.50	121.13		2.5	10		NEAR PINNACLES NATIONAL MONUMENT.
06/20/1965	02-56-43.5	36.33	120.37		2.7	11		N OF COALINGA.
06/30/1965	15-21-27.7	36.35	120.71		2.5	9		N OF PRIEST.
07/23/1965	05-31-52.7	35.71	121.23		3.4	13		N OF SAN SIMEON.
07/24/1965	15-25-57.4	36.36	120.98		2.5	7		SW OF LLANADA.
08/01/1965	06-47-27.3	36.23	120.85		2.5	7		NW OF PRIEST.
08/01/1965	13-28-32.9	36.23	120.84		2.5	6		AFTERSHOCK OF 06-47-27.3.
08/13/1965	07-36-08.4	36.46	121.08		2.6	9		SW OF LLANADA.
08/13/1965	13-46-16.5	34.35	119.63	B	3.7		F	IV AT CARPINTERIA AND SANTA BARBARA.
08/13/1965	21-28-51.8	36.48	121.13		2.4	8		W OF LLANADA.
08/15/1965	23-06-52.5	36.00	120.20		4.0		F	AT PAICINES.
08/21/1965	20-09-35.4	36.46	121.07		2.5	8		SW OF LLANADA.
09/06/1965	18--?-57.8	35.96	120.36	C	3.4			
09/12/1965	08-50-05.5	36.49	121.12		2.5	7		W OF LLANADA.
09/19/1965	15-42-07.8	35.98	120.34	C	4.8		F	V AT ARMONA, AVENAL, CHOLAME, KETTLEMAN CITY, AND STRATFORD.
10/22/1965	02-29-22	36.00	121.70		2.7	6		OFF COAST, W OF KING CITY.
12/02/1965	22-29-13.0	36.20	121.68		2.8	9		W OF PARAISO.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
01/28/1966	01-49-47.4	35.83	120.45		3.0		F	PARKFIELD SEQUENCE; MC EVILLY, ET AL, (1967) THE PARKFIELD, CALIFORNIA EARTHQUAKE OF 1966, BULL. SEISM. SOC. AM.
02/01/1966	-?-20-44.3	36.03	120.57		2.9			PARKFIELD SEQUENCE - SEE 01/28/1966 AT 01-49-47.4.
02/14/1966	-?-24-03.9	36.02	120.57		2.4			PARKFIELD SEQUENCE - SEE 01/28/1966 AT 01-49-47.4.
02/25/1966	01-34-38.0	36.05	120.63		2.4			PARKFIELD SEQUENCE - SEE 01/28/1966 AT 01-49-47.4.
03/31/1966	21-38-45.2	36.05	120.60		2.5			PARKFIELD SEQUENCE - SEE 01/28/1966 AT 01-49-47.4.
04/05/1966	20-44-58.7	36.24	120.85		2.7	9		10 KM NW OF PRIEST (UC BERKELEY SEISMOGRAPH STATION (SS)).
04/12/1966	15-31-39.8	36.07	120.70		2.3			PARKFIELD SEQUENCE.
05/11/1966	17-37-01.1	35.98	120.57		2.3			PARKFIELD SEQUENCE.
05/23/1966	08-07-37.6	36.02	120.57		2.5			PARKFIELD SEQUENCE.
05/23/1966	08-11-07.0	36.02	120.57		2.2			PARKFIELD SEQUENCE.
05/27/1966	15-36-03.7	35.98	120.49		2.7			PARKFIELD SEQUENCE.
06/18/1966	16-32-17.6	35.96	120.53		2.0			PARKFIELD SEQUENCE.
06/20/1966	23-19-18.8	36.33	120.96		2.8	9		NE OF KING CITY.
06/24/1966	21-42-50.4	36.50	120.85		3.1	10		SE OF LLANADA.
06/28/1966	01--?-31.5	35.95	120.52		3.1		F	PARKFIELD SEQUENCE; FELT AT CHOLAME, PARKFIELD, VALLETON, AND WORK RANCH.
06/28/1966	01-14-55	35.95	120.50		1.8			PARKFIELD SEQUENCE.
06/28/1966	04-08-55.2	35.97	120.50		5.1			PARKFIELD SEQUENCE FIRST MAIN SHOCK (FELT REPORTS FOR THE 2 MAIN SHOCKS ARE NOT SEPARATED.) FELT OVER 20,000 SQ. MI., MINOR SURFACE FAULTING ALONG SAN ANDREAS FAULT FROM PARKFIELD TO CHOLAME (20 MI.), MAXIMUM DISPLACEMENT 4 IN. VII AT CHOLAME AND PARKFIELD, VI AT ANNETTE, BITTERWATER VALLEY, COALINGA, HIDDEN VALLEY RANCH, PASO ROBLES, SAN LUIS OBISPO, SAN MIGUEL, SHAFTER, SHANDON, SLACK CANYON, VALLETON, WAITI RANCH, AND WORK RANCH, AND V AT ADELAIDA, ALPAUGH, ARROYO GRANDE, ATASCADERO, AVILA BEACH, BAKERSFIELD, BAYWOOD PARK, BRYSON, BURREL, BUTTONWILLOW, EARLIMART, FELLOWS, FRAZIER PARK, GREENFIELD, HARMONY, INDIAN VALLEY, KETTLEMAN CITY, KING CITY, LAPANZA, LOST MARICOPA, MEE RANCH, MORRO BAY, MOSS LANDING, MUSICK, NIPOMO, OCEANO, OLD RIVER, PANOCHE, PINE CANYON, PISMO BEACH, POZO, PRIEST VALLEY, SAN ARDO, SAN JOAQUIN, SAN LUCAS, SAN SIMEON, SIMMLER, STRATFORD, TEMPLETON, AND VANDENBURG A.F.B.
06/28/1966	04-09-53	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-18-34.0	35.95	120.53		2.6		F	PARKFIELD SEQUENCE - FELT AT CANTUA CREEK AND SOQUEL.
06/28/1966	04-26-13.4	35.95	120.50		5.5		F	PARKFIELD SEQUENCE - SECOND MAIN SHOCK.
06/28/1966	04-26-28	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-26-34	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-27-37	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-27-58	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-28-19	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-28-36	35.95	120.50		4.5			PARKFIELD SEQUENCE.
06/28/1966	04-28-46	35.95	120.50					PARKFIELD SEQUENCE.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/28/1966	04-29-13	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-31-55	35.95	120.50		3.0			PARKFIELD SEQUENCE.
06/28/1966	04-32-50	35.95	120.50		3.5		F	PARKFIELD SEQUENCE - FELT AT CANTUA CREEK, CHOLAME, AND HERNANDEZ.
06/28/1966	04-34-59.1	35.81	120.40		3.0			PARKFIELD SEQUENCE.
06/28/1966	04-39-08.1	35.95	120.50		3.0		F	PARKFIELD SEQUENCE - FELT AT PARKFIELD AND WORK RANCH.
06/28/1966	04-42-33.6	35.83	120.38		2.4			PARKFIELD SEQUENCE.
06/28/1966	04-43-54.8	35.95	120.57		2.7			PARKFIELD SEQUENCE.
06/28/1966	04-46-22	35.95	120.50		3.0			PARKFIELD SEQUENCE.
06/28/1966	04-51-43	35.95	120.50		2.4			PARKFIELD SEQUENCE.
06/28/1966	05--?-59.5	35.85	120.40		3.1			PARKFIELD SEQUENCE.
06/28/1966	05-03-44.7	35.88	120.45		2.4			PARKFIELD SEQUENCE.
06/28/1966	05-09-48.3	35.83	120.13		2.5			PARKFIELD SEQUENCE.
06/28/1966	05-12-42.5	35.92	120.47		2.9			PARKFIELD SEQUENCE.
06/28/1966	05-17-05	35.95	120.50		2.1			PARKFIELD SEQUENCE.
06/28/1966	05-21-05	35.95	120.50		2.0			PARKFIELD SEQUENCE.
06/28/1966	05-29-14.9	35.92	120.48		2.1			PARKFIELD SEQUENCE.
06/28/1966	05-37-04.6	35.88	120.44		2.5			PARKFIELD SEQUENCE.
06/28/1966	05-40-19.4	35.94	120.48		2.7			PARKFIELD SEQUENCE.
06/28/1966	05-45-59.1	35.75	120.33		3.2			PARKFIELD SEQUENCE.
06/28/1966	05-48-26	35.95	120.50		2.2			PARKFIELD SEQUENCE.
06/28/1966	05-51-34.0	35.86	120.44		2.1			PARKFIELD SEQUENCE.
06/28/1966	05-52-06	35.95	120.50		2.3			PARKFIELD SEQUENCE.
06/28/1966	05-52-58	35.95	120.50		2.4			PARKFIELD SEQUENCE.
06/28/1966	05-56--?	35.95	120.50		2.1			PARKFIELD SEQUENCE.
06/28/1966	06-11-03.5	35.81	120.35		2.6			PARKFIELD SEQUENCE.
06/28/1966	06-32-17.9	35.94	120.52		3.4		F	PARKFIELD SEQUENCE - FELT AT CHOLAME, COALINGA, AND PARKFIELD.
06/28/1966	06-35-11.4	35.80	120.38		3.0			PARKFIELD SEQUENCE.
06/28/1966	06-39-31.2	35.90	120.47		2.2			PARKFIELD SEQUENCE.
06/28/1966	07-01-03.8	35.92	120.48		2.2			PARKFIELD SEQUENCE.
06/28/1966	07-33-52.7	35.90	120.45		2.7			PARKFIELD SEQUENCE.
06/28/1966	07-41-43	35.95	120.50		2.3			PARKFIELD SEQUENCE.
06/28/1966	07-45-48.3	35.90	120.47		3.0		F	PARKFIELD SEQUENCE - FELT AT CHOLAME AND PARKFIELD.
06/28/1966	08-14-48.6	35.83	120.42		2.4			PARKFIELD SEQUENCE.
06/28/1966	08-47-52.4	35.85	120.42		2.0			PARKFIELD SEQUENCE.
06/28/1966	08-54-49.5	35.92	120.50		2.3			PARKFIELD SEQUENCE.
06/28/1966	08-59-52.3	35.85	120.42		2.5			PARKFIELD SEQUENCE.
06/28/1966	09-31-26.5	35.77	120.35		2.4			PARKFIELD SEQUENCE.
06/28/1966	09-35-54.3	35.77	120.36		2.2			PARKFIELD SEQUENCE.
06/28/1966	09-56-09.7	35.83	120.40		2.5			PARKFIELD SEQUENCE.
06/28/1966	10-15-53.3	35.92	120.53		2.1			PARKFIELD SEQUENCE.
06/28/1966	10-20-16.4	35.85	120.42		2.3			PARKFIELD SEQUENCE.
06/28/1966	10-23-22.8	35.55	120.42		2.0			PARKFIELD SEQUENCE.
06/28/1966	10-23-22.8	35.94	120.48		2.5			PARKFIELD SEQUENCE.
06/28/1966	10-46-22.9	35.94	120.50		2.0			PARKFIELD SEQUENCE.
06/28/1966	11-15-13.9	35.85	120.42		2.0			PARKFIELD SEQUENCE.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/28/1966	11-28-41.4	35.85	120.38		2.0			PARKFIELD SEQUENCE.
06/28/1966	11-30-14.0	35.90	120.47		2.2			PARKFIELD SEQUENCE.
06/28/1966	12-31-52.1	35.94	120.48		2.5			PARKFIELD SEQUENCE.
06/28/1966	12-52-22.0	35.97	120.53		2.3			PARKFIELD SEQUENCE.
06/28/1966	13-48-22	35.97	120.53		2.7			PARKFIELD SEQUENCE.
06/28/1966	14-13-09.3	35.94	120.48		2.6			PARKFIELD SEQUENCE.
06/28/1966	14-21-36.3	35.94	120.48		2.2			PARKFIELD SEQUENCE.
06/28/1966	14-51-53.6	35.90	120.47		2.3			PARKFIELD SEQUENCE.
06/28/1966	18-12-19.4	35.92	120.50		2.3			PARKFIELD SEQUENCE.
06/28/1966	18-22-32.4	35.92	120.50		2.0			PARKFIELD SEQUENCE.
06/28/1966	18-54-55.3	35.88	120.45		2.5			PARKFIELD SEQUENCE.
06/28/1966	19-59-37.8	35.92	120.47		2.8			PARKFIELD SEQUENCE.
06/28/1966	20--?-38.7	35.92	120.48		2.5			PARKFIELD SEQUENCE.
06/28/1966	20-46-56.4	35.77	120.40		3.1		F	PARKFIELD SEQUENCE - FELT AT BAR B RANCH AND WORK RANCH.
06/28/1966	22-01-13.9	35.85	120.44		2.0			PARKFIELD SEQUENCE.
06/28/1966	22-37-56.7	35.88	120.42		2.0			PARKFIELD SEQUENCE.
06/28/1966	23-57-22.3	35.77	120.35		2.5			PARKFIELD SEQUENCE.
06/29/1966	-?-17-32.6	35.85	120.44		2.3			PARKFIELD SEQUENCE.
06/29/1966	02-19-39.9	35.92	120.52		3.6		F	PARKFIELD SEQUENCE - FELT AT CHOLAME, PARKFIELD, AND WORK RANCH.
06/29/1966	04-06-40.3	35.92	120.53		2.8			PARKFIELD SEQUENCE.
06/29/1966	07-28-59.4	35.92	120.48		2.3			PARKFIELD SEQUENCE.
06/29/1966	08-55-52.4	35.88	120.45		2.9			PARKFIELD SEQUENCE.
06/29/1966	09-20-50.1	35.78	120.36		2.5			PARKFIELD SEQUENCE.
06/29/1966	10-13-44.0	35.97	120.50		2.3			PARKFIELD SEQUENCE.
06/29/1966	10-56-58.8	35.75	120.33		3.0			PARKFIELD SEQUENCE.
06/29/1966	12-30-09.0	35.94	120.50		2.4			PARKFIELD SEQUENCE.
06/29/1966	13-11-59.7	35.82	120.38		3.1		F	PARKFIELD SEQUENCE - FELT AT CHOLAME AND PARKFIELD.
06/29/1966	15-18-38.9	35.95	120.33		2.0			PARKFIELD SEQUENCE.
06/29/1966	15-34-22.2	35.92	120.48		2.3			PARKFIELD SEQUENCE.
06/29/1966	16-03-30.1	35.86	120.45		2.1			PARKFIELD SEQUENCE.
06/29/1966	17-10-28.3	35.82	120.36		2.0			PARKFIELD SEQUENCE.
06/29/1966	19-53-25.9	35.95	120.53		5.0		F	PARKFIELD SEQUENCE - FELT AT ADELAIDA, BITTERWATER, CHOLAME, COALINGA, FRESNO, MEE RANCH, MORRO BAY, SAN LUIS OBISPO, SAN MIGUEL, SANTA MARGARITA, SHANDON, AND WORK RANCH.
06/29/1966	20-44-40.0	35.74	120.28		2.5			PARKFIELD SEQUENCE.
06/29/1966	23-48-12.0	35.74	120.28		2.3			PARKFIELD SEQUENCE.
06/30/1966	01-17-36.1	35.86	120.45		4.1			PARKFIELD SEQUENCE.
06/30/1966	03-36-16.8	35.92	120.47		2.6			PARKFIELD SEQUENCE.
06/30/1966	05-04-12.9	35.88	120.45		2.0			PARKFIELD SEQUENCE.
06/30/1966	06-07-21.5	35.94	120.48		2.4			PARKFIELD SEQUENCE.
06/30/1966	06-23-32.4	35.90	120.47		2.1			PARKFIELD SEQUENCE.
06/30/1966	07-37-12.1	35.90	120.47		2.0			PARKFIELD SEQUENCE.
06/30/1966	08-01-38.4	35.90	120.47		2.9			PARKFIELD SEQUENCE.
06/30/1966	11-07-55.1	35.78	120.33		2.8			PARKFIELD SEQUENCE.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/30/1966	13-26-05.7	35.78	120.35		2.3			PARKFIELD SEQUENCE.
06/30/1966	13-29-56.6	35.86	120.40		2.0			PARKFIELD SEQUENCE.
06/30/1966	13-40-50.9	35.83	120.38		2.1			PARKFIELD SEQUENCE.
06/30/1966	16-05-02.7	35.97	120.50		2.3			PARKFIELD SEQUENCE.
06/30/1966	19-06-17.5	35.86	120.42		2.1			PARKFIELD SEQUENCE.
07/01/1966	09-41-21.9	35.94	120.52	3.2			F	PARKFIELD SEQUENCE - FELT AT WORK RANCH.
07/02/1966	12-08-34.8	35.79	120.33		3.7		F	PARKFIELD SEQUENCE - FELT AT PARKFIELD.
07/02/1966	12-16-15.8	35.81	120.35		3.4		F	PARKFIELD SEQUENCE - FELT AT PARKFIELD.
07/02/1966	12-25-06.8	35.80	120.35		3.1		F	PARKFIELD SEQUENCE - FELT AT PARKFIELD.
07/05/1966	18-54-54.5	35.92	120.48		3.0		F	PARKFIELD SEQUENCE - FELT AT PARKFIELD.
07/25/1966	22-49-39	36.40	120.30		2.5	4		NE OF COALINGA.
07/27/1966	08-12- 0.2	35.90	120.48		3.0			PARKFIELD SEQUENCE.
08/03/1966	12-39-05.8	35.80	120.38		3.4		F	PARKFIELD SEQUENCE; V AT CHOLAME, PARKFIELD, AND WORK RANCH.
08/04/1966	-?-54-24.5	35.74	121.35		3.0	8		NW OF SAN SIMEON.
08/07/1966	17-03-24.9	35.94	120.55		3.0			PARKFIELD SEQUENCE.
08/19/1966	22-51-20.1	35.90	120.45		3.3			PARKFIELD SEQUENCE.
09/07/1966	-?-20-50.5	35.83	119.94		3.2	9		SE OF COALINGA.
09/18/1966	15-09-55.7	35.74	120.35		3.1			PARKFIELD SEQUENCE.
10/27/1966	12-06-03.9	35.94	120.50		3.8		F	PARKFIELD SEQUENCE; V AT ATASCADERO, AVENAL, COALINGA, PARKFIELD, SAN MIGUEL, TEMPLETON, AND WORK RANCH.
11/05/1966	13-31-31.2	35.94	120.50		3.3			PARKFIELD SEQUENCE.
11/18/1966	23-39-42.3	35.75	120.33		3.3			PARKFIELD SEQUENCE.
12/30/1966	10-23-48	36.47	120.40		2.5	4		N OF COALINGA.
01/08/1967	23-03-50.9	35.90	120.40		2.8	8		35 KM SE OF PRIEST (UC BERKELEY SS).
01/09/1967	23-18-59.5	35.86	120.10		3.1	9		SE OF COALINGA.
02/01/1967	13-55-54.1	35.70	120.25		3.0	8		NE OF SAN LUIS OBISPO.
02/26/1967	15-17-53.9	36.40	121.06		2.5	9		SW OF LLANADA.
03/13/1967	21-59-48.4	36.00	120.61		3.1	8	F	15 KM S OF PRIEST (UC BERKELEY SS). IV AT SAN MIGUEL; FELT AT INDIAN VALLEY AND RANCHITO CANYON.
03/21/1967	02-24-28.3	36.21	120.85		2.8	8		17 KM NW OF PRIEST (UC BERKELEY SS).
03/23/1967	11-39-56.4	36.16	120.18		3.0	5		20 KM E OF COALINGA.
04/13/1967	09-06-42.5	36.15	120.80		2.7	8		13 KM W OF PRIEST (UC BERKELEY SS).
05/17/1967	14-16-52.2	35.95	120.73		3.0	6		30 KM S OF PRIEST (UC BERKELEY SS).
06/03/1967	20-10-53.0	35.71	121.48		2.6	7		OFF COAST NW OF SAN SIMEON.
06/06/1967	06-11-38.5	35.81	120.43		3.0	10	F	40 KM SE OF PRIEST (UC BERKELEY SS); IV AT WORK RANCH; FELT IN INDIAN VALLEY, SOUTHERN MONTEREY COUNTY, AND VINEYARD CANYON.
06/13/1967	12-54-10.7	35.81	121.50		3.3	10		OFF COAST, 35KM NW OF SAN SIMEON.
07/24/1967	07-08-52.9	35.96	120.50		3.7	9		PARKFIELD AREA.
07/28/1967	14-44-40.1	35.75	121.38		3.0	6		NEAR SAN SIMEON.
08/01/1967	22-14-13.0	35.75	121.40		2.7	6		NW OF SAN SIMEON.
08/08/1967	18-11-20.3	36.42	120.42		2.5	7		N OF COALINGA.
08/12/1967	18-57-40.4	35.80	120.45		4.1	18	F	PARKFIELD AREA; V AT ESTRELLA AREA, HOG CANYON ROAD TO PARKFIELD, AND SHANDON, AND IV AT CHOLAME.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
08/12/1967	23-21-07.8	36.11	120.80		2.8	6		SE OF KING CITY.
08/12/1967	23-22-05.3	36.13	120.76		2.5	7		SE OF KING CITY.
08/17/1967	23-12-02.7	35.91	121.50		2.6	5		NW OF SAN SIMEON.
08/25/1967	02-28-14.4	35.81	121.27		2.7	6		NW OF SAN SIMEON.
08/25/1967	16-35-27.8	36.05	120.00		3.2	7		SE OF COALINGA.
08/25/1967	16-40-50.2	36.01	119.95		3.0	7		SE OF COALINGA.
08/31/1967	18-10-40.4	35.86	121.35		2.8	7		NW OF SAN SIMEON.
09/09/1967	21-35-05.6	35.81	121.63		2.4	3		OFF SHORE SAN SIMEON.
10/14/1967	12-02-43.6	36.50	120.61		2.7	5		NEAR MT. CIERVO.
10/21/1967	12-05-21.8	35.83	120.46		3.1	7		PARKFIELD AREA.
10/25/1967	23-05-30.5	35.73	121.45		2.6	4		NEAR SAN SIMEON.
11/11/1967	22-10-06.8	36.50	120.81		3.3	9		S OF PANOCHE VALLEY.
11/11/1967	22-33-47.5	36.48	120.78		2.8	6		S OF PANOCHE VALLEY.
11/12/1967	07-11-20.4	36.48	120.80		2.6	7		S OF PANOCHE VALLEY.
11/14/1967	-?--?-51.7	35.78	120.53		3.1	3		PARKFIELD.
11/25/1967	15-27-43.4	36.46	121.06		2.5	6		BEAR VALLEY.
12/21/1967	05-13-11.3	35.36	120.85		2.6	3		S OF SAN SIMEON.
12/21/1967	19-08-53.8	35.91	119.53		3.1	5		NW OF DELANO.
12/21/1967	23-58-60.2	35.93	120.56		3.0	3		PARKFIELD.
12/31/1967	23-48-13.5	35.75	120.45		4.3	3	F	PARKFIELD AREA; V AT CRESTON, PARKFIELD, SALINAS DAM, SAN MIGUEL, SHANDON, TEMPLETON, AND WORK RANCH.
02/03/1968	19-07-26.4	35.73	121.25		2.8	5		NEAR SAN SIMEON.
02/23/1968	20-20-57.9	35.86	121.31		2.5	7		EAST OF HOLLISTER.
03/25/1968	11-32-07.4	36.37	120.70		3.6	8	F	SE OF LLANADA; MAXIMUM INTENSITY V.
03/28/1968	04-53-26.5	36.36	120.19		3.1	5	F	SE OF COALINGA; FELT AT AVENAL - INTENSITY IV.
04/14/1968	06-20-54.6	36.18	121.65		2.5	6		SE OF MONTEREY.
04/23/1968	15-09-14.9	35.52	120.82		3.4	7		SE OF SAN SIMEON.
04/27/1968	14-32-37.4	36.22	120.83		2.7	7		NW OF PRIEST (UC BERKELEY SS).
04/28/1968	06-31-32.9	35.46	120.83		3.5	7		NW OF SAN LUIS OBISPO.
05/31/1968	07-07-37.9	35.80	120.60		3.0	5		S OF COALINGA.
06/11/1968	11-43-28.1	35.90	121.70		3.3	9		OFFSHORE, NW OF SAN SIMEON.
06/22/1968	12-50-50.1	36.43	121.04		2.9	9		S OF LLAN
07/03/1968	17-52-52	35.80	121.50		2.5	7		NW OF SAN SIMEON.
07/29/1968	04-27-51.9	36.38	120.69		2.7	9		N OF PRIEST (UC BERKELEY SS).
07/29/1968	05-29-19.9	36.37	120.70		2.8	9		N OF PRIEST (UC BERKELEY SS).
07/31/1968	-?-49-25.4	36.37	120.70		2.9	9		N OF PRIEST (UC BERKELEY SS).
08/19/1968	16-30-18.2	36.40	121.91		3.3	9		S OF CARMEL.
09/01/1968	21-56-24.4	36.45	121.02		2.7	8		E OF PINNACLES NATIONAL MONUMENT.
11/06/1968	08-58-23.2	35.88	120.45		2.8	10	F	NEAR PARKFIELD; FELT NEAR SAN MIGUEL.
11/10/1968	04-06-03.9	35.70	121.18		3.2	9		NEAR SAN SIMEON.
11/17/1968	01-03-47.0	36.29	120.94		3.0	6		NEAR KING CITY.
12/11/1968	12-19-52.4	35.81	120.48		3.0	10		NEAR PARKFIELD.
12/16/1968	01-14-10.9	36.17	120.85		2.7	7		W OF PRIEST (UC BERKELEY SS).
01/09/1969	09-42-47.2	35.94	120.57		3.8	7	F	CHOLAME VALLEY; FELT IN PARKFIELD AND SLACK CANYON - MAXIMUM INTENSITY V.
02/04/1969	-?-45-25	36.40	120.38		3.0	4		NORTH OF COALINGA.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/19/1969	07-05-08	36.12	119.58		3.5	8	F	NEAR TULARE; FELT IN CORCORAN, DINUBA, HANFORD, IVANHOE, LEMON COVE, STRATHMORE, AND TIPTON. MAXIMUM INTENSITY IV.
06/24/1969	14-25-37	36.42	120.13		3.0	4		SOUTHWEST OF FRESNO.
07/16/1969	04-06-35	35.83	120.28		3.2	7		15 KM SOUTHEAST OF PARKFIELD.
09/06/1969	13-44-45	35.30	121.10		3.8	10	F	50 KM WEST OF SAN LUIS OBISPO.
09/16/1969	03-32-24	36.18	120.80		2.5	8		13 KM WEST OF PRIEST (UC BERKELEY SS).
10/02/1969	06--?-58.9	36.32	120.32		3.3	10		10 KM NORTH OF COALINGA.
11/17/1969	20-49-10.4	36.43	121.05		4.4	10	F	NNE OF KING CITY; FELT IN MONTEREY - SWAYED BUILDINGS IN SALINAS
11/19/1969	06-23-50	36.45	121.52		4.2	8	F	GONZALES AND SALINAS VALLEY; FELT IN SALINAS AND SANTA CRUZ - RATTLED WINDOWS IN MONTEREY.
11/26/1969	-?-06-59	36.48	120.60		2.5	6		50 KM NORTHEAST OF KING CITY.
11/30/1969	15-11-54	35.30	120.90		2.5	10		20 KM EAST OF KING CITY; 2 SMALL FORESHOCKS RECORDED.
12/10/1969	13-25-31	35.75	120.40		3.5	7		40 KM SOUTH OF COALINGA.
12/14/1969	19-07-57	35.92	120.68		3.2	9		20 KM NORTH OF PASO ROBLES.
01/29/1970	02-49-12.9	36.11	120.99		2.5	6		20 KM SOUTHWEST OF KING CITY.
02/01/1970	21-19-45.7	36.41	121.08		2.6	12		30 KM EAST OF PARAISO.
02/08/1970	-?-14-13.3	36.40	120.97		2.7	13		25 KM SOUTH OF LLANADA.
02/09/1970	16--?-46.1	35.77	120.35		3.1	16		60 KM SOUTH OF PRIEST (UC BERKELEY SS).
02/14/1970	15-44-58.0	36.09	120.64		2.8	14		5 KM SOUTH OF PRIEST (UC BERKELEY SS).
04/18/1970	13-16-53.4	36.49	120.01		3.0	15		35 KM SOUTHWEST OF FRESNO.
04/21/1970	22-29-25.9	35.66	120.43		3.0	8		65 KM SOUTH OF PRIEST (UC BERKELEY SS).
04/23/1970	03-25-18.9	35.97	121.45		2.5	10		25 KM SOUTHWEST OF KING CITY.
05/27/1970	10-42-19.3	35.99	120.91		3.4	8		40 KM SOUTHWEST OF PRIEST (UC BERKELEY SS).
07/20/1970	23-24-55	35.95	121.57		2.5	5		8 KM SOUTH OF LOPEZ POINT - OFFSHORE.
07/21/1970	05-24-16.1	35.99	121.57		2.5	5		5 KM SOUTHEAST OF LOPEZ POINT.
08/05/1970	06-47-36.4	35.82	119.94		2.9	8		KETTLEMAN HILLS.
08/05/1970	16-51-45.7	36.23	121.69		3.0	11		25 KM SOUTHWEST OF PARAISO.
08/13/1970	05-06-19.8	36.17	121.70		3.7	11		20 KM WEST OF LOPEZ POINT.
09/05/1970	11-29-11	36.20	120.10		3.1	4		EAST-NORTHEAST OF COALINGA.
09/10/1970	23-45-59	36.40	120.50		3.2	11		30 KM NORTHWEST OF COALINGA.
09/11/1970	15-20-08	35.98	120.05		3.3	9		8 KM EAST OF AVENAL.
09/16/1970	18-22-10.7	35.96	121.27		2.6	7		NEAR MILPITAS.
10/07/1970	17-57-06.3	36.30	121.40		2.5	9		30 KM NORTHWEST OF KING CITY.
12/01/1970	06-05-59	35.38	121.13		3.3	7	F	25 KM WEST OF MORRO BAY; INTENSITY V AT BRYSON - NO DAMAGE.
12/12/1970	22-29-20	35.65	121.55		2.5	6		30 KM WEST OF SAN SIMEON.
01/02/71	06-27-37.5	35°55.1'	120°32.2'		3.0			10 km NW of Parkfield
01/16/71	05-33-27.8	36°00'	120°12'		3.1			Kettleman Hills
01/26/71	21-53-53	35°12'	120°42'		3.0			Near San Luis Obispo.
01/31/71	12-22-49.5	35°55.6'	120°30.6'		3.0			NW of Parkfield; sharp, rapid jolting at Shandon.
04/05/71	01-40-34.2	36°24.8'	120°59.0'		3.0			20 km SE of Pinnacles National Monument.
04/19/71	09-35-58.8	36°13.7'	120°50.3'		3.0			25 km E of King City.
04/29/71	02-13-15.7	36°30.3'	120°32.5'		3.0			40 km NW of Coalinga.
06/20/71	12-41-39.8	35°3'	120°20'		3.4			Near Cholame.
07/06/71	09-24-35	35°34'	121°35'		3.0			SW of San Simeon.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/21/71	09-14-26.2	36°13.7'	120°50.8'		3.2			Near Coalinga.
08/06/71	20-03-16.3	36°00.8'	120°02.2'		3.0			Near Coalinga.
10/06/71	14-43-30.6	35°51.3'	120°22.5'		3.5			S of Coalinga; intensity IV at Cholame, Parkfield, and Shandon.
10/21/71	22-09-45.4	35°58.8'	120°50.2'		3.7			SE of King City; intensity V at San Ardo (small objects shifted) and intensity IV at Jolon, King City, Lockwood, Pine Canyon, and San Lucas.
11/07/71	14-03-30.4	35°31.2'	119°50.2'		4.0			SE of Coalinga.
11/18/71	04-03-52.4	36°14.5'	120°50.6'		3.4			NE of King City.
11/30/71	09-45-42.8	36°03.6'	119°53.4'		3.0			SE of Coalinga.

END OF SELECTED EARTHQUAKES

END OF QUAKES PROGRAM FOR SELECTION OF EARTHQUAKES

TABLE 2.5-2

SUMMARY, REVISED EPICENTERS OF REPRESENTATIVE SAMPLES OF EARTHQUAKES OFF THE COAST OF CALIFORNIA NEAR SAN LUIS OBISPO

Date	Event Number	Original Hypocenter / Revised Hypocenter Lat.	Long.	Distance Hypocenter Moved, km	Error Ellipse km	Mag., M_L
May 27, 1935	1	35.370 35.621	120.960 121.639	66NW	7 x 14	3.0
Sept. 7, 1939	6	35.420 35.459	121.070 121.495	40W	8 x 8	3.0
Oct. 6, 1939	7	35.800 36.232	121.500 121.763	54NW	16 x 31	3.5
July 11, 1945	8	35.670 35.809	121.250 121.408	21NW	7 x 24	4.0
Mar. 23, 1947	12	35.150 34.577	121.300 121.137	66S	12 x 24	3.7
Mar. 27, 1947	15	35.000 34.739	121.000 120.896	32SW	20 x 20	4.2
Dec. 20, 1948	9	35.800 35.683	121.500 121.364	16SE	9 x 38	4.5
Dec. 31, 1948	10	35.670 35.598	121.400 121.226	17SE	8 x 29	4.6
Nov. 22, 1952 Bryson Earthquake	17	35.730 35.830 35.836	121.190 121.170 121.204	U.C. Berkeley Richter (1969) 12N	7 x 24	6.0
Mar. 13, 1954	21	35.000 34.960	120.690 120.490	19E	9 x 18	3.4
Mar. 5, 1955	23	35.600 35.863	121.400 121.149	38NE	15 x 29	3.3
June 21, 1957	25A	35.100 35.255	120.900 120.951	15NW	10 x 19	3.7
Jan. 2, 1960	26	35.400 35.778	121.190 121.066	44NE	15 x 29	4.0
Feb. 1, 1962	52	34.880 35.031	120.670 120.846	22NW	6 x 16	4.5

TABLE 2.5-2

Date	Event Number	Original Hypocenter Revised Hypocenter Lat.	Long.	Distance Hypocenter Moved, km	Error Ellipse km	Mag., M_L
Mar. 5, 1962	54	34.600 34.622	121.590 121.416	17E	8 x 10	4.5
Mar. 10, 1962	54A	34.600 34.667	121.590 121.372	22NE	6 x 20	4.2
Feb. 22, 1963	28	35.110 34.730	121.440 121.400	42S	7 x 28	3.3
Sept. 6, 1969	31	35.300 35.355	121.090 121.033	9NE	5 x 10	3.6
Oct. 22, 1969	56	34.830 34.649	121.340 121.471	23SW	14 x 50	5.4

TABLE 2.5-3

DISPLACEMENT HISTORY OF FAULTS IN THE SOUTHERN COAST RANGES OF CALIFORNIA

Fault	Distance From Diablo Site, miles	Time of Principal Activity	Youngest Formation Cut By Fault	Oldest Formation Capping Fault
San Andreas	45	Mid-Tertiary - present		Currently active
Faults in ground between San Andreas and Sur-Nacimiento-Rinconada, La Panza, Cuyama, Red Hills, East Huasna	18-45	Tertiary	Pleistocene (possible Holocene) (Ref. 14)	Not Known
Sur-Nacimiento (zone)	18	Late Mesozoic, (Benioff-subduction zone)	Pleistocene (possible Holocene) (Ref. 14)	Late Quaternary terrace deposits (Ref. 11)
West Huasna-Suey	11	Late Tertiary	Post late-Miocene	Late Quaternary terrace deposits (Ref. 36)
Edna	4.5	Late Tertiary	Plio-Pleistocene (Paso Robles Fm)	Late Pleistocene (Ref. 20)
Miguelito	5	Late Tertiary	Early Pliocene (Miguelito Member of Careaga Fm) (Ref. 21)	Poss. capped by mid-Pliocene Squire Member of Careaga Fm; Plio-Pleistocene Paso Robles Fm

TABLE 2.5-3

Fault	Distance From Diablo Site, miles	Time of Principal Activity	Youngest Formation Cut By Fault	Oldest Formation Capping Fault
Faulting in the Mesozoic rocks near Pt. San Luis	4	Mesozoic	Mesozoic	Late Pleistocene (Ref. 20)
Unnamed faults near Pt. San Simeon	35	Probable Tertiary	Not known; possible Holocene	Not known
Offshore structural zone	4.5	Late Tertiary	Possible Holocene (Ref. 19) (northern part)	Holocene-upper Pliocene (Ref. 19) (southern part)
Faults in the Santa Maria Basin	40	Not known	Possible Pleistocene (orcutt Fm) (Ref. 23)	Pleistocene-Holocene

TABLE 2.5-1

LISTING OF EARTHQAKES WITHIN 75 MILES OF THE DIABLO CANYON POWER PLANT SITE
SELECTED EARTHQUAKES

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
-?/-?/1800	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
03/25/1806	08--?--?	34.50	119.67	D			F	VIII AT SANTA BARBARA.
12/21/1812	18--?--?	34.50	120.00	D			F	VIII AT SAN FERNANDO.
12/21/1812	19--?--?	34.50	120.00	D			F	IX AT SAN FERNANDO.
01/18/1815	-?--?--?	34.50	119.67	D			F	SANTA BARBARA; 5 SHOCKS.
01/30/1815	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
07/08/1815	-?--?--?	34.50	119.67	D			F	SANTA BARBARA; 6 SHOCKS ON THE EIGHTH AND NINTH.
-?/-?/1830	-?--?--?	35.25	120.67	D			F	VIII AT SAN LUIS OBISPO.
07/03/1841	-?--?--?	36.30	122.30		6.3			(CALTECH FILE)
06/13/1851	-?--?--?	35.25	120.67	D			F	V AT SAN LUIS OBISPO.
10/26/1852	-?--?--?	35.67	121.17	D			F	X AT SAN SIMEON; 11 SHOCKS.
12/17/1852	-?--?--?	35.25	120.67	D			F	IX AT SAN LUIS OBISPO; 2 SHOCKS.
01/10/1853	-?--?--?	35.25	120.67	D			F	DANA RANCHO.
01/29/1853	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
02/01/1853	21--?--?	35.67	121.17	D			F	VIII AT SAN SIMEON.
02/14/1853	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
03/01/1853	-?--?--?	34.50	119.67	D			F	V AT SAN LUIS OBISPO.
04/20/1854	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
04/29/1854	-?--?--?	34.50	119.67	D			F	III AT SANTA BARBARA.
05/03/1854	13-10--?	34.50	119.67	D			F	SANTA BARBARA; 3 SEVERE SHOCKS.
05/13/1854	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
05/29/1854	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
05/31/1854	12-50--?	34.50	119.67	D			F	VI AT SANTA BARBARA; 3 SHOCKS.
01/14/1855	02-30--?	35.75	120.67	D			F	SAN BENITO AND SAN MIGUEL.
06/25/1855	22--?--?	34.50	119.67	D			F	V AT SANTA BARBARA.
01/08/1857	14--?--?	34.50	119.67	D			F	SANTA BARBARA.
01/08/1857	17--?--?	34.50	119.67	D			F	SANTA BARBARA.
01/08/1857	18--?--?	34.50	119.67	D			F	SANTA BARBARA.
01/09/1857	07-20--?	34.50	119.67	D			F	IX AT SANTA BARBARA.
01/21/1857	-?--?--?	36.50	121.08	D			F	III AT A POINT NORTHWEST OF SAN BENITO.
03/14/1857	23--?--?	34.50	119.67	D			F	V AT MONTECITO AND SANTA BARBARA.
09/02/1858	-?--?--?	34.50	119.67	D			F	V AT SANTA BARBARA.
04/03/1860	04--?--?	36.50	121.08	D			F	VI AT SAN JOSE.
04/17/1860	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
-?/-?/1862	-?--?--?	34.42	119.63	D			F	VIII AT GOLETA.
09/13/1869	-?--?--?	35.25	120.67	D			F	V AT SAN LUIS OBISPO.
09/14/1869	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
12/15/1869	-?--?--?	35.25	120.67				F	V AT SAN LUIS OBISPO.
02/06/1872	-?--?--?	34.50	119.67	D			F	SANTA BARBARA; FIRST SINCE APRIL 1860.
11/07/1875	-?--?--?	36.50	121.08	D			F	V IN SAN BENITO COUNTY.
12/21/1875	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
05/10/1876	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
05/30/1877	-?--?--?	35.67	120.67	D			F	V AT PASO ROBLES.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/24/1877	07-30--?	34.50	119.67	D			F	SANTA BARBARA.
01/08/1878	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
11/13/1880	06-30--?	34.50	119.67	D			F	SANTA BARBARA.
02/02/1881	-?--?--?	36.37	121.67	D			F	III AT SALINAS.
08/31/1881	03--?--?	34.50	119.67	D			F	III AT SANTA BARBARA.
09/13/1883	22-30--?	34.50	119.67	D			F	IV AT SANTA BARBARA.
08/03/1884	-?--?--?	34.50	119.67	D			F	III AT SANTA BARBARA; NIGHT.
08/04/1884	09--?--?	34.50	119.67	D			F	III AT SANTA BARBARA; 3 SHOCKS.
03/31/1885	-?--?--?	36.30	121.00		7.0			(CALTECH FILE)
04/07/1885	10--?--?	34.50	119.67	D			F	SANTA BARBARA AND SAN BUENAVENTURA.
04/09/1885	-?--?--?	35.58	121.08	D			F	CAMBRIA.
04/12/1885	04-05--?	36.25	120.80	D			F	IX IN CENTRAL CALIFORNIA; FELT OVER AN AREA OF 125,000 SQ. MI.- EPICENTER PROBABLY EAST OF KING CITY.
04/12/1885	11--?--?	36.33	119.67	D			F	HANFORD.
07/09/1885	09-15--?	34.50	119.67	D			F	V AT SANTA BARBARA.
07/09/1885	16-15--?	34.50	119.67	D			F	V AT SANTA BARBARA; 5 EARTHQUAKES.
10/03/1888	20-52--?	35.75	120.67	D			F	III AT SAN MIGUEL.
10/03/1888	21-02--?	35.75	120.67	D			F	VI AT SAN MIGUEL.
10/04/1888	-?--?--?	35.67	120.67	D			F	PASO ROBLES.
05/01/1889	19-55--?	34.67	120.42	D			F	SUSANVILLE.
05/26/1889	15-13--?	36.50	121.42	D			F	GONZALES, SAN FRANCISCO, AND SANTA CRUZ; RECORDED AT MT. HAMILTON.
07/10/1889	-?--?--?	35.17	120.58	D			F	ARROYO GRANDE; SHOCKS FOR SEVERAL DAYS.
09/30/1889	20-17--?	36.50	119.58	D			F	KINGSBURG.
01/-?/1890	23-30--?	34.50	119.67	D			F	SANTA BARBARA.
11/13/1892	-?--?--?	36.30	122.00		6.0			(CALTECH FILE)
05/19/1893	-?-35--?	34.17	119.50	D			F	VII FELT FROM SAN DIEGO TO LOMPOC, INLAND TO SAN BERNADINO. MOST SEVERE SE OF VENTURA. POSSIBLY OF SUBMARINE ORIGIN OFF THE COAST OF VENTURA COUNTY
06/01/1893	12--?--?	34.50	119.67	D			F	VII AT NORDHOFF (OJAI), SANTA BARBARA, AND VENTURA.
06/01/1893	12--?--?	34.50	119.67	D			F	NORDHOFF, SANTA BARBARA, AND VENTURA.
06/01/1893	12-10--?	34.50	119.67	D			F	NORDHOFF, SANTA BARBARA, AND VENTURA.
12/06/1893	04-56--?	35.67	121.33	D			F	PIEDRAS BLANCAS LIGHTHOUSE.
07/27/1895	-?-10--?	34.50	119.67	D			F	SANTA BARBARA.
12/24/1895	05-30--?	34.50	119.67	D			F	SANTA BARBARA.
06/24/1897	14-10--?	34.50	119.67	D			F	SANTA BARBARA.
07/18/1897	-?--?--?	34.50	119.67	D			F	CASTLE PINCKNEY.
07/20/1897	07-45--?	34.50	119.67	D			F	SANTA BARBARA.
05/30/1898	03-03--?	34.50	119.67	D			F	SANTA BARBARA.
06/04/1898	06-20--?	34.67	120.08	D			F	LOS OLIVOS; FELT THROUGHOUT THE SANTA YNEZ VALLEY; AT SANTA BARBARA THE HEAVIEST FOR SOME YEARS.
02/08/1899	04-55--?	36.33	121.92	D			F	POINT SUR LIGHT STATION.
06/05/1899	-?--?--?	35.83	120.83	D			F	BRADLEY.
06/25/1899	-?--?--?	35.75	120.67	D			F	SAN MIGUEL.
06/09/1900	-?--?--?	36.00	120.92	D			F	SAN ARDO.
10/18/1900	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
03/03/1901	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/03/1901	07-45--?	36.08	120.58	D			F	IX AT STONE CANYON - SURFACE CRACKS IN THE GROUND; ALSO FELT AT ADELAIDA, ESTRELLA, PARKFIELD, PASO ROBLES, PORTERVILLE, SAN JOSE, SAN LUIS OBISPO, AND SAN MIGUEL.
03/05/1901	-?--?--?	35.67	120.67	D			F	PASO ROBLES.
03/06/1901	-?--?--?	36.00	120.92	D			F	SAN ARDO AND SAN LUIS OBISPO.
06/03/1901	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/30/1901	19--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
08/14/1901	11-11--?	35.42	120.92	D			F	CAYUCOS, HOLLISTER, SALINAS, SAN LUIS OBISPO, AND SANTA CRUZ.
02/07/1902	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
02/09/1902	15--?--?	34.50	119.67	D			F	PINE CREST, SAN LUIS OBISPO, SANTA BARBARA, AND VENTURA.
04/06/1902	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/21/1902	-?--?--?	34.75	120.00	D			F	PINE CREST.
07/28/1902	06-57--?	34.75	120.25	D			F	IX AT LOMPOC AND LOS ALAMOS; CONFINED TO THE NORTHERN PART OF SANTA BARBARA COUNTY.
07/28/1902	13--8--?	35.25	120.67	D			F	SAN LUIS OBISPO; AFTERSHOCK OF 06-57-?.
07/31/1902	09-20--?	34.75	120.25	D			F	IX AT LOS ALAMOS AND SURROUNDING COUNTRY; FISSURES, CRACKS IN THE GROUND, AND LANDSLIDES.
08/01/1902	-?--?--?	34.75	120.25	D			F	LOS ALAMOS. SEVERAL SHOCKS.
08/01/1902	03-30--?	34.75	120.25	D			F	VIII AT LOS ALAMOS.
08/02/1902	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
08/03/1902	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
08/04/1902	10 -5--?	34.75	120.25	D			F	LOS ALAMOS.
08/04/1902	11-18--?	34.75	120.25	D			F	LOS ALAMOS.
08/04/1902	12-15--?	34.75	120.25	D			F	LOS ALAMOS.
08/04/1902	21-29--?	34.75	120.25	D			F	LOS ALAMOS.
08/04/1902	23-40--?	34.75	120.25	D			F	LOS ALAMOS.
08/05/1902	-?-55--?	34.75	120.25	D			F	LOS ALAMOS.
08/10/1902	-?--?--?	34.75	120.25	D			F	LOS ALAMOS; DISTINCT EARTHQUAKE DETONATION AND TREMOR.
08/10/1902	10-40--?	34.75	120.25	D			F	LOS ALAMOS; HEAVY DETONATION FOLLOWED BY TREMBLING.
08/10/1902	22-40--?	34.50	119.67	D			F	SANTA BARBARA.
08/14/1902	10-15--?	34.75	120.25	D			F	LOS ALAMOS.
08/14/1902	11-05--?	34.75	120.25	D			F	LOS ALAMOS.
08/14/1902	11-20--?	34.75	120.25	D			F	LOS ALAMOS; SHOOK GROUND VIOLENTLY.
08/14/1902	21-50--?	34.75	120.25	D			F	LOS ALAMOS.
08/14/1902	23-50--?	34.75	120.25	D			F	LOS ALAMOS.
08/28/1902	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
08/31/1902	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
09/11/1902	05-30--?	34.25	120.25	D			F	V AT LOS ALAMOS.
10/21/1902	21-45--?	34.75	120.25	D			F	LOMPOC AND LOS ALAMOS.
10/21/1902	22-15--?	34.75	120.25	D			F	LOMPOC AND LOS ALAMOS.
10/22/1902	10--?--?	34.75	120.25	D			F	LOS ALAMOS.
12/12/1902	-?--?--?	34.75	120.25	D			F	VIII AT LOS ALAMOS -3 SHOCKS IN 5 MINUTES; FELT THROUGHOUT THE NORTHERN PART OF SANTA BARBARA COUNTY, ESPECIALLY AT LOMPOC, LOS ALAMOS, SAN LUIS OBISPO, SANTA BARBARA, AND SANTA MARIA.
01/11/1903	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/07/1903	-?--?--?	36.50	121.42	D			F	GONZALES.
03/24/1903	-?--?--?	36.50	121.42	D			F	GONZALES AND SANTA MARGARITA.
04/24/1903	-?--?--?	35.42	120.58	D			F	SANTA MARGARITA.
07/29/1903	07-13--?	35.67	121.33	D			F	V AT POINT PIEDRAS BLANCAS LIGHTHOUSE.
07/29/1903	10-30--?	35.67	121.33	D			F	POINT PIEDRAS BLANCAS LIGHTHOUSE.
08/24/1903	-?--?--?	34.67	120.08	D			F	LOS OLIVOS.
01/22/1904	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
01/23/1904	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
09/10/1904	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
05/26/1905	05-49--?	35.25	120.67	D			F	LOS GATOS, SALINAS, SAN FRANCISCO, SAN LUIS OBISPO, SANTA CRUZ AND SOLEDAD.
07/06/1906	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/22/1906	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
08/01/1906	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
12/07/1906	06-40--?	35.67	121.33	D			F	VII AT SAN LUIS OBISPO AND SANTA MARIA; DURATION 30 SECONDS, FOLLOWED BY SECOND SHOCK HALF AN HOUR LATER.
+12/08/1906	06-55--?	35.75	120.67	D			F	SAN MIGUEL.
06/19/1907	12--?--?	36.17	120.67	D			F	PRIEST VALLEY.
07/02/1907	18-10--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/21/1907	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/29/1907	05-10--?	34.75	120.00	D			F	PINE CREST.
08/-?/1907	-?--?--?	34.75	120.00	D			F	PINE CREST AND SANTA BARBARA.
12/27/1907	09-15--?	34.50	119.67	D			F	SANTA BARBARA; ALSO FELT AT VENTURA; REPORTED FROM OJAI AND PINE CREST.
04/27/1908	10-50--?	36.00	121.17	D			F	JOLON, PASO ROBLES, PRIEST VALLEY, SAN LUIS OBISPO, SANTA MARGARETA, AND SAN MIGUEL.
05/19/1908	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
09/16/1908	-?--?--?	36.17	120.67	D			F	PRIEST VALLEY.
11/-?/1908	19-30--?	36.17	120.67	D			F	PRIEST VALLEY.
01/23/1909	14-58--?	34.50	119.67	D			F	PINE CREST AND SANTA BARBARA.
04/10/1909	-?--?--?	34.50	119.67	D			F	MONO RANCH AND SANTA PAULA CANYON.
06/17/1909	08-20--?	36.42	121.33	D			F	SOLEDAD.
07/03/1909	07--?--?	34.50	119.67	D			F	MONTECITO AND SANTA BARBARA.
07/05/1909	06-10--?	34.50	119.67	D			F	III AT SANTA BARBARA.
07/16/1909	10-28--?	34.50	119.67	D			F	IV AT LOS ANGELES AND SANTA BARBARA.
07/31/1909	19-37--?	34.50	119.67	D			F	IV AT OJAI AND SANTA BARBARA.
08/18/1909	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
11/24/1909	15--?--?	36.00	121.17	D			F	JOLON.
03/08/1910	09-30--?	36.17	120.67	D			F	PRIEST VALLEY.
04/30/1910	18-25--?	36.17	120.67	D			F	PRIEST VALLEY; 3 SHOCKS, THE SECOND ONE QUITE VIOLENT.
11/-?/1910	-?--?--?	34.50	119.67	D			F	SANTA BARBARA; 2 SLIGHT QUAKES DURING NOVEMBER.
02/02/1911	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
03/22/1911	10-55--?	35.75	120.67	D			F	SAN MIGUEL; QUITE SEVERE.
06/02/1911	-?--?--?	36.17	120.67	D			F	PRIEST VALLEY.
06/18/1912	22-27--?	36.00	121.17	D			F	JOLON. (RECORDED AT BERKELEY.)
10/20/1913	11-25--?	35.25	120.67	D			F	BETTERAVIA, PASO ROBLES, SAN LUIS OBISPO,AND SANTA MARIA.
11/27/1913	19--?--?	34.50	119.67	D			F	MONO RANCH.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/26/1913	12--?--?	36.17	121.00	D			F	SAN LUCAS.
11/24/1914	04-25--?	35.25	120.67	D			F	II AT SAN LUIS OBISPO; ABRUPT TREMBLING, LASTING 20 SECONDS.
01/12/1915	-?--?--?	34.92	120.50	D			F	BETTERAVIA.
01/12/1915	04-31--?	34.75	120.25	B			F	VIII AT LOS ALAMOS - EPICENTER 2 OR 3 MI. EAST OF LOS ALAMOS; FELT FROM SAN JOSE TO LOS ANGELES; SHAKEN AREA IN EXCESS OF 50,000 SQ. MI. - PRACTICALLY EVERY CHIMNEY DAMAGED AT LOS ALAMOS, VII AT LOMPOC, VI-VII AT SANTA MARIA, V AT SAN LUIS OBISPO AND SANTA BARBARA, IV AT PASO ROBLES, AND II AT LOS ANGELES, WEATHER BUREAU REPORTED V-VI AT SANTA BARBARA, V AT OZENA AND SAN LUIS OBISPO, IV AT PASO ROBLES, III AT OJAI, AND II IN PRIEST VALLEY; ALSO II AT BAKERSFIELD.
01/14/1915	-?--?--?	34.92	120.50	D			F	BETTERAVIA.
01/15/1915	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
01/20/1915	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
01/26/1915	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
01/27/1915	-?--?--?	34.75	120.25	D			F	LOS ALAMOS.
04/21/1915	09-58--?	35.25	120.67	D			F	IV AT SAN LUIS OBISPO; ALSO FELT 3 MI. NW OF PRIEST VALLEY.
08/23/1915	23-15--?	34.75	119.75	D			F	HILL CAMP.
08/31/1915	21--?--?	34.75	119.75	D			F	HILL CAMP.
09/08/1915	12-45--?	35.67	120.67	D			F	V IN REGION EAST OF PASO ROBLES; ANTELOPE - 2 SHOCKS, FIRST THE HEAVIER, OIL CAME UP WITH WATER IN WELL AFTER SHOCK. AT SHANDON A SEATED MAN WAS SHAKEN SO HARD HE THOUGHT A PERSON WAS SHAKING HIM. AT CRESTON THE SHOCK WAS SHORT AND SHARP. A SLIGHT LANDSLIDE AT PORT SAN LUIS. WEATHER BUREAU REPORTS -PASO ROBLES V AND SAN LUIS OBISPO III-IV.
09/14/1915	-?--?--?	34.75	119.75	D			F	HILL CAMP; 3 HARD SHOCKS - EARTH TREMBLED FOR 15 MINUTES AFTERWARDS.
02/27/1916	13-26--?	34.75	120.25	D			F	LOS ALAMOS.
03/01/1916	19-15--?	34.75	120.25	D			F	LOS ALAMOS.
05/06/1916	03-45--?	34.75	120.25	D			F	III AT LOS ALAMOS. FELT BY MANY AT EL ROBLAR RANCH, 2 MI. SE OF LOS ALAMOS.
08/06/1916	-?--?--?	36.00	121.00		7.0			(CALTECH FILE)
10/24/1916	13-03--?	35.25	120.67	D			F	II AT SAN LUIS OBISPO; PROBABLY NEXT SHOCK, WITH TIME ERROR.
10/24/1916	13-30--?	36.00	121.17	D			F	V AT JOLON; III AT A POINT 3.5 MI. NW OF PRIEST VALLEY.
12/01/1916	22-53--?	35.17	120.75	D			F	VII AT AVILA - CONSIDERABLE GLASS BROKEN AND GOODS IN STORES THROWN FROM SHELVES. FELT AT SAN LUIS OBISPO; WATER IN BAY DISTURBED, PLASTER IN COTTAGES JARRED LOOSE, SMOKESTACKS OF UNION OIL CO. REFINERY TOPPLED OVER. SEVERE AT PORT SAN LUIS; III AT SANTA MARIA.
02/01/1917	05-18--?	34.92	120.42	D			F	III AT SANTA MARIA.
04/05/1917	19--?--?	34.67	120.33	D			F	IV AT SANTA RITA; ALSO FELT AT LOMPOC.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
04/13/1917	03-59--?	34.25	119.67	D			F	VI AT SANTA BARBARA CHANNEL REGION; FELT OVER AN AREA OF COAST SOUTH AND EAST OF SANTA BARBARA AS FAR AS VENTURA, AND ON SANTA CRUZ ISLAND.
04/21/1917	06-59--?	34.25	119.67	D			F	V AT SANTA BARBARA CHANNEL; PERCEPTIBLE OVER AN AREA OF PERHAPS 4000 SQ. MI.
07/07/1917	20-57--?	35.25	120.50	D			F	LOPEZ CANYON; ALSO AT SAN LUIS OBISPO.
07/07/1917	21-02--?	35.25	120.50	D			F	LOPEZ CANYON.
07/07/1917	21-15--?	35.25	120.50	D			F	LOPEZ CANYON.
07/08/1917	03-20--?	34.92	120.42	D			F	II AT SANTA MARIA.
07/08/1917	11-29--?	35.25	120.50	D			F	IV IN LOPEZ CANYON.
07/09/1917	22-22--?	35.25	120.50	D			F	VII IN LOPEZ CANYON; IV AT SAN LUIS OBISPO.
07/09/1917	22-38--?	35.25	120.50	D			F	LOPEZ CANYON.
07/10/1917	-?-43--?	35.25	120.50	D			F	LOPEZ CANYON.
07/10/1917	-?-45--?	35.25	120.50	D			F	LOPEZ CANYON.
07/26/1917	08-31--?	34.92	120.42	D			F	V AT SANTA MARIA - FURNITURE MOVED. IV AT LOS OLIVOS - AWAKENED SLEEPERS AT SAN LUIS OBISPO
12/05/1918	02-38--?	35.67	120.67	D			F	IV AT PASO ROBLES; II AT SAN LUIS OBISPO.
12/05/1918	04-30--?	35.25	120.67	D			F	SAN LUIS OBISPO.
03/01/1919	04-19--?	36.17	120.67	D			F	IV IN PRIEST VALLEY.
03/15/1919	07-53--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/31/1919	21-31--?	36.33	120.67	D			F	V IN SAN BENITO COUNTY; FELT AT IDRIA - ORIGIN SOME DISTANCE FROM IDRIA
08/26/1919	12-12--?	34.50	119.67	D			F	V IN SANTA BARBARA COUNTY - FELT AT OJAI, SAN LUIS OBISPO (3 SHOCKS), SANTA BARBARA.
08/26/1919	14.57--?	34.50	119.67	D			F	V IN SANTA BARBARA COUNTY - THIS SHOCK STRONGER AT SANTA BARBARA THAN PREVIOUS SHOCK. BUILDINGS AND WHARVES SWAYED; FELT AT OJAI.
12/18/1919	07-15--?	35.67	120.67	D			F	PASO ROBLES.
01/30/1920	23-30--?	34.50	119.67	D			F	III AT SANTA BARBARA.
01/30/1920	23-33--?	34.50	119.67	D			F	II AT SANTA BARBARA.
01/30/1920	23-35--?	34.50	119.67	D			F	II AT SANTA BARBARA.
01/30/1920	23-38--?	34.50	119.67	D			F	II AT SANTA BARBARA.
01/31/1920	01--?--?	34.50	119.67	D			F	III AT SANTA BARBARA.
01/31/1920	01-03--?	34.50	119.67	D			F	III AT SANTA BARBARA.
01/31/1920	01-07--?	34.50	119.67	D			F	III AT SANTA BARBARA.
03/20/1920	07-04--?	35.25	120.67	D			F	II AT SAN LUIS OBISPO.
05/07/1920	01-59--?	35.25	120.67	D			F	IV AT SAN LUIS OBISPO.
06/28/1920	09-01--?	35.25	120.67	D			F	V AT SAN LUIS OBISPO.
12/01/1920	01-30--?	35.17	119.50	D			F	VI AT TAFT - MANY PEOPLE MADE "SEASICK", DISHES SHAKEN FROM SHELVES, IV AT MARICOPA.
12/05/1920	11-58--?	34.50	119.67	D			F	V IN SANTA BARBARA COUNTY MOUNTAINS, V AT LOMPOC, LOS ALAMOS, MARICOPA, OJAI, AND SANTA BARBARA.
12/06/1920	-?--?--?	35.25	120.67	D			F	SAN LUIS OBISPO.
03/10/1922	11-21-20	35.75	120.25	C	6.5	43	F	IX IN CHOLAME VALLEY REGION OF SAN ANDREAS FAULT. FELT OVER AN AREA OF 100,000 SQ. MI. - CRACKS IN THE GROUND AND NEW SPRINGS. VII-VIII AT PARKFIELD AND SHANDON. VI-VII AT SAN LUIS OBISPO AND SIMMLER, AND V AT LOS ANGELES.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/16/1922	23-10--?	35.75	120.33	D			F	VI IN CHOLAME VALLEY - RATHER STRONG AFTERSHOCKS, V AT PASO ROBLES AND SAN LUIS OBISPO, AND IV AT ANTELOPE VALLEY; ALSO IV AT SHANDON.
03/19/1922	11--?--?	35.67	120.67	D			F	III AT PASO ROBLES.
03/23/1922	10--?--?	35.67	120.67	D			F	III AT PASO ROBLES.
03/25/1922	12--?--?	35.67	120.67	D			F	III AT PASO ROBLES.
05/31/1922	01-25--?	35.67	120.67	D			F	III AT PASO ROBLES; 2 SHOCKS.
07/05/1922	19--?--?	34.75	120.25	D			F	LOS ALAMOS.
07/09/1922	12--?--?	34.75	120.25	D			F	LOS ALAMOS.
07/11/1922	03--?--?	34.75	120.25	D			F	LOS ALAMOS.
07/11/1922	15-30--?	34.75	120.25	D			F	LOS ALAMOS.
08/18/1922	05-12--?	35.75	120.33	D			F	VII IN CHOLAME VALLEY; V AT PASO ROBLES AND SAN LUIS OBISPO.
08/20/1922	21-14--?	35.50	120.67	D			F	III AT ATASCADERO.
09/04/1922	10-15--?	35.67	120.67	D			F	IV AT PASO ROBLES.
09/05/1922	09-05--?	35.25	120.67	D			F	V AT SAN LUIS OBISPO; 2 SHOCKS.
12/29/1922	11--?--?	35.67	120.67	D			F	III AT PASO ROBLES.
12/29/1922	12--?--?	35.67	120.67	D			F	III AT PASO ROBLES.
03/12/1923	06--?--?	34.75	120.25	D			F	LOS ALAMOS.
05/04/1923	22-45--?	35.25	120.67	D			F	V AT SAN LUIS OBISPO; 2 SHOCKS, SECOND EQUALED INTENSITY II.
05/08/1923	05-02--?	35.75	120.33	D			F	II AT CHOLAME.
06/16/1923	20-40--?	35.67	120.67	D			F	IV AT PASO ROBLES - DURATION 15-20 SECONDS.
06/25/1923	13-21--?	35.25	120.67	D			F	II AT SAN LUIS OBISPO.
12/19/1923	07-35--?	34.92	120.42	D			F	II AT SANTA MARIA - DURATION 20 SECONDS.
07/02/1924	58-02--?	34.50	119.67	D			F	SANTA BARBARA.
12/30/1924	12-17--?	34.50	119.67	D			F	SANTA BARBARA.
12/30/1924	14-15--?	34.50	119.67	D			F	SANTA BARBARA.
06/29/1925	14-42-16	34.30	119.80	B	6.3	1	F	IX AT SANTA BARBARA; FELT OVER AN AREA OF 100,000 SQ. MI. - RECORDED WORLD-WIDE. RUPTURE AT DEPTH ON THE MESA AND RECORDED WORLD-WIDE. RUPTURE AT DEPTH ON THE MESA AND SANTA YNEZ FAULTS (BAILEY WILLIS); A FEW DEATHS, SEVERAL MILLION DOLLARS DAMAGE; IX AT GOLETA, NAPLES, AND SANTA BARBARA; VIII AT GAVIOTA, MIRAMAR, AND SANTA YNEZ, LOS ALAMOS, LOS OLIVOS; VII AT ARROYO GRANDE, NIPOMO, ORCOTT, ALAMOS, LOS OLIVOS; VII AT ARROYO GRANDE, NIPOMO, ORCOTT, ALAMOS, LOS OLIVOS; VII AT ARROYO GRANDE, NIPOMO, ORCOTT, PISMO BEACH, SANTA MARIA, AND VENTURA, AND VI AT AVILA, LOMPOC, AND PORT SAN LUIS.
06/29/1925	15-20--?	35.25	120.67	D			F	III AT SAN LUIS OBISPO.
06/29/1925	16-35--?	34.50	119.67	D			F	SANTA BARBARA; II AT OXNARD.
06/29/1925	18-54--?	34.50	119.67	D			F	IV AT SANTA BARBARA; II AT OXNARD - STRONGEST AFTERSHOCK OF THE DAY.
06/30/1925	01-37--?	34.50	119.67	D			F	SANTA BARBARA.
06/30/1925	02-47--?	34.50	119.67	D			F	SANTA BARBARA.
06/30/1925	09-19--?	34.50	119.67	D			F	SANTA BARBARA - VIOLENT; FELT AT OJAI AND OXNARD.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/03/1925	16-38--?	34.50	119.67	D			F	VII AT SANTA BARBARA; III AT PASADENA AND OJAI - STIFF TREMOR AT VENTURA.
07/03/1925	18-21--?	34.50	119.67	D			F	VII AT SANTA BARBARA - STRONGEST AFTERSHOCK; FELT AT LOS ANGELES, OJAI, AND PASADENA.
07/03/1925	18-46--?	34.50	119.67	D			F	SANTA BARBARA.
07/04/1925	19-18--?	34.50	119.67	D			F	SANTA BARBARA - ANOTHER SHOCK FELT LATER IN DAY.
07/05/1925	12--?--?	34.50	119.67	D			F	SANTA BARBARA; 11 SHOCKS IN THE NEXT 19 HOURS.
07/06/1925	21-45--?	34.50	119.67	D			F	SANTA BARBARA - SEVERAL FAIRLY SEVERE SHOCKS.
07/09/1925	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
07/20/1925	09-50--?	34.50	119.67	D			F	SANTA BARBARA.
07/29/1925	14--?--?	34.50	119.67	D			F	V AT WASIOJA - CEMENT WALK CRACKED.
07/30/1925	09-50--?	34.50	119.67	D			F	SANTA BARBARA.
07/30/1925	12--?--?	34.50	119.67	D			F	SANTA BARBARA.
08/13/1925	11--?--?	34.50	119.67	D			F	SANTA BARBARA - 5 LIGHT SHOCKS DURING NIGHT; THE STRONGEST TOOK PLACE JUST BEFORE 11--?--?.
10/04/1925	-?-50--?	34.50	119.67	D			F	SANTA BARBARA.
10/08/1925	21-30--?	34.50	119.67	D			F	SANTA BARBARA.
10/30/1925	09-45--?	34.50	119.67	D			F	SANTA BARBARA.
10/30/1925	13-30--?	34.50	119.67	D			F	SANTA BARBARA AND VENTURA.
02/18/1926	18-18--?	34.17	119.50	D			F	VII ORIGIN AT SEA, SW OF VENTURA; FELT ALONG COAST FROM SAN LUIS OBISPO ON NW TO SOUTH OF SANTA ANA, A DISTANCE OF 200 MI. AT SANTA BARBARA WINDOWS OF A SCHOOL WERE BROKEN, WATER PIPE IN ROUNDHOUSE WAS BROKEN. THERE WAS DAMAGE TO TELEPHONE EQUIPMENT AT SIMI. ALSO FELT AT LOS ANGELES, PASADENA, SANTA MONICA, SANTA SUSANA, AND VENTURA.
04/29/1926	12-18--?	34.67	120.17	D			F	IV AT BUELLTON.
06/18/1926	-?--?--?	34.50	119.67	D			F	SANTA BARBARA.
06/24/1926	15-30--?	34.50	119.67	D			F	V AT SANTA BARBARA.
06/29/1926	23-21--?	34.50	119.67	D			F	VII-VIII AT SANTA BARBARA - ONE PERSON KILLED BY FALLING CHIMNEY. VI AT BUELLTON AND VENTURA; ALSO FELT AT CAMARILLO, LOS ANGELES, OJAI, OXNARD, PORT HUENEME, AND SANTA PAULA - POSSIBLY SUBMARINE ORIGIN; FELT OVER AN AREA OF 30,000 SQ. MI.
07/03/1926	23--?--?	34.50	119.67	D			F	II AT SANTA BARBARA.
07/06/1926	17-45--?	34.50	119.67	D			F	V AT SANTA BARBARA.
07/25/1926	-?--?--?	36.30	120.30					(CALTECH FILE)
08/06/1926	17-42--?	34.50	119.67	D			F	IV IN SANTA BARBARA REGION; 2 SHOCKS AT OJAI - LASTED 30 SECONDS AT VENTURA WITH SHARP SHOCK AT SANTA BARBARA.
08/09/1926	04-12--?	34.50	119.67	D			F	V AT SANTA BARBARA; 2 SHOCKS AT VENTURA.
10/22/1926	10-10--?	35.67	120.67	D			F	III AT PASO ROBLES.
10/22/1926	-?--?--?	36.45	122.00					(CALTECH FILE)
12/09/1926	-?-03--?	35.67	120.67	D			F	IV AT PASO ROBLES - PROBABLY MISTIMED REPORT OF SHOCK AT -? 41-?.
12/09/1926	-?-41--?	35.25	120.67	D			F	NE OF SAN LUIS OBISPO; AT SAN LUIS OBISPO DURATION 20 SECONDS; FELT AT COALINGA WITH ORIGIN ABOUT 120 MI. FROM MT HAMILTON.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/27/1926	09-19--?	36.17	120.33	D			F	VI NEAR COALINGA; FELT OVER AN AREA OF 25,000 SQ. MI. FELT AT FIREBAUGH, FRESNO, LOS BANOS, MENDOTA, OAKDALE, OILFIELDS, PORTERVILLE, AND SAN LUIS OBISPO.
11/04/1927	11--?--?	34.58	120.67	D			F	LOMPOC, POINT ARGUELLO, AND SAN LUIS OBISPO.
11/04/1927	11-30--?	34.58	120.67	D			F	LOMPOC.
11/04/1927	13-50-53	34.54	121.40	A	7.3	3	F	X AT SEA, WEST OF POINT ARGUELLO. AREA SHAKEN WITH INTENSITY VI OR GREATER WAS 40,000 SQ. MI. A SMALL SEA WAVE WAS PRODUCED, RECORDED ON TIDE GAUGES AT SAN DIEGO AND SAN FRANCISCO, AND OBSERVED AS 6 FEET HIGH AT SURF; IX AT HONDA, ROBERDS RANCH, SURF, AND WHITE HILLS, VIII AT ARLIGHT, ARROYO GRANDE, BERROS, BETTERAVIA, CAMBRIA, CASMALIA, CAYUCOS, GUADOCEANO, PISMO BEACH, POINT CONCEPTION, SAN JULIAN RANCH, SAN LUIS OBISPO, AND SANTA MARIA, VI-VII AT GUADOCEANO, PISMO BEACH, POINT CONCEPTION, SAN JULIAN RANCH, SAN LUIS OBISPO, AND SANTA MARIA, VI-VII AT ALUPE, HALCYON, HARRISTON, HUASNO, LOMPOC, LOS ALAMOS, LOS OLIVOS, MORRO BAY, NIPOMO, ADELAIDA, ATASCADERO, BAKERSFIELD, BICKNELL, BUTTONWILLOW,CARPINTERIA CHOLAME, CRESTON, EDNA GAVIOTA, GOLETA, HARMONY, KING CITY, LAS CRUCES, NAPLES, OXNARD, PASO ROBLES, REWARD, SANTA BARBARA, SANTA MARGARITA, SANTA YNEZ, SOLVANG, TAFT, TEMPLETON, VENTURA, AND WASIOJA, AND IV-V AT ANNETTE, BIG SUR, CASTROVILLE, COALINGA, FELLOWS, GONZALES, GORMAN, HOLLISTER, LOCKWOOD, LUCIA, MCKITTRICK, MONTEREY, PARKFIELD, PATTIWAY, PORT SAN LUIS, POZO, PRIEST, SALINAS, SANGER, SAN LUCAS, SAN SIMEON, SANTA PAULA, SCHEIDECK, SESPE, SIMMLER, SOLEDAD, AND TEHACHAPI. DATA FROM BSSA V. 17, P. 258 AND V. 20, P. 53.
11/04/1927	14-12--?	34.58	120.67	D			F	SANTA MARIA - AFTERSHOCK.
11/04/1927	14-14--?	34.58	120.67	D			F	SANTA MARIA - AFTERSHOCK.
11/04/1927	15--?--?	34.58	120.67	D			F	SAN LUIS OBISPO - AFTERSHOCK.
11/04/1927	15-42--?	34.58	120.67	D			F	SANTA MARIA - AFTERSHOCK.
1/05/1927	08-17--?	34.58	120.67	D			F	POINT ARGUELLO - AFTERSHOCK; MILD AT SURF.
11/05/1927	09--?--?	34.58	120.67	D			F	POINT ARGUELLO - AFTERSHOCK; REPORTED FROM PASO ROBLES TO HADLEY TOWER.
11/05/1927	11-37--?	34.58	120.67	D			F	POINT ARGUELLO - AFTERSHOCK; REPORTED FROM SURF TO HADLEY TOWER, AND SOUTH OF SAN LUIS OBISPO.
11/06/1927	-?-06--?	34.67	120.17	D			F	IV AT BUELLTON.
11/06/1927	02-25--?	34.67	120.17	D			F	POINT ARGUELLO - AFTERSHOCK; STRONGEST IMMEDIATE AFTERSHOCK AT LOMPOC.
11/06/1927	03-10--?	34.67	120.17	D			F	IV AT BUELLTON.
11/06/1927	22-10--?	34.67	120.17	D			F	OFF POINT CONCEPTION.
11/06/1927	22-50--?	34.67	120.17	D			F	IV AT BUELLTON.
11/06/1927	23-10--?	34.67	120.17	D			F	OFF POINT CONCEPTION.
11/08/1927	10-10--?	34.67	120.17	D			F	IV AT BUELLTON - SHARP BUMPING AT 10-02--?, AROUSED NEARLY ALL. AT LOMPOC MANY AWAKENED BY SHOCK AT 10-15--?.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
11/19/1927	03-32--?	34.92	120.42	D			F	VII AT SANTA MARIA - CENTERED TO NW OF ORIGIN OF NOVEMBER 4 QUAKE -WEAKER, YET NEARLY AS STRONG AT SANTA MARIA,AND VI AT BETTERAVIA AND BICKNELL; REPORTED FROM SAN MIGUEL AND PARKFIELD ON THE NORTH TO SANTA BARBARA CHANNEL ON THE SOUTH.
12/05/1927	11-45--?	34.58	120.67	D			F	IV AT POINT ARGUELLO, AND IV AT BUELLTON WITH 2 SHOCKS 15 SECONDS APART; FELT AT GUADALUPE, SANTA MARGARITA, SANTA MARIA AND SURF.
12/31/1927	10-10--?	34.58	120.67	D			F	V AT POINT ARGUELLO.
03/15/1928	12-03--?	34.92	120.42	D			F	SANTA MARIA.
03/15/1928	12-20--?	34.50	119.67	D			F	SANTA BARBARA.
03/16/1928	14-30--?	34.92	120.42	D			F	SANTA MARIA.
03/29/1928	06-25--?	34.92	120.42	D			F	VII AT SANTA MARIA.
06/09/1928	08-22--?	35.17	119.50	D			F	TAFT.
06/09/1928	08-31--?	35.17	119.50	D			F	TAFT.
06/09/1928	12-25--?	35.17	119.50	D			F	TAFT.
09/03/1928	04-01-54	34.50	122.50	D	5.0	1		OFF POINT ARGUELLO - LICK OBSERVATORY S-P= 39 SECONDS.
11/02/1928	05--?--?	34.67	120.42	D			F	LOMPOC.
05/28/1929	07-10--?	36.17	120.33	D			F	COALINGA.
07/03/1929	09-24--?	34.50	119.67	D			F	SANTA BARBARA.
07/12/1929	13-10--?	36.17	120.33	D			F	COALINGA.
08/28/1929	18-10--?	34.50	119.67	D			F	SANTA BARBARA.
09/09/1929	05-15--?	34.50	119.67	D			F	GAVIOTA, NAPLES, AND SANTA BARBARA.
09/16/1929	03-16--?	35.42	120.92	D			F	CAYUCOS.
09/16/1929	06-15--?	35.42	120.92	D			F	CAYUCOS.
10/05/1929	20-03--?	36.17	120.33	D			F	COALINGA AND LIGHTHIPE.
10/06/1929	21-14--?	36.17	120.33	D			F	COALINGA.
10/07/1929	08--?--?	36.17	120.33	D			F	COALINGA.
10/07/1929	11-30--?	34.83	120.42	D			F	ORCUTT.
10/11/1929	17-55--?	36.17	120.33	D			F	COALINGA.
10/15/1929	22-02--?	36.17	120.67	D			F	COALINGA, KETTLEMEN HILLS, OILFIELDS, AND PRIEST VALLEY,.
11/07/1929	06-30--?	36.33	119.67	D			F	HANFORD.
11/09/1929	02-30--?	36.17	120.33	D			F	BITTER WATER, COALINGA, AND MCKITTRICK.
11/20/1929	22-50--?	36.42	121.00	D			F	BITTER WATER.
11/24/1929	09-54--?	36.42	121.00	D			F	LONOAK, BITTER WATER, AND LEWIS CREEK.
11/26/1929	08-05--?	36.42	121.00	D			F	V AT BITTER WATER AND SAN ARDO; FELT FROM HOLLISTER TO SANTA MARGARITA.
11/26/1929	09--?--?	36.42	120.83	D			F	HERNANDEZ.
11/26/1929	18-06--?	36.42	121.00	D			F	BITTER WATER.
12/05/1929	07-40--?	36.33	119.67	D			F	HANFORD.
03/11/1930	23-59--?	36.42	121.25	D			F	PINNACLES.
06/21/1930	05-15--?	34.83	120.50	D			F	CASMALIA.
08/05/1930	11-25--?	34.42	119.50	D			F	NEAR SANTA BARBARA - FELT OVER AN AREA OF 9000 SQ. MI. V-VI AT CARPINTERIA, GOLETA, OJAI, OXNARD, AND SANTA BARBARA,.
08/08/1930	16.46--?	34.42	119.67	D			F	SANTA BARBARA AND GOLETA.
08/18/1930	13-09--?	34.33	120.58	D			F	OFF POINT CONCEPTION; V OVER A LAND AREA OF 500 SQ. MI. NEAR POINT CONCEPTION.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
08/28/1930	05-15--?	36.42	121.33	D			F	SOLEDAD.
09/02/1930	13-35--?	35.00	121.00	D			F	OFF COAST - FELT AT HALCYON AND SAN LUIS OBISPO.
09/09/1930	05-27--?	34.42	119.50	D			F	SANTA BARBARA.
10/02/1930	14-18--?	34.58	120.67	D			F	OFF POINT ARGUELLO - FELT AT HALCYON.
10/28/1930	13-57--?	35.42	120.92	D			F	OFF COAST NEAR CAYUCOS - FELT AT NIPOMO.
12/08/1930	01-23--?	34.50	119.67	D			F	GOLETA AND SANTA BARBARA.
12/08/1930	01-29--?	34.50	119.67	D			F	GOLETA AND SANTA BARBARA.
02/21/1931	08-10--?	35.67	121.33	D			F	NW OF SAN LUIS OBISPO - FELT AT BRYSON AND PIEDRAS BLANCAS.
02/23/1931	10-01--?	35.83	120.50	D			F	OVER AN AREA OF 5000 SQ. MI.; V AT CAYUCOS, PARKFIELD, AND TEMPLETON.
02/23/1931	10-33--?	35.83	120.50	D			F	SAME AS ABOVE.
04/05/1931	03--?--?	36.17	121.00	D			F	SE OF KING CITY.
07/15/1931	18-40--?	35.00	120.58	D			F	GUADALUPE, NIPOMO, AND SANTA MARGARITA.
07/21/1931	03-25--?	35.25	120.67	D			F	SAN LUIS OBISPO.
07/21/1931	12-08--?	35.25	120.67	D			F	IV AT HALCYON, LOS ALAMOS, NIPOMO, OCEANO, AND TEMPLETON:ALSO FELT AT CAMBRIA, GAVIOTA, PIEDRAS BLANCAS, PORT SAN LUIS, SAN LUIS OBISPO, SANTA MARGARITA, AND SANTA MARIA
09/03/1931	13-50--?	34.50	119.67	D			F	SANTA BARBARA.
09/10/1931	14-35--?	35.50	120.67	D			F	ATASCADERO.
09/30/1931	14-35--?	35.50	120.67	D			F	ATASCADERO.
10/13/1931	12-25--?	36.33	121.67	D			F	JAMESBURG.
10/18/1931	19-58--?	36.33	121.67	D			F	IV AT HOLLISTER, JAMESBURG, AND SPRECKLES; ALSO FELT AT APTOS, CARMEL, CHUALAR, MOSS LANDING, MONTEREY, PARAISO, SALINAS, AND SANTA CRUZ.
12/04/1931	-?-53--?	36.50	121.67	D			F	10 MI. S OF SPRECKELS. FELT AT HOLLISTER, METZ, PIGEON POINT, SPRECKELS, AND SANTA CRUZ.
02/04/1932	16-02-58	34.55	119.73	C	3.0	1	F	SANTA BARBARA AND VENTURA.
02/05/1932	04-14-45	35.83	121.47	C	3.5	1	F	COAST OF MONTEREY COUNTY; FELT AT PIEDRAS BLANCAS LIGHT AND SALMON CREEK.
02/05/1932	06-46-54	35.83	121.47	C	3.5	1	F	COAST OF MONTEREY COUNTY; FELT AT PIEDRAS BLANCAS LIGHT AND SALMON CREEK.
02/05/1932	07-10--?	35.83	121.47	C			F	AFTERSHOCK OF PRECEDING.
02/26/1932	16-58--?	36.00	121.00		5.0		F	IV AT APTOS, ASILOMAR, CARMEL, DEL MONTE, GONZALES, METZ, MONTEREY, PACIFIC GROVE, AND PEBBLE BEACH.
03/13/1932	23-09-24	34.44	120.17	B	3.5	1	F	OFF POINT CONCEPTION; FELT AT BUELLTON.
04/21/1932	03-36-20	35.50	120.67	D	3.0		F	ATASCADERO.
05/06/1932	03-37-08	36.00	120.50	C	3.0	1	F	PARKFIELD.
06/27/1932	05-17-25	36.00	122.00	D	4.0			COAST OF MONTEREY COUNTY.
10/24/1932	04-45--?	35.75	120.75	D			F	PASO ROBLES.
01/30/1933	17--?--?	34.67	120.42	D			F	LOMPOC.
02/26/1933	09-34-32	36.40	121.30	D			F	III AT HOLLISTER, SALINAS, AND SPRECKLES.
04/12/1933	10-03--?	36.33	121.75	D			F	IV AT PORTERVILLE AND VISALIA.
06/26/1933	06-26--?	34.42	120.50	D			F	V AT BUELLTON AND POINT CONCEPTION.
06/26/1933	06-29--?	34.42	120.50	D			F	V AT BUELLTON AND POINT CONCEPTION.
01/09/1934	12-48--?	35.13	120.08	C	3.0			

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
01/12/1934	12-50--?	34.45	120.15	D			F	IV AT LOS ALAMOS.
02/01/1934	16-09--?	34.55	119.53	B	3.5		F	II AT SANTA BARBARA.
02/11/1934	15-16--?	34.55	119.53	C	2.0			
03/20/1934	11-48--?	36.00	120.00	D	3.0			
05/06/1934	20-14--?	35.83	120.75	B	3.5			
05/10/1934	11-28--?	34.50	119.58	C	3.0			
05/19/1934	06-37--?	34.58	120.75	D	3.0			
05/24/1934	06-52--?	34.42	119.75	C	2.5			
05/24/1934	09-04--?	34.42	119.75	C	2.5			
05/24/1934	11-18--?	34.42	119.75	C	2.0			
06/05/1934	09-51--?	35.80	120.33	D			F	COALINGA AND KETTLEMAN HILLS; ALSO FELT AT MONTEREY AND SANTA CRUZ.
06/05/1934	11-30--?	35.80	120.33	D			F	SAN MIGUEL AND SHANDON.
06/05/1934	11-47--?	35.80	120.33	B	3.0			
06/05/1934	13-46--?	35.80	120.33	C	3.0			
06/05/1934	21-30--?	35.80	120.33	D			F	SAN MIGUEL.
06/05/1934	21-48--?	35.80	120.33	B	5.0		F	V AT ADELAIDA, PARKFIELD, AND PRIEST, IV AT ATASCADERO, AVENAL, BIG SUR, BRYSON, CARMEL, HANFORD, KING CITY, LEMOORE, LONOAK, PARAISO, SAN MIGUEL, SANTA CRUZ, SHANDON, AND TEMPLETON, III AT APTOS, BOULDER CREEK, CAMBRIA, CHUALAR, COALINGA, GONZALES, HOLLISTER, MONTEREY, MORRO BAY, PASO ROBLES, SALINAS, SAN FRANCISCO, SAN JOAQUIN VALLEY, SAN LUIS OBISPO, SOLEDAD, SPRECKLES, ETC.; NOT FELT AT ANTIOCH, ETC., BAKERSFIELD, FRESNO, GILROY, LIVERMORE, LOS GATOS, MARICOPA, MERCED, MODESTO, MORGAN HILL, REDWOOD CITY, SAN JOSE, SANTA MARIA, TULARE, OR WATSONVILLE.
06/05/1934	22-52--?	35.80	120.33	C	4.0		F	VI AT ADELAIDA; IV AT ATASCADERO.
06/05/1934	23-30--?	35.80	120.33	D			F	V AT LEMOORE; ALSO FELT AT CASTROVILLE.
06/06/1934	-?-55--?	35.80	120.33	C	3.0			
06/06/1934	16-40--?	35.80	120.33	C	3.5			
06/06/1934	22-40--?	35.80	120.33	C	3.5		F	ADELAIDA, GRAEAGLE, AND PAYNES CREEK.
06/07/1934	22-30--?	35.80	120.33	D			F	STONE CANYON.
06/08/1934	04-15--?	35.80	120.33	D			F	IV AT GONZALES AND MCKITTRICK.
06/08/1934	04-30--?	35.80	120.33	B	5.0		F	VI TO VII AT CHOLOME RANCH, PARKFIELD, AND STONE CANYON DURATION 30 SECONDS, DAMAGE SLIGHT, V AT ATASCADERO, AT ANTELOPE, BIG SUR, CAMBRIA, CASTROVILLE, DELANO, MONTEREY, PASO ROBLES, SAN LUIS OBISPO, SANTA BARBARASANTA MARGARITA, SANTA MARIA, SOLEDAD, TAFT, VENTURA, VISALIA, ETC., AND III OR LESS AT ARVIN, BAKERSFIELD, FRESNO, KERNVILLE, LOMPOC, LOS ANGELES, MENDOTA, PORTERVILLE, SALINAS, SAN BENITO, SANTA ANA, SANTA BARBARA, TULARE, WATSONVILLE, ETC.; NOT FELT AT BIG BASIN, CAJON, COYOTE, GILROY, HUNTINGTON BEACH, INDEPENDENCE, INYOKERN, LANCASTER, MERCED, POMONA, OR SAN JOSE.
06/08/1934	04-37--?	35.60	121.30	D			F	IV AT PIEDRAS BLANCAS, SAN LUIS OBISPO, AND SANTA CRUZ; ALSO FELT AT BRYSON AND LOS ALAMOS.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/08/1934	04-45--?	35.80	120.33	D			F	ATASCADERO, COALINGA, LOCKWOOD, PASO ROBLES, PORT SAN LUIS, PRIEST, SAN MIGUEL, AND WESTHAVEN.
06/08/1934	04-47--?	35.80	120.33	B	6.0		F	WITHIN A RADIUS OF 250 KM FROM THE EPICENTER NEAR THE SOUTHEASTERN ANGLE OF MONTEREY COUNTY; VII TO VIII AT PARKFIELD, VI AT COALINGA, KETTLEMAN CITY, LEMOORE, AND STONE CANYON, V AT ATASCADERO, DUDLEY, HOLLISTER, KING CITY, OILFIELDS, SAN MIGUEL, SEASIDE, SHALE PUMP STATION, AND SHANDON, IV AT ANTELOPE, AVILA, CANOGA PARK, HANFORD, LOS ALAMOS, MARICOPA, MORRO BAY, NIPOMO, PASO ROBLES, PRIEST, SAN LUIS OBISPO, SANTA CRUZ, SANTA MARIA, SOLEDAD, VISALIA ETC., AND III OR LESS AT APTOS, FRESNO, KERNVILLE, LONE PINE, LOS BANOS, MENDOTA, MONTEREY, OAKLAND HARBOR, SALINAS, SAN BENITO, SANTA ANA, TEHACHAPI, TULARE, ETC.
06/08/1934	05--?--?	35.60	121.30	D			F	PIEDRAS BLANCAS LIGHT; ALSO BRYSON, KERNVILLE, LA PANZA, LEMOORE, PARKFIELD, SANDBERG, AND SAN FERNANDO.
06/08/1934	05-20--?	35.80	120.33	D			F	III AT ATASCADERO.
06/08/1934	05-23--?	35.80	120.33	C	3.5		F	ATASCADERO AND SAN MIGUEL.
06/08/1934	05-36--?	35.80	120.33	C	3.0			
06/08/1934	05-42--?	35.80	120.33	B	4.5		F	ATASCADERO, BIG SUR, COALINGA, KING CITY, PASO ROBLES, AND WESTHAVEN.
06/08/1934	05-50--?	35.80	120.33	D			F	IV AT ATASCADERO; ALSO FELT AT COALINGA AND SAN LUIS OBISPO.
06/08/1934	09-30--?	35.80	120.33	B	4.0		F	ATASCADERO AND PARKFIELD.
06/08/1934	15-30--?	35.80	120.33	C	3.5			
06/08/1934	16-30--?	35.80	120.33	D			F	PARKFIELD.
06/08/1934	23-23--?	35.80	120.33	B	4.0		F	NEAR PARKFIELD.
06/10/1934	06-47--?	35.80	120.33	C	3.0			
06/10/1934	08-03--?	35.80	120.33	B	4.5		F	NEAR PARKFIELD; IV AT SAN MIGUEL.
06/10/1934	20-02--?	35.80	120.33	D			F	IV AT SAN MIGUEL; ALSO PARKFIELD AND WOODY.
06/11/1934	03-25--?	35.80	120.33	C	3.0			
06/12/1934	10-47--?	35.80	120.33	C	3.5			
06/14/1934	14-55--?	35.80	120.33	C	4.0		F	IV AT ATASCADERO; ALSO FELT AT SAN MIGUEL AND TEMPLETON.
06/14/1934	15-54--?	35.80	120.33	C	4.0		F	III AT ATASCADERO AND SAN MIGUEL.
06/14/1934	19-26--?	35.80	120.33	C	4.5		F	ATASCADERO AND TEMPLETON.
06/14/1934	22-02--?	35.80	120.33	C	3.5		F	ATASCADERO.
06/15/1934	04-48--?	35.80	120.33	C	3.0			
06/16/1934	23-03--?	36.50	121.00	D	4.0		F	IV AT HOLLISTER AND MONTEREY, AND III AT GONZALES, PARKFIELD, AND SALINAS.
07/02/1934	18-44--?	35.80	120.33	B	3.0			
08/04/1934	-?-18--?	35.80	120.33	B	3.0			
08/21/1934	03-37--?	36.08	120.58	D			F	IV IN STONE CANYON.
08/25/1934	18-52--?	34.42	119.75	C	2.5			
08/26/1934	03-02--?	35.57	119.85	B	3.0			
09/06/1934	23-24--?	36.00	120.55	C	3.0			
09/16/1934	14-38--?	35.83	120.33	C	3.5			
10/07/1934	-?-18--?	34.55	120.78	C	3.5			

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
10/08/1934	04-57--?	34.50	119.58	C	2.0			
10/10/1934	10-52--?	34.55	120.78	C	3.0			
10/19/1934	15-39--?	35.80	120.33	C	3.0			
11/04/1934	22-17--?	34.53	119.67	B	3.0			
11/21/1934	01-02--?	34.58	119.62	B	2.5			
12/01/1934	13-05--?	36.00	121.50	D			F	15 MI. S OF PARAISO; V AT PIEDRAS BLANCAS LIGHT AND IV AT PARAISO.
12/02/1934	16-07--?	35.97	120.58	C	4.0		F	SAN MIGUEL.
12/03/1934	01-54--?	35.95	121.50	C	4.5		F	IV AT BRYSON, KING CITY, AND PARAISO; ALSO FELT AT PARKFIELD, PASO ROBLES, SAN LUCAS, AND SAN MIGUEL.
12/17/1934	11-10--?	34.58	120.33	B	4.5		F	VI AT LOS ALAMOS.
12/17/1934	13-51--?	34.58	120.33	C	2.5		F	LOS ALAMOS.
12/17/1934	15-16--?	34.55	119.67	C	2.5			
12/17/1934	15-35--?	34.58	120.33	C	2.5			
12/18/1934	03-09--?	34.58	120.33	C	4.0		F	LOS ALAMOS.
12/18/1934	04-34--?	34.58	120.33	C	3.0		F	LOS ALAMOS.
12/18/1934	05-28--?	34.58	120.33	C	3.0		F	LOS ALAMOS.
12/19/1934	20-39--?	34.28	119.50	B	2.5			
12/20/1934	12-37--?	34.58	120.33	C	2.5		F	LOS ALAMOS.
12/20/1934	12-39--?	34.58	120.33	C	3.0			
12/20/1934	22-21--?	34.58	120.33	C	3.0			
12/23/1934	16-08--?	34.58	120.33	C	2.5			
12/24/1934	10-22--?	34.58	120.33	B	3.0		F	LOS ALAMOS.
12/24/1934	16-26--?	35.93	120.48	B	5.0		F	IV AT LOS ALAMOS AND SHANDON; ALSO FELT AT KING CITY TEMPLETON.
12/25/1934	04-03--?	34.58	120.33	C	3.0			
01/06/1935	04-04--?	35.98	120.48	C	4.0		F	IV AT PARKFIELD; ALSO FELT AT SHANDON.
01/06/1935	04-25--?	35.90	120.45	D			F	IV AT PARKFIELD.
01/06/1935	04-40--?	35.98	120.48	C	4.0		F	IV AT PARKFIELD AND III AT SHANDON.
01/07/1935	-?-11--?	35.75	119.67	D	3.0			
01/23/1935	03-16--?	34.58	120.33	C	3.5		F	IV AT LOS ALAMOS.
01/27/1935	09-49--?	34.50	119.62	B	2.5			
02/18/1935	04-02--?	35.93	120.48	C	3.5			
02/19/1935	14-17--?	35.93	120.48	D	3.0			
02/28/1935	19-06--?	35.80	120.33	C	3.0			
03/03/1935	11-26--?	36.42	121.75	C	3.0			
03/06/1935	23-14--?	34.43	119.87	C	3.5		F	III AT SANTA BARBARA.
03/19/1935	03-59--?	34.55	120.78	B	4.0			OFF POINT ARGUELLO.
04/05/1935	10-13--?	35.93	120.48	C	3.5			
05/05/1935	12-58--?	34.58	119.68	C	2.5			
05/18/1935	04-36--?	34.58	120.33	B	3.5		F	IV AT LOS ALAMOS.
05/19/1935	03-44--?	34.58	120.33	C	3.0			
05/20/1935	23-44--?	34.58	120.33	C	3.0			
05/27/1935	16-08--?	35.37	120.97	C	3.0		F	III AT TEMPLETON.
06/10/1935	02-02--?	35.33	119.83	C	3.5			
06/18/1935	08-52--?	34.60	119.60	C	2.0			
06/23/1935	23-53--?	34.55	119.68	C	3.0			

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/30/1935	23-28--?	36.00	121.00	D	4.0		F	SE OF SALINAS; III AT HOLLISTER.
07/25/1935	04-16--?	35.80	120.33	C	3.0		F	V AT PARKFIELD.
07/28/1935	06--?--?	35.70	121.12	B	4.0		F	SAN SIMEON.
08/06/1935	19-05--?	34.62	119.62	C	3.0		F	SANTA BARBARA.
08/07/1935	22-30--?	34.55	120.78	C	3.5			
08/09/1935	17-14--?	36.17	120.98	C	3.5		F	PRIEST VALLEY.
08/31/1935	09-28--?	34.50	119.70	C	2.5			
10/18/1935	09-24--?	35.80	120.70	D	3.5		F	IV AT PARKFIELD - AFTERSHOCK.
10/22/1935	18-37--?	35.93	120.48	C	4.0		F	PARKFIELD.
10/25/1935	19-43--?	36.40	121.55	D			F	13 MI. W OF SOLEDAD; IV AT SAN BENITO.
10/26/1935	10-46--?	35.85	121.40	D			F	AFTERSHOCK.
12/22/1935	06-54--?	34.55	120.78	C	3.0			
02/03/1936	09-12--?	34.75	119.75	C	2.5			
02/21/1936	23-06--?	34.42	119.67	C	3.0			
02/22/1936	-?-18--?	34.42	119.67	C	3.0			
02/22/1936	-?-21--?	34.42	119.67	C	2.5			
02/22/1936	-?-23--?	34.42	119.67	C	3.0			
02/22/1936	04-55--?	34.42	119.67	C	3.0			
03/06/1936	03-45--?	35.90	120.40	D	3.0			
03/17/1936	01-55--?	36.50	120.92	C	4.0		F	IV AT CHUALAR, HOLLISTER, AND TRES PINOS.
03/18/1936	09-07--?	35.93	120.48	C	2.5			
03/27/1936	-?-58--?	34.55	120.78	C	3.0			
03/29/1936	09-26--?	34.50	119.62	C	2.5			
05/20/1936	17-22--?	35.93	120.48	C	3.0			
05/23/1936	04-41--?	36.17	120.92	C	4.0		F	IV AT KING CITY.
05/27/1936	19-55--?	36.50	121.17	C	4.5			SAN BENITO COUNTY.
06/24/1936	12-23--?	35.12	120.08	C	3.0		F	SAN LUIS OBISPO CO.; IV AT LOS ALAMOS.
07/13/1936	18-09--?	34.50	119.60	D	2.5			
07/22/1936	04-03--?	34.50	119.80	C	2.5			
07/30/1936	09-36--?	34.57	119.63	C	3.0			
09/07/1936	16-47--?	34.37	120.38	C	3.0			
09/09/1936	04-54--?	34.37	120.38	C	4.0		F	LOS ALAMOS.
09/10/1936	21-21--?	34.40	120.40	D	3.0			
09/12/1936	13-56--?	34.75	120.33	C	3.5			
09/15/1936	-?-09--?	34.50	120.50	D	2.5			
10/16/1936	15-30--?	34.83	120.58	C	4.0			NEAR CASMALIA.
10/16/1936	15-36--?	34.83	120.58	C	3.0			
10/17/1936	01-17--?	34.83	120.58	C	3.0			
10/19/1936	14-01--?	34.83	120.58	C	3.0			
11/01/1936	15-10--?	34.55	120.78	B	4.0			OFF POINT ARGUELLO.
11/02/1936	01-29--?	34.55	120.78	C	3.0			
11/05/1936	14-30--?	35.85	121.40	D			F	HOLLISTER.
11/08/1936	16-51--?	34.55	120.78	C	3.0			
11/08/1936	22-43--?	34.55	120.78	C	3.0			
11/18/1936	17-15--?	35.35	120.60	D			F	POZO, SAN LUIS OBISPO, AND SANTA MARGARITA.
11/18/1936	18-02--?	34.70	120.25	C	4.5		F	IV AT ARROYO GRANDE, ATASCADERO, BETTERAVIA, LOS ALAMOS OCEANO, POZO, SAN LUIS OBISPO, AND SANTA MARGARITA.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
11/22/1936	02-16--?	34.58	120.78	C	3.5			
11/25/1936	21-51--?	34.58	120.78	C	3.0			
12/23/1936	17-16--?	35.93	120.48	B	3.5			
12/26/1936	01-12--?	34.55	119.68	C	2.5			
01/12/1937	15-44--?	34.50	120.80	D	3.0			
01/28/1937	17-36--?	34.43	119.87	C	2.5			
02/16/1937	17-40--?	34.55	120.78	C	4.0			OFF POINT ARGUELLO.
02/17/1937	03-33--?	36.50	121.58	C	4.5		F	9 MI. SE OF PAICINE; FELT AT ANTELOPE, HOLLISTER, AND PANOCHE.
02/20/1937	09-58--?	35.93	120.48	C	4.0		F	PARKFIELD AND PASO ROBLES.
02/22/1937	18-10--?	36.17	121.53	C	4.0		F	KING CITY.
02/24/1937	13-37--?	34.50	119.70	C	2.0			
02/25/1937	03-20--?	34.50	119.70	C	2.0			
03/26/1937	21-35--?	34.60	119.70	C	3.5			
03/31/1937	17-43--?	34.50	119.70	C	3.0			
04/17/1937	08-30--?	34.60	119.70	C	2.5			
04/30/1937	08-16--?	34.50	119.70	D	2.5			
05/31/1937	15-33--?	36.50	120.70	C	3.0			
06/02/1937	09-32--?	34.40	119.70	C	2.5			
07/31/1937	14-18--?	34.22	119.55	C	3.0			
07/31/1937	15-14--?	34.22	119.55	C	2.5			
08/15/1937	19-01--?	36.50	120.70	D	3.0			
08/22/1937	01-56--?	35.00	121.00	D	3.5			
09/16/1937	02-48--?	35.93	120.48	B	3.5		F	NEAR PARKFIELD; FELT AT BRADLEY.
09/18/1937	13-29--?	36.50	121.50	D	4.0		F	9 MI. SE OF PAICINES; FELT AT CHUALAR, SALINAS, AND SPRECKLES.
09/22/1937	02-41--?	34.50	119.70	C	3.0			
09/29/1937	22-39--?	34.50	119.70	C	3.0			
10/13/1937	08-32--?	34.40	119.70	C	2.5			
11/01/1937	21-40--?	36.50	121.40	D			F	6 MI. N OF GONZALES.
11/03/1937	10--?--?	36.15	121.00	D			F	V AT SAN LUCAS; FELT ALSO AT KING CITY AND SAN ARDO.
11/22/1937	04-12--?	34.55	120.78	C	4.5		F	OFF POINT ARGUELLO; V AT BUELLTON, GOLETA, PISMO BEACH, POINT D SANTA MARIA, AND IV AT ARLIGHT, BETTERAVIA, BICKNELL, E, GAVIOTA, GUADALUPE, LOMPOC, LOS ALAMOS, LOS OLIVOS, SANTA URF.
11/22/1937	04-51--?	34.55	120.78	C	3.5			
11/28/1937	09-55--?	34.55	120.78	C	3.5			
12/03/1937	15-28--?	34.55	120.78	C	4.0		F	OFF POINT ARGUELLO; FELT AT GAVIOTA AND POINT CONCEPTION.
12/03/1937	21-13--?	34.55	120.78	C	3.5			
12/05/1937	01-36--?	36.00	121.00	D	3.5		F	19 MI. S OF LOS BANOS; V AT LOS BANOS.
12/05/1937	01-37--?	36.00	121.00	D	4.0			SAN BENITO COUNTY.
12/05/1937	02-05--?	36.00	121.00	D	3.0		F	19 MI. S OF LOS BANOS.
12/24/1937	11-57--?	34.50	120.80	D	4.0		F	OFF POINT ARGUELLO. FELT AT CASMALIA, LOS ALAMOS, POINT CONCEPTION.
12/25/1937	13-01--?	36.00	120.00	D	3.0			
01/01/1938	01-59--?	34.55	120.78	C	3.5			

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
01/18/1938	04-35--?	34.55	120.78	B	3.5			
01/24/1938	04-38--?	34.55	120.78	C	3.5			
01/25/1938	12-24--?	34.55	120.78	C	3.5			
02/01/1938	18-14--?	34.55	120.78	C	3.5			
02/20/1938	14--?--?	34.55	120.78	C	3.5			
02/21/1938	10-59--?	35.93	120.78	C	3.0			
03/04/1938	15-14--?	34.30	119.57	C	2.5			
03/04/1938	18-25--?	34.30	119.57	C	2.5			
04/12/1938	01-50--?	34.55	120.78	C	3.5			
05/10/1938	10-32--?	36.20	121.30	D	4.5		F	BIG SUR, HOLLISTER, KING CITY, PINNACLES, SALINAS, SOLEDAD, SOQUEL, AND TRES PINOS-6 SHOCKS FELT AT PINNACLES.
05/10/1938	10-41--?	36.20	121.30	D	4.0		F	SAN BENITO.
05/13/1938	19-34--?	36.20	121.30	D	4.0			MONTEREY COUNTY.
05/27/1938	22-03--?	36.20	120.00	D	3.5			
06/01/1938	05-17--?	34.55	119.68	D			F	SANTA BARBARA.
06/01/1938	06-17--?	34.55	119.68	D	3.0			
06/06/1938	02-55--?	34.50	119.67	C	3.0			
09/16/1938	06-11--?	36.40	121.20	D	4.0		F	PINNACLES.
09/27/1938	10-21--?	34.50	119.70	C	2.5			
09/27/1938	12-23--?	36.30	120.90	C	5.0		F	OVER AN AREA OF 9000 SQ. MI. OF WEST-CENTRAL CALIFORNIA, ALONG THE COAST AS FAR NORTH AS PESCADERO AND SOUTH TO SAN LUIS OBISPO. INLAND IT WAS FELT AT COALINGA, MENDOTA, AND STEVENSON, WITH A V AT BIG SUR, BRYSON, CHUALAR, GONZALES, GREENFIELD, HARMONY, HOLLISTER, JOLON, LOCKWOOD, PAICINES, PARAISO, PINNACLES, SAN ARDO, SAN BENITO, SAN LUCAS, SOLEDAD, AND SPRECKLES, AND IV AT BEN LOMOND, CAMBRIA, CARMEL, CASTROVILLE, DOS PALOS, GILROY, KING CITY, LOS BANOS, MENDOTA, MONTEREY, PASO ROBLES, PRIEST, SALINAS, SAN LUIS OBISPO, TRES PINOS, WATSONVILLE, ETC.
09/27/1938	16-20--?	36.45	121.25	D			F	PAICINES AND PINNACLES.
09/29/1938	12-12--?	34.55	120.78	C	4.0			OFF POINT ARGUELLO.
10/02/1938	18-45--?	34.33	119.58	C	4.0		F	SANTA BARBARA AND SUMMERLAND.
10/24/1938	13-40--?	36.45	121.25	D			F	HOLLISTER AND PINNACLES.
10/28/1938	10-07--?	35.80	120.33	C	3.5			
11/01/1938	22-46--?	35.12	120.08	C	3.0			
11/16/1938	13-39--?	35.80	120.33	C	3.0			
11/22/1938	15-30--?	35.93	120.48	B	4.5		F	NEAR PARKFIELD; FELT AT ATASCADERO, CAMBRIA, CRESTON, MORRO BAY, PARKFIELD, PASO ROBLES, SAN MIGUEL, AND SHANDON.
01/01/1939	-?-53--?	34.58	120.33	C	3.0			
01/21/1939	07-08--?	36.45	121.25	D			F	PINNACLES.
01/22/1939	15-52--?	34.40	119.70	C	2.5			
02/05/1939	03-30--?	35.65	120.65	D			F	PASO ROBLES.
02/09/1939	06-44--?	35.93	120.48	C	3.0		F	NEAR PARKFIELD.
02/12/1939	03-12--?	34.42	119.83	B	3.0		F	GOLETA AND SANTA BARBARA .
03/24/1939	02-49--?	34.55	120.78	C	3.5			

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/25/1939	03-45--?	36.45	121.25	D			F	PINNACLES.
03/30/1939	10-11--?	34.50	119.80	C	2.5			
05/02/1939	18-49--?	35.93	120.48	C	4.0		F	IV AT PARKFIELD.
05/03/1939	07-55--?	34.55	120.78	C	3.0			
05/03/1939	12-39--?	35.65	120.65	D			F	PASO ROBLES.
05/18/1939	23--?--?	35.80	120.33	C	3.0			
06/15/1939	21-12--?	34.50	119.70	C	2.5			
06/17/1939	04-30--?	34.75	120.25	D			F	LOS ALAMOS. REPORTS OF SEVERAL SHOCKS.
06/24/1939	12-55--?	35.85	120.85	D			F	BRADLEY.
06/24/1939	13-02--?	36.40	121.00	C	5.5		F	OVER AN AREA OF 10,000 SQ. MI. IN WEST-CENTRAL CALIFORNIA, ALONG THE COAST AS FAR NORTH AS HALF MOON BAY AND SOUTH TO ESTERO BAY. INLAND IT WAS FELT AT COALINGA, TRANQUILITY, AND VOLTA, WITH A VII AT HOLLISTER, VI AT KING CITY AND PAICINES, V AT CAYUCOS, SOLEDAD, AND SPRECKLES, AND IV AT PAICINES, V AT CAYUCOS, SOLEDAD, AND SPRECKLES, AND IV AT CAMBRIA, CARMEL, CASTROVILLE, CHUALAR, GILROY, GONZALES, LOCKWOOD, MILPITAS, MONTEREY, NIPOMO, PASO ROBLES, PINNACLES, SALINAS, SAN ARDO, SAN BENITO, SAN JUAN, SAN MIGUEL, SAN SIMEON, SANTA CRUZ, TRES PINOS, AND WATSONVILLE.
07/04/1939	10-49--?	36.40	121.00	C	4.0		F	HOLLISTER, PAICINES, AND SALINAS.
07/10/1939	18-33--?	36.40	121.25	D			F	PINNACLES.
07/24/1939	09-30--?	36.25	121.80	D			F	BIG SUR.
07/24/1939	13--?--?	36.00	121.15	D			F	JOLON.
09/06/1939	01-53-43	34.58	120.42	C	3.0			
09/07/1939	02-50-30	35.42	121.08	C	3.0		F	OFF SAN LUIS OBISPO CO.; FELT AT CAMBRIA.
09/08/1939	01-57--?	34.75	120.25	D			F	LOS ALAMOS.
09/08/1939	05--?--?	34.75	120.25	D			F	LOS ALAMOS.
09/12/1939	-?--?-47	34.25	119.75	C	3.0			
09/24/1939	11-57-40	36.40	121.00	D	3.5			
10/06/1939	04-39--?	35.80	121.50	D	3.5			
10/17/1939	19-21-41	34.55	120.78	C	3.5			
10/17/1939	20-42-43	34.55	120.78	C	4.0			OFF POINT ARGUELLO.
11/02/1939	14-02--?	34.40	120.50	D			F	POINT CONCEPTION LIGHT STATION.
11/04/1939	14-11-33	36.20	120.90	D	3.0		F	SALINAS AND SAN LUCAS.
12/14/1939	03-45-18	36.10	120.00	D	3.0			
12/25/1939	15-36-23	34.28	119.83	C	3.5			
12/28/1939	12-15-38	35.80	120.33	B	5.0		F	OVER AN AREA OF 15,000 SQ. MI. IN WEST-CENTRAL CALIFORNIA, ON THE COAST FROM SANTA CRUZ SOUTH TO POINT ARGUELLO, AND INLAND TO LOST HILLS AND FRESNO. V AT COALINGA, FRESNO, GREENFIELD, PRIEST, SAN ARDO, AND SAN LUCAS, AND IV AT APTOS, ATASCADERO, BIG SUR, CAMBRIA, CARMEL, CASTROVILLE, CAYUCOS, CHUALAR, GONZALES, HOLLISTER, KING CITY, MENDOTA, MONTEREY, MORRO BAY, PARKFIELD, PASO ROBLES, PINNACLES, SALINAS, SAN JUAN BAUTISTA, SAN LUIS OBISPO, SANTA CRUZ, SOLEDAD, TAFT, ETC.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/29/1939	04--?--?	36.40	121.25	D			F	PINNACLES.
12/30/1939	15-24-37	35.80	120.33	D	3.5		F	NEAR PARKFIELD. FELT AT SAN LUCAS.
02/27/1940	11-40-25	34.25	119.50	B	3.0			
05/21/1940	10-05-34	35.28	120.48	B	4.0		F	ATASCADERO, CAMBRIA, CAYUCOS, MORRO BAY, PASO ROBLES, PISMO BEACH, AND SAN LUIS OBISPO.
06/16/1940	09-25-04	34.55	120.78	C	4.0		F	OFF POINT ARGUELLO; FELT AT GUADALUPE AND LOS ALAMOS.
06/26/1940	08-56--?	36.08	120.32	C	3.5			
06/28/1940	04-06-42	34.55	120.78	C	3.0			
08/13/1940	22-07-29	36.23	120.32	B	4.0			(DEPT. OF WATER RESOURCES DATA.)
08/31/1940	08-52-46	34.55	120.78	B	3.5			
09/07/1940	10-36-30	36.50	121.50	D	3.5			
09/07/1940	10-38-36	36.50	121.50	D	3.5			
09/07/1940	13-02-06	36.50	121.50	D	4.5		F	CARMEL AND SALINAS.
10/20/1940	22-18-45	34.55	120.78	C	3.0			
11/10/1940	10-25-10	34.35	119.77	C	4.0		F	SANTA BARBARA CHANNEL; FELT AT GOLETA, PARADISE CAMP, AND SANTA BARBARA.
11/17/1940	21-23-43	35.00	119.50	C	3.0			
01/29/1941	08-54-01	34.48	119.53	B	3.0			
02/04/1941	03-19-12	34.55	119.68	C	3.0			
02/04/1941	03-42-09	34.55	119.68	C	3.0			
02/08/1941	15-58-50	34.55	119.68	C	3.5		F	SANTA BARBARA.
02/09/1941	23-49-18	34.50	119.70	C	2.0			
02/11/1941	06-43-30	34.27	119.57	B	3.5		F	SANTA BARBARA.
02/12/1941	20-10-24	34.40	119.70	C	3.0			
02/14/1941	22-19-06	34.40	119.70	C	2.5			
05/07/1941	16-17-34	34.55	120.78	C	3.5			
05/15/1941	03-29--?	36.15	120.35	D			F	COALINGA.
05/15/1941	06--?--?	36.15	120.35	D			F	COALINGA.
07/01/1941	07-50-57	34.33	119.58	A	6.0		F	SANTA BARBARA; FELT OVER AN AREA OF 20,000 SQ. MI. VIII AT CARPINTERIA AND SANTA BARBARA, VII AT GOLETA AND VENTURA, VI AT FILLMORE, KEYSTONE, LOS ALAMOS, OJAI, OXNARD, PORT HUENEME, SANTA PAULA, SUMMERLAND, AND WHEELER SPRINGS, AND V AT ACTON, ALTADENA, ARLIGHT, ARTESIA, ARVIN, BETTERAVIA, BUELLTON, BURBANK, CAMARILLO, CANOGA PARK, CASMALIA, CAYUCOS, CHATSWORTH, COMPTON, EL SEGUNDO, GAVIOTA, GLENDALE, HERMOSA BEACH, INGLEWOOD, LA CRESCENTA, LAGUNA BEACH, LANCASTER, LOMITA, LOMPOC, LONG BEACH, LOS ANGELES, LOS OLIVOS, MAYWOOD, MCKITTRICK, MONTALVO, MOORPARK, NEWBURY PARK, NEWPORT, NIPOMO, NORTH HOLLYWOOD, OCEANO, ORCUTT, PASADENA, PATTIWAY, IRU, POINT CONCEPTION, SANDBERG, SAN NICHOLAS ISLAND, SAN PEDRO, SANTA ANA, SANTA MARIA, SANTA MONICA, SANTA YNEZ, SIERRA MADRE, SIMI, STANTON, SUNLAND, SURF, TEHACHAPI, UPPER SESPE MOUNTAINS, VALYERMO, WHEELER RIDGE, AND WHITTIER.
07/01/1941	07-57--?	34.33	119.58	B	3.0			
07/01/1941	07-58--?	34.33	119.58	B	3.5			

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/01/1941	08-05--?	34.33	119.58	B	3.0			
07/01/1941	08-07--?	34.33	119.58	B	3.0			
07/01/1941	08-10--?	34.33	119.58	B	3.0			
07/01/1941	08-13--?	34.33	119.58	B	3.0			
07/01/1941	08-15--?	34.33	119.58	B	3.0			
07/01/1941	08-19--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57 (THIS DATE).
07/01/1941	08-21--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	08-25--?	34.33	119.58	B	3.5			
07/01/1941	08-30--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/01/1941	08-48--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	08-58--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	09-05--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	09-45--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	10-25--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	12-37--?	34.33	119.58	B	3.0			
07/01/1941	14-22--?	34.33	119.58	B	3.0			
07/01/1941	18-13--?	34.33	119.58	B	3.0			
07/01/1941	18-20--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/01/1941	19-48--?	34.33	119.58	B	3.0			
07/01/1941	20-15--?	34.33	119.58	B	3.5			
07/01/1941	22-51--?	34.33	119.58	B	3.5			
07/01/1941	23-54--?	34.33	119.58	B	4.5		F	AFTERSHOCK OF 07-50-57; FELT AT FILLMORE, GAVIOTA, LOS ALAMOS, AND SANTA BARBARA.
07/02/1941	-?-17--?	34.33	119.58	B	3.0			
07/02/1941	04-33--?	34.33	119.58	B	3.5			
07/02/1941	08-45--?	34.33	119.58	B	3.5			
07/02/1941	11-41--?	34.33	119.58	B	3.0			
07/02/1941	22-19--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/03/1941	-?-25--?	34.33	119.58	B	3.5			
07/03/1941	19-26--?	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07-50-57.
07/07/1941	01-06--?	34.33	119.58	B	3.0			
07/07/1941	06-25--?	34.33	119.58	B	3.5			
07/08/1941	19-37--?	34.33	119.58	B	3.0			
07/12/1941	16-18--?	34.33	119.58	B	4.5		F	AFTERSHOCK OF 07-50-57; FELT AT FILLMORE, GLENDALE, MONTROSE, SATICOY, SAUGUS, AND WHEELER SPRINGS.
07/12/1941	16-41--?	34.33	119.58	B	3.0			
07/12/1941	21-07--?	34.33	119.58	B	3.0			
07/12/1941	21-12--?	34.33	119.58	B	3.0			
07/13/1941	06-11--?	34.33	119.58	B	3.5			
07/16/1941	23-10--?	34.33	119.58	B	3.0			
07/17/1941	18-31--?	34.33	119.58	B	3.0			
07/27/1941	12-44--?	34.33	119.58	B	3.0			
07/31/1941	13-23--?	34.33	119.58	B	3.0			
08/02/1941	12-31-19	34.33	119.58	C	3.0			
08/09/1941	05-05-24	34.33	119.58	C	3.5			

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
08/12/1941	22-35-24	34.33	119.58	C	3.5			
08/19/1941	10-20-25	34.33	119.58	C	3.0			
08/25/1941	06-58-22	34.33	119.58	C	3.0			
08/27/1941	17-11-02	34.33	119.58	C	3.0			
08/29/1941	08-43-24	34.60	120.30	C	3.0			
09/08/1941	03-12-45	34.33	119.58	B	4.5		F	AFTERSHOCK OF 07/01/41, 07-50-57. V AT GOLETA AND SANTA BARBARA; FELT STRONGLY AT LOS ALAMOS AND SUMMERLAND.
09/08/1941	03-14-23	34.33	119.58	B	4.0		F	TWIN SHOCK OF 03-12-45; SAME "FELT" REPORT.
09/08/1941	04-45-16	34.33	119.58	B	3.5		F	SANTA BARBARA.
09/09/1941	03-23-17	34.33	119.58	B	3.5		F	SANTA BARBARA.
09/09/1941	13-44-46	34.33	119.58	B	3.0			
09/14/1941	01-45-18	34.33	119.58	B	4.0		F	AFTERSHOCK OF 07/01/41, 07-50-57.
09/14/1941	02-20-42	34.33	119.58	B	3.0			
09/15/1941	01-37-02	34.33	119.58	B	4.0		F	GOLETA, SANTA BARBARA, AND SUMMERLAND.
09/15/1941	01-55-18	34.33	119.58	B	3.0			
09/15/1941	02-49-06	34.33	119.58	B	3.5			
09/16/1941	07-27--?	34.33	119.58	B	3.5			
09/25/1941	05-12-56	34.33	119.58	B	4.0		F	GOLETA AND SANTA BARBARA.
10/07/1941	12-05-42	34.33	119.58		3.0			
10/19/1941	23-22-19	34.33	119.58	B	3.0			
11/05/1941	16-36--?	35.00	121.00	D	3.5		F	OFF POINT CONCEPTION; FELT AT SAN SIMEON.
11/17/1941	17-30-27	34.33	119.58	C	3.0			
11/18/1941	18-08-10	34.33	119.58	C	4.0		F	CARPINTERIA AND SANTA BARBARA.
11/21/1941	16-56-03	34.33	119.58	C	4.0		F	GOLETA AND SANTA BARBARA.
11/25/1941	20-01-48	34.33	119.58	C	3.0			
11/28/1941	06-33--?	35.00	120.00	D	3.5			
12/08/1941	-?-29-42	36.00	121.00	D	3.5			
12/22/1941	-?-54-09	35.93	120.48	C	4.0		F	NEAR PARKFIELD-NOT RECORDED ON BERKELEY NETWORK.
01/06/1942	09-20--?	36.15	120.65	D			F	PRIEST VALLEY-RECORDED AT TINEMAHA.
01/06/1942	09-23--?	36.15	120.65	D			F	PRIEST VALLEY-RECORDED AT TINEMAHA.
01/08/1942	18-21-05	34.13	119.58	C	2.5			
01/18/1942	11-35--?	36.40	121.25	D			F	PINNACLES.
01/18/1942	16-50--?	36.40	121.25	D			F	PINNACLES.
02/19/1942	18-33--?	36.40	121.25	D			F	PINNACLES.
03/09/1942	05-57-42	34.30	119.60		3.0			
03/25/1942	-?--?--?	36.40	121.25	D			F	PINNACLES; LIGHT SHOCK.
04/19/1942	04-02-47	34.30	119.60	D	3.0			
04/22/1942	05-32-52	35.30	119.50	D	3.0			
05/08/1942	17-19-13	34.33	119.58	C	3.0			
06/06/1942	06-42-11	34.35	119.85	C	3.0		F	GOLETA.
06/29/1942	21-07-30	35.60	120.80	D	4.0		F	IV AT CAMBRIA AND SAN LUIS OBISPO.
07/19/1942	10-42-07	36.40	121.10	D	1.6			SW OF LLANADA.
09/15/1942	10-36-33	36.13	122.18	B	3.0			SW OF KING CITY.
10/04/1942	10--?--?	34.60	120.00	D			F	IV AT SANTA YNEZ PEAK.
10/11/1942	23-48-23	36.48	121.40	C	1.9			FORESHOCK OF QUAKE ON OCTOBER 15 AT 13-53-56.
10/15/1942	13-53-56	36.48	121.40	B	4.3		F	IV AT BIG SUR, GONZALES, GREENFIELD, HOLLISTER, SALINAS, AND SOLEDAD.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
10/18/1942	08--?--?	36.00	121.00	D			F	CAMBRIA.
10/18/1942	12-01-42	36.00	121.00	D			F	V AT CAMBRIA.
10/19/1942	10-23--?	34.50	119.65	D			F	V AT SANTA BARBARA.
10/20/1942	10-25--?	36.00	121.00	D			F	V AT CAMBRIA.
10/26/1942	01-09-01	36.40	121.60	D	1.8			DEPTH ABOUT 12 KM.
12/02/1942	11-46--?	34.33	119.58	C	3.5		F	V AT SANTA BARBARA.
12/06/1942	16-57-49	35.93	120.48	C	3.5			
01/24/1943	06-55-57	34.33	119.58	C	3.0			
03/16/1943	09-27-47	34.28	119.60	C	3.0			
04/01/1943	13-39-66	34.68	121.75	B	3.1			OFF COAST, WEST OF POINT ARGUELLO.
06/29/1943	02-50-53	36.50	121.10	D	3.1			SW OF LLANADA.
07/05/1943	16-30-29	36.38	121.83	C	3.9			SOUTH OF SALINAS.
07/15/1943	-?-44-42	36.00	120.15	D			F	NEAR AVENAL.
08/07/1943	16-59-47	34.28	119.57	C	3.5			
08/12/1943	15-56-33	34.75	121.15	C	3.5			
08/27/1943	08-16-53	34.43	119.87	C	3.5		F	IV AT SANTA BARBARA.
09/13/1943	12-40--?	35.65	120.65	D			F	PASO ROBLES, POSSIBLY GUN FIRE.
09/18/1943	17-07-16	34.37	119.58	C	3.0			
10/22/1943	12--?--?	36.00	120.90	D			F	SAN ARDO; 2 SHOCKS.
10/26/1943	22-10--?	34.75	120.25	D			F	LOS ALAMOS.
10/31/1943	17-54-06	35.80	120.40	D	3.5			
10/31/1943	20--?--?	36.40	121.00	D			F	LONOAK.
11/08/1943	11-33-46	36.00	119.92	C	3.0		F	KETTLEMAN HILLS; FELT AT AVENAL.
11/30/1943	21-57-18	36.30	120.50	D	4.0			NEAR COALINGA.
12/01/1943	04-51--?	36.50	121.10	D			F	SAN BENITO.
01/04/1944	18-06-40	34.10	120.40	D	3.3			
02/18/1944	16-29-37	34.10	119.52	C	2.1			
02/21/1944	13--?-11	36.17	120.93	C	3.8			WEST OF PRIEST.
03/06/1944	21-32-16	36.40	121.25	C	3.4			NE OF PARAISO.
04/03/1944	02-33--?	34.50	121.40	D	4.0			OFF POINT ARGUELLO.
04/12/1944	15-33-10	34.27	119.52	C	4.0		F	OFF CARPINTERIA; FELT EAST OF SANTA BARBARA.
06/13/1944	08-27-32	34.67	120.50	C	4.6		F	NEAR LOMPOC; VI AT LOS ALAMOS AND IV AT SANTA MARIA.
06/13/1944	08-46-43	34.67	120.50	C	4.0		F	AFTERSHOCK OF 08-27-32.
06/13/1944	11-07-24	34.67	120.50	C	4.4		F	AFTERSHOCK OF 08-27-32.
07/11/1944	22-33--?	36.50	121.10	D			F	SAN BENITO.
07/15/1944	19-22-37	34.37	119.62	C	3.1			
09/04/1944	02-47-46	35.00	120.00	D	3.4		F	LOS ALAMOS.
09/04/1944	05--?--?	35.00	120.00	D			F	LOS ALAMOS.
09/15/1944	14-12-42	34.70	120.20	D	2.6		F	KETTLEMAN HILLS REGION; FELT AT PARKFIELD.
09/18/1944	01-30--?	35.00	120.00	D	3.5			
11/04/1944	08-12-01	36.33	120.08	C	3.4			
11/08/1944	16-12-36	34.33	119.72	C	3.1			
11/28/1944	10-36--?	35.80	120.00	D	3.3			
11/30/1944	18-53-15	34.72	120.42	C	4.1		F	NEAR LOS ALAMOS; FELT AT LOS ALAMOS AND LOS OLIVOS.
12/02/1944	15-09-12	35.80	120.00	D	3.2			
01/27/1945	17-50-31	34.75	120.67	C	3.9			
02/25/1945	20-18-38	36.00	120.48	C	3.6			

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
04/15/1945	22-59-57	34.13	119.83	C	3.1			
06/11/1945	03-54-52	34.50	120.80	D	3.2			
07/11/1945	16-13--?	35.67	121.25	D	4.0		F	NEAR SAN SIMEON; IV AT CAMBRIA.
07/28/1945	02-33-48	34.70	120.10	D	4.2		F	EAST OF SANTA MARIA; IV AT LOS ALAMOS.
09/04/1945	12-38-31	34.32	119.63	C	3.2			
09/07/1945	11-34-20	35.83	120.75	C	4.2		F	NEAR BRADLEY; IV AT CAMBRIA, PARKFIELD, PASO ROBLES, AND SAN MIGUEL.
11/04/1945	-?-46-34	36.38	121.28	C	3.3			NEAR SOLEDAD.
02/09/1946	02-55-28	34.33	119.92	C	2.5			
02/10/1946	11-01-19	36.50	121.00	D	4.2		F	OVER AN AREA OF 2000 SQ. MI. IN WEST CENTRAL CALIFORNIA. V AT SAN BENITO, AND IV AT BIG SUR, CHUALAR, GREENFIELD, HOLLISTER, LONOAK, SAN LUCAS, SAN MIGUEL, SANTA CRUZ, AND SOLEDAD.
02/15/1946	12-07-00	35.90	121.45	D			F	PARKFIELD; LIGHT SHOCK.
04/19/1946	12-50--?	34.00	120.40	D			F	SANTA MARIA.
07/08/1946	19-59-44	34.83	120.53	C	3.2			
08/06/1946	04-55-07	34.95	120.18	C	2.8		F	E OF SANTA MARIA; FELT AT LOS ALAMOS.
09/02/1946	10-09-47	34.18	119.62	C	3.0			
09/09/1946	11-20--?	34.90	120.40	D			F	SANTA MARIA.
09/19/1946	06-35-44	35.83	119.67	C	3.2			
10/24/1946	18-26-50	34.37	119.62	C	2.7			
11/22/1946	09-47-59	34.83	120.68	D	3.0			
11/27/1946	14-44-51	35.50	120.92	C	4.3		F	NEAR CAYUCOS; V AT MORRO BAY AND SANTA MARGARITA; ALSO FELT ATASCADERO, LOS ALAMOS, PISMO BEACH, AND SAN LUIS OBISPO.
12/13/1946	-?-40-01	34.17	119.53	C	3.5			
01/06/1947	21-05-47	35.85	120.47	C	3.6			
01/13/1947	19-38-31	34.32	119.65	C	2.2			
01/14/1947	20-49-27	34.23	119.65	C	2.7			
01/18/1947	12--?-42	34.20	121.50	D	3.3			
01/19/1947	19-32--?	35.60	120.30	D	3.1		F	PASO ROBLES.
02/05/1947	06-14--?	38.23	120.65	B	5.0		F	VI AT LONOAK, V AT COALINGA, IDRIA, AND KING CITY, AND IV AT BIG SUR, HURON, PARKFIELD, SAN ARDO, AND WESTHAVEN.NEAR COALINGA - AFTERSHOCK OF 2/5/47 OF 06-14--?.
02/25/1947	11-45-18	36.20	120.50	D	4.2			
03/23/1947	16-04-51	35.15	121.30	D	3.7			
03/27/1947	09-16-46	35.00	121.00	D	4.2		F	OFF COAST; V AT LOMPOC.
04/29/1947	07-44--?	34.33	119.55	C	3.2			
06/25/1947	18-39-53	34.25	119.50	C	3.1		F	NEAR CARPINTERIA.
06/25/1947	13-41-21	34.25	119.50	C	3.6		F	NEAR CARPINTERIA.
06/25/1947	18-48-26	34.25	119.50	C	2.5			
06/25/1947	20-55-16	34.25	119.50	C	3.2		F	NEAR CARPINTERIA.
06/25/1947	20-55-54	34.25	119.50	C	3.8			
07/13/1947	05-35--?	36.08	121.10	D	3.4			SOUTH OF KING CITY.
07/14/1947	05-40-06	35.92	119.92	C	4.0		F	KETTLEMAN HILLS; IV AT KETTLEMAN CITY.
10/6/1947	18-39--?	36.50	121.23	A	3.2			EAST OF GONZALES.
12/14/1947	05-42--?	36.45	121.08	B	3.4			SW OF LLANADA.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/16/1947	09-21-03	36.25	120.77	C	3.6		F	IV AT SAN LUCAS.
12/18/1947	19-30-06	36.12	120.90	D			F	IV AT PARKFIELD.
12/25/1947	06-05--?	35.60	121.10	D			F	CAMBRIA.
12/25/1947	06-20--?	35.60	121.10	D			F	CAMBRIA.
01/11/1948	05-37-28	36.43	121.48	B	4.3		F	IV AT HOLLISTER.
02/01/1948	17--?-54	34.42	119.92	C	3.0			
02/15/1948	08-04-06	35.88	120.37	A	3.4			EAST OF PARKFIELD.
03/07/1948	07-46-22	36.10	120.40	D	3.0			NEAR COALINGA.
03/10/1948	23-24-34	34.43	119.73	C	2.6			
03/18/1948	09-35-05	34.40	119.60	C	2.8			
03/29/1948	02-40--?	35.85	121.40	D			F	IV AT HOLLISTER.
04/23/1948	15-23-43	34.10	120.93	C	3.7			
05/05/1948	06-47-06	34.45	119.72	B	2.7			
05/07/1948	12--?-32	36.20	121.90	D	3.0			WEST OF PRIEST.
05/09/1948	11-10--?	34.75	120.25	D			F	V AT LOS ALAMOS.
07/14/1948	11-05-37	34.67	120.92	C	3.2			
07/17/1948	05-26-31	34.55	120.05	C	3.4			
07/28/1948	01-30-57	36.05	120.53	C	3.1			SE OF PRIEST.
07/29/1948	13-16-23	35.12	120.47	C	3.4			
08/04/1948	10-22-57	35.92	120.33	C	3.6			
09/03/1948	23-42-26	34.33	119.53	C	3.9		F	SANTA BARBARA.
09/17/1948	15-41-01	34.40	119.62	C	3.1			
10/27/1948	03-05--?	34.75	120.25	D			F	IV AT LOS ALAMOS.
10/29/1948	03-04-59	34.10	120.40	D	3.4		F	V AT ARLIGHT AND POINT ARGUELLO LIGHT STATION.
11/02/1948	19-06-45	34.37	119.58	C	2.9			
12/04/1948	06-44-20	34.43	119.72	C	2.8			
12/04/1948	23-32-51	34.42	119.50	C	2.7			
12/20/1948	04-42-46	35.80	121.50	C	4.5		F	OFF COAST, NEAR PIEDRAS BLANCAS POINT; III AT SAN SIMEON.
12/31/1948	14-35-46	35.67	121.40	B	4.6		F	ALONG THE COAST FROM LOMPOC TO MOSS LANDING; VI AT SAN SIMEON AND V AT CAYUCOS, CRESTON, MOSS LANDING, AND PIEDRAS BLANCAS LIGHT STATION.
01/25/1949	04-29--?	34.90	120.40	D			F	V AT ORCUTT AND SANTA MARIA.
03/27/1949	06-31-16	34.25	119.62	C	2.6			
04/06/1949	14-07--?	35.00	120.00		2.6			
04/08/1949	13-17-07	34.60	120.35	C	3.2		F	IV AT LOS ALAMOS.
04/14/1949	01-46-12	34.28	119.52	C	2.6			
04/23/1949	09-18-09	36.38	121.37	C	3.7			NORTH OF PARAISO.
05/06/1949	04-23-46	34.50	121.00	C	3.4			
05/10/1949	06-20--?	35.90	120.40	D			F	SANTA MARIA - SLIGHT.
05/10/1949	11--?--?	35.90	120.40	D			F	SANTA MARIA - SLIGHT.
05/16/1949	03-01-03	34.72	120.02	C	3.2			
05/17/1949	23-57-55	35.63	121.15	D	4.1		F	IV AT SAN SIMEON.
06/27/1949	10-35-31	35.80	121.10	D	4.5		F	V AT SAN ARDO AND SAN MIGUEL; ALSO FELT AT PASO ROBLES, SAN LUIS OBISPO, AND SANTA MARGARITA.
07/21/1949	16-50--?	36.15	120.35	D			F	IV AT COALINGA.
07/21/1949	17-01--?	36.15	120.35	D			F	IV AT COALINGA.
07/24/1949	03-04-05	36.00	120.00	D	2.3			SE. KINGS C0. AFTER SHOCK AT 06-26--?, MAG. 2.0.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/27/1949	18-21-35	34.53	120.37	C	3.6			
08/01/1949	-?-07-24	36.90	121.20	D	3.0			SOUTH OF KING CITY.
08/07/1949	01-38-43	36.50	121.50	D	2.3			NO. MONTEREY CO.
08/10/1949	09-17-39	36.50	121.00	C	2.6			CENTRAL SAN BENITO CO.
08/22/1949	03--?--?	36.00	120.00	D			F	KETTLEMAN HILLS. FIFTH SHOCK IN 2 WEEKS.
08/26/1949	16-52-32	34.50	120.50	D	4.2		F	NEAR POINT CONCEPTION. VI AT ARLIGHT AND SURF. IV AT GUADALUPE, LOMPOC, AND LOS ALAMOS.
08/27/1949	14-15--?	34.50	120.50	D			F	ARLIGHT. SLIGHT SHOCK.
08/27/1949	14-51-46	34.50	120.50	D	4.9		F	NEAR POINT CONCEPTION. VI AT ARLIGHT, LOMPOC, AND SUDDEN. V AT COSMALIA, LOS ALAMOS, NIPOMO, SANTA BARBARA, AND SURF.
08/29/1949	12-07-20	36.00	120.10	D	3.0		F	IV IN AVENAL AND KETTLEMAN CITY.
10/28/1949	08-07-02	36.80	120.90	C	2.6			NW OF PRIEST.
11/17/1949	05-06-06	34.80	120.70	D	2.8		F	IV AT SANTA MARIA.
12/28/1949	09-17-12	36.20	120.70	D	2.6			NEAR PRIEST.
02/19/1950	08-29-44	34.50	120.70	D	3.5			
03/09/1950	23-43-19	36.35	121.22	C	3.2		F	NORTH OF KING CITY; V AT ROBLES DEL RIO.
03/22/1950	01-31-57	35.97	120.63	C	3.7			
03/29/1950	12-43-20	35.97	120.88	D	3.5			
04/15/1950	11-56-32	35.75	119.62	C	4.6		F	NE OF LOST HILLS; V AT ASH MOUNTAIN, (SEQUOIA NATIONAL PARK), KERNVILLE, AND SHAFTER, AND IV AT BUTTONWILLOW, JAWBONE AQUEDUCT STATION, LOST HILLS, THREE RIVERS, AND VISALIA.
04/21/1950	13-17-29	34.38	119.58	B	3.0		F	IV AT SANTA BARBARA.
04/26/1950	07-23-29	35.20	120.60	C	3.5		F	V AT SANTA MARIA; ALSO FELT AT ORCUTT.
04/26/1950	07-38--?	35.20	120.60	D			F	SANTA MARIA.
05/21/1950	18-59-03	34.57	119.63	C	2.6			
05/21/1950	19-26-48	35.88	119.73	C	3.4			
05/24/1950	01-46-57	36.43	120.77	C	2.9			SE OF LLANADA.
07/13/1950	15-01-47	34.33	119.50	C	2.8		F	OFF CARPINTERIA; V AT MONTECITO; ALSO FELT AT SANTA BARBARA AND NEARBY AREAS.
08/01/1950	21-08-43	36.20	122.23	B	2.0			OFF COAST, WEST OF BIG SUR.
08/02/1950	06-50-48	34.67	120.63	C	3.3			
08/23/1950	09-10--?	34.40	119.50	D			F	IV AT RINCON POINT; FELT AT CARPINTERIA.
09/24/1950	04-45--?	34.50	120.50	D			F	III AT ARLIGHT.
09/24/1950	12-23--?	34.22	119.58	C	3.3			
09/24/1950	21-51-44	36.20	120.50	D	2.9			EAST OF PRIEST.
10/20/1950	08-23-25	36.33	121.07	C	2.7			SOUTH OF KING CITY.
11/21/1950	04-30--?	30.90	120.40	D			F	SANTA MARIA.
03/02/1951	02-13-44	36.10	120.60	D	3.1			SE OF PRIEST
03/04/1951	13-32--?	34.90	120.40	D			F	IV AT SANTA MARIA; 2 SHOCKS.
03/05/1951	09-50--?	34.90	120.40	D			F	IV AT SANTA MARIA.
03/10/1951	05-35--?	34.50	120.50	D			F	IV AT ARLIGHT.
03/15/1951	13-50-43	35.02	120.48	C	3.8		F	IV AT LOS ALAMOS.
03/26/1951	06-07-34	34.62	119.50	C	3.5		F	IV AT OJAI AND SUMMERLAND; FELT AT VENTURA.
05/04/1951	03-28-36	36.20	120.20	D	3.1			FORESHOCK OF QUAKE AT 20-08-10.
05/04/1951	20-08-10	36.20	120.20	D	3.2			EAST OF COALINGA.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
05/06/1951	03-18-03	36.40	120.40	D	2.8			NORTH OF COALINGA.
05/25/1951	05-11-18	36.30	120.30	D	3.1			NORTH OF COALINGA.
05/29/1951	05-08-24	35.08	119.65	C	3.2		F	ELKHORN HILLS; IV IN CUYAMA VALLEY.
05/31/1951	06-28-42	36.30	120.20	D	2.7			NE OF COALINGA.
06/16/1951	19-01-17	34.40	120.08	C	3.3			
06/19/1951	06-13-47	35.97	120.42	C	3.6			SOUTH OF COALINGA.
07/01/1951	-?-13-19	36.20	120.95	B	3.2			EAST OF KING CITY.
07/07/1951	05-53-33	34.75	120.75	C	3.5			
08/02/1951	05-09-25	36.35	121.27	B	3.9		F	NEAR GREENFIELD; IV AT BIG SUR, AT 7 MI. S OF HOLLISTER, AND ROBLES DEL RIO.
08/08/1951	19-42--?	34.80	120.40	D			F	IV AT ORCUTT.
08/09/1951	09-20-48	36.15	121.75	C	2.2			NEAR BIG SUR.
08/25/1951	01-04-10	36.47	121.15	B	3.1			SW OF LLANADA.
08/28/1951	22-12-27	34.60	121.00	D	3.5		F	OFF POINT ARGUELLO; III AT LOS ALAMOS.
09/18/1951	02-30--?	36.25	121.80	D			F	IV AT BIG SUR.
09/19/1951	22-50--?	36.25	121.80	D			F	IV AT BIG SUR.
10/03/1951	13-44-33	35.92	120.52	C	3.8			
10/26/1951	16-25-40	34.42	119.73	C	3.0			
11/17/1951	03-19-48	34.70	120.50	D	2.5		F	NEAR LOMPOC; III AT LOS ALAMOS.
11/25/1951	23-15-39	35.33	119.50	B	3.8			
12/20/1951	04-13-06	36.00	120.05	C	3.7			
01/24/1952	-?-32-38	34.18	119.88	C	2.7			
01/30/1952	11-05-33	36.30	121.13	C	2.7			NEAR KING CITY.
01/31/1952	20-09-02	34.18	119.53	C	2.6			
01/31/1952	21-33-12	36.40	121.40	C	3.6			SOUTHEAST CF SOLEDAD.
02/09/1952	22-26-39	34.07	120.75	C	3.6			
03/25/1952	09-18-50	34.18	120.95	C	3.6			
04/02/1952	05-21-10	36.45	121.25	B	3.1			NEAR SOLEDAD.
05/07/1952	05-45--?	34.40	119.60	D			F	IV AT MONTECITO AND SUMMERLAND.
06/18/1952	04--?--?	34.60	120.65	D			F	IV AT POINT ARGUELLO LIFEBOAT STATION.
07/01/1952	15-29-24	34.30	119.80	D	3.1			
07/15/1952	06-07-55	36.42	121.00	C	2.5			ABOUT 15 MI. NE OF KING CITY.
07/27/1952	18-15-14	34.18	119.70	C	3.1			
07/27/1952	20-20-35	34.22	119.67	C	3.2			
07/27/1952	20-30-05	34.20	119.67	B	3.5			
08/07/1952	19-l6-12	34.33	120.68	C	3.6		F	OFF POINT CONCEPTION; IV AT LOS ALAMOS.
08/11/1952	21-42-29	34.17	119.67	C	3.1			
08/23/1952	20-10--?	34.85	119.50	D			F	IV AT VENTUCOPA - SECOND SHOCK AT 21-20--?.
08/30/1952	14-58-11	34.35	119.62	B	3.3			
09/01/1952	12-03--?	34.30	119.60	D	3.0			
09/12/1952	21--?-15	34.25	119.70	C	3.0			
09/14/1952	11-46-06	35.90	120.30	D	3.3			
10/09/1952	14-46-02	34.20	122.20	D	4.6			(DEPT. OF WATER RESOURCES DATA)

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
11/22/1952	07-46-37	35.73	121.20	B	6.0		F	6 MI. NORTH OF SAN SIMEON, NEAR BRYSON; FELT OVER AN AREA OF 20,000 SQ. MI. VII AT BRADLEY AND BRYSON, VI AT ARROYO GRANDE, ATASCADERO, CAMBRIA, CAMP COOKE, CARMEL VALLEY, CAYUCOS, CHUALAR, CRESTON, GORDA STATION, GUADALUPE, HARMONY, HEARST RANCH, KING CITY, LOCKWOOD, LONOAK, MORRO BAY, OCEANO, PARKFIELD, PASO ROBLES, PISMO BEACH, SALINAS, SAN ARDO, SAN LUIS OBISPO, SAN SIMEON, SANTA MARGARITA, AND TEMPLETON, AND V AT AVENAL, BEN LOMOND, BIG SUR, BUELLTON, BUTTONWILLOW, CARUTHERS, CASMALIA, CHOLAME, COALINGA, CORCORAN, DOS PALOS, HOLLISTER, HUASNA, KETTLEMAN CITY, LOMPOC, LOST HILLS, LUCIA, MARICOPA, MONTEREY, MOSS LANDING, NIPOMO, ORCUTT, PAICINES, RIVERDALE, SAN MIGUEL, SANTA CRUZ, SANTA MARIA, SHAFTER, STRATFORD, SUDDEN, AND SURF.
11/22/1952	08-02-40	35.73	121.20	B	3.2			SAN SIMEON AFTERSHOCK.
11/22/1952	08-29-47	35.73	121.20	B	3.1			SAN SIMEON AFTERSHOCK.
11/22/1952	08-53-04	35.73	121.20	B	3.4		F	SAN SIMEON AFTERSHOCK; IV AT ARVIN, CALIENTE, JOLON, LOST HILLS, MALIBU, MARICOPA, MCFARLAND, MIRACLE HOT SPRINGS, MORGAN HILL, NIPOMO, PISMO BEACH, AND SHAFTER.
11/22/1952	11-08-44	35.73	121.20	B	3.1			SAN SIMEON AFTERSHOCK.
11/22/1952	11-45-31	35.73	121.20	B	3.1			SAN SIMEON AFTERSHOCK.
11/22/1952	12-34-44	35.73	121.20	B	3.0			SAN SIMEON AFTERSHOCK.
11/22/1952	13-37-31	35.73	121.20	B	4.0		F	SAN SIMEON AFTERSHOCK; V AT CALIENTE, MIRACLE HOT SPRINGS, AND WHEELER SPRINGS.
11/22/1952	19-25-21	35.73	121.20	B	3.9			SAN SIMEON AFTERSHOCK.
11/22/1952	19-36-27	35.70	121.20	D	3.1			SAN SIMEON AFTERSHOCK.
11/22/1952	23-39-20	35.70	121.20	D	3.1			SAN SIMEON AFTERSHOCK.
11/23/1952	09-22-35	36.00	120.90	D	3.2			20 MI. SE OF KING CITY.
11/23/1952	18-40-19	35.67	121.17	C	4.2			SAN SIMEON AFTERSHOCK.
11/25/1952	19-17-54	36.20	120.00	D	3.2			
11/25/1952	20-14-45	35.73	121.20	C	3.6			SAN SIMEON AFTERSHOCK.
11/25/1952	21-59-17	35.73	121.20	C	4.4			SAN SIMEON AFTERSHOCK.
11/26/1952	13-32-09	35.73	121.20	C	3.5			SAN SIMEON AFTERSHOCK.
11/27/1952	17-37-05	35.70	121.20	D	3.3			SAN SIMEON AFTERSHOCK.
11/28/1952	10-22-33	35.90	121.20	D	3.0			SAN SIMEON AFTERSHOCK.
11/29/1952	16--?--?	36.00	121.15	D			F	IV AT JOLON - TIME MAY BE 04--?--? ON 11/30/1952.
11/29/1952	23-15-58	35.70	121.20	D	3.5			SAN SIMEON AFTERSHOCK.
12/05/1952	01-05-57	36.50	120.70	D	3.0			14 MI. SE OF LLANADA.
12/06/1952	23-50--?	35.66	120.65	D			F	IV AT PASO ROBLES; FELT AT ADELAIDA.
12/12/1952	-?-27-07	36.40	120.97	B	3.0		F	17 MI. NE OF KING CITY; III AT LONOAK.
12/25/1952	16-44-10	34.40	121.40	D	3.6			
01/12/1953	13-05-18	35.80	121.10	D	3.2			14 MI. NE OF SAN SIMEON.
01/24/1953	-?--?--?	35.90	121.00	D			F	TEN SHOCKS REPORTED FELT FROM 1/24 TO 1/31 AT BRYSON (E. WEFERLING RANCH).
01/29/1953	20-31-19	35.80	121.10	D	3.1			14 MI. NE OF SAN SIMEON.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
02/03/1953	14-50-18	35.47	120.75	C	4.1		F	12 MI. NNW OF SAN LUIS OBISPO; V AT ATASCADERO, BRYSON, CRESTON, MORRO BAY, SANTA MARGARITA, AND IV AT CAYUCOS, PASO ROBLES, SAN LUIS OBISPO, AND TEMPLETON.
02/05/1953	02-54-12	35.90	121.00	D	2.8		F	IV AT BRYSON (E. WEFERLING RANCH).
02/15/1953	15-30--?	35.90	121.00	D			F	BRYSON (E. WEFERLING RANCH).
02/17/1953	08-06--?	35.90	121.00	D			F	III AT BRYSON (PLEYTO SCHOOL) - SEVERAL MILD SHOCKS REPORTED FELT DAILY SINCE SHOCK OF 11/21/52, 23-46-38 (NOT LISTED).
02/18/1953	14-10--?	35.90	121.00	D			F	BRYSON (E. WEFERLING RANCH) - MILD.
03/01/1953	18-53--?	35.90	121.00	D			F	V AT BRYSON.
03/04/1953	03-40--?	35.90	121.00	D			F	BRYSON (PLEYTO SCHOOL) - LIGHT.
03/15/1953	21--?-32	34.87	121.53	C	3.7			
03/18/1953	05-03--?	35.90	121.00	D			F	III AT BRYSON (PLEYTO SCHOOL).
03/29/1953	17-19-48	35.90	120.20	D	3.7			
04/08/1953	-?-59-20	34.80	120.60	D	3.6		F	NEAR CASMALIA; IV AT LOS ALAMOS.
04/15/1953	-?-29-10	35.83	121.07	C	3.1		F	14 MI. NNE OF SAN SIMEON; IV AT BRYSON.
04/15/1953	05-30--?	35.90	121.00	D			F	BRYSON - LIGHT.
04/29/1953	05-26-53	36.00	121.15	C	3.5		F	14 MI. S OF KING CITY - USCGS GIVES TIME AS 05-26-52, LOCATION AS N35.8 121.2W, REPORT AS NEAR BRYSON; V AT PLEYTO SCHOOL. 22 MI. NE OF KING CITY.
05/01/1953	22-16-51	36.40	120.80	D	3.0			
05/08/1953	08-15--?	34.65	120.45	D			F	III AT LOMPOC.
05/14/1953	03-36--?	36.00	120.00	D	3.3			
05/14/1953	09-36-09	35.75	121.08	B	3.7		F	9 MI. NE OF SAN SIMEON - USCGS GIVES N35.52 121.28W, OFF CAMBRIA; V AT BRYSON.
05/15/1983	07-15--?	35.90	121.00	D			F	IV AT BRYSON (PLEYTO SCHOOL).
05/28/1953	03-51-13	35.88	120.50	B	4.3		F	20 MI. SW OF COALINGA; IV AT PASO ROBLES AND III AT SAN MIGUEL.
05/28/1953	07-58-33	35.88	120.50	C	3.5		F	AFTERSHOCK OF 03-51-13; FELT AT SAN MIGUEL.
05/29/1953	10-20-16	35.90	121.20	D	2.9			20 MI. SOUTH OF KING CITY.
05/31/1953	23-51-17	36.10	120.40	D	3.2			NEAR COALINGA.
06/04/1953	11-40--?	35.50	120.50	D			F	V AT CRESTON - PROBABLY A BLAST.
06/06/1953	20-26-33	36.00	120.30	D	2.9			10 MI. SOUTH OF COALINGA.
06/19/1953	11-24-50	36.30	120.70	D	2.8			20 MI. EAST OF KING CITY.
06/22/1953	15-22-35	35.93	120.38	C	4.3		F	15 MI. WSW OF COALINGA; FELT AT COALINGA AND PASO ROBLES.
07/01/1953	22-17-20	34.60	121.35	D	3.2		F	OFF POINT ARGUELLO; IV AT POINT ARGUELLO LIGHT STATION.
08/14/1953	01-40-06	36.30	120.30	D	2.9			8 MI. NORTH OF COALINGA.
08/14/1953	09-22-50	36.50	121.20	D	2.3			20 MI. NORTH OF KING CITY.
09/02/1953	09-41-20	35.90	120.80	D	3.0			30 MI. SE OF KING CITY.
09/03/1953	11--?--?	35.50	120.50	D			F	CRESTON.
09/04/1953	03-54-25	35.90	120.32	C	3.5		F	15 MI. SOUTH OF COALINGA; IV AT CRESTON AND PASO ROBLES.
09/22/1953	07-36-58	36.40	121.20	D	3.8			NORTH OF KING CITY.
09/23/1953	06-21-51	35.70	121.10	D	3.5		F	NEAR SAN SIMEON; V AT BRYSON.
10/01/1953	03-56-15	36.25	121.83	C	3.4		F	25 MI. S OF MONTEREY; IV AT BIG SUR.
10/16/1953	03-45-35	35.95	120.53	C	3.4			SOUTHWEST OF COALINGA.
10/21/1953	16-02-38	34.32	119.70	B	4.0		F	OFF SANTA BARBARA; V AT SANTA BARBARA AND VICINTIY, AND IV AT GOLETA AND LOS PRIETOS RANGER STATION.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
10/24/1953	13-24-30	35.90	121.10	D	3.6			SOUTH OF KING CITY.
10/25/1953	08-43-25	36.50	121.50	D	3.2			NORTHWEST OF KING CITY.
11/02/1953	-?-52-06	36.40	121.30	D	3.4			NORTHWEST OF KING CITY.
01/04/1954	23-03-11	36.12	120.63	B	3.2			14 MI. WEST OF COALINGA.
01/05/1954	-?-23-23	35.93	120.00	C	3.0			SOUTHEAST OF COALINGA.
01/15/1954	22-02-18	36.50	121.23	C	2.6			NORTH OF KING CITY.
01/24/1954	19-06-45	35.78	121.08	C	3.1			30 MI. SOUTH OF KING CITY.
01/26/1954	09-43-22	34.50	120.33	C	3.8		F	W OF LAS CRUCES; III AT SANTA YNEZ.
03/09/1954	19-55-30	35.90	120.50	D	3.6		F	16 MI. SSE OF COALINGA; FELT NEAR PARKFIELD.
03/15/1954	22-43-50	35.00	120.70	D	3.4			
03/18/1954	12-07-53	35.40	120.90	D	3.0			NORTHWEST OF SAN LUIS OBISPO.
04/01/1954	12-04-38	36.05	120.20	C	3.3			6 MI. SOUTHEAST OF COALINGA.
04/09/1954	07-38-23	35.78	121.08	D	3.1			10 MI. NORTHEAST OF SAN SIMEON.
04/09/1954	14-58--?	35.90	121.00	D			F	IV AT BRYSON (PLEYTO SCHOOL); SECOND SHOCK REPORTED FELT AT 23-40--?.
04/20/1954	09-32-18	36.63	121.03	D	2.6			12 MI. NORTHEAST OF KING CITY.
05/10/1954	14-24-28	36.08	120.80	C	3.1		F	NE OF SAN ARDO - SLIGHT AT KING CITY.
06/04/1954	11-58-38	36.45	121.13	C	3.5			16 MI. SOUTHWEST OF LLANADA.
07/05/1954	07-25-39	36.20	121.80	D	3.2			30 MI. SOUTH OF MONTEREY.
08/13/1954	13-36-44	34.25	120.50	C	3.2			
08/13/1954	13-44-23	34.25	120.50	C	3.2			
08/19/1954	11-45-08	34.25	120.50	C	3.2			
08/21/1954	22-50-49	35.47	121.33	B	3.3			40 MI. SOUTH OF HOLLISTER.
08/22/1954	08-34-40	34.33	120.67	C	3.8			
08/22/1954	12-36-07	34.33	120.67	C	3.8			
12/22/1954	21-12-24	36.00	121.00	D	3.7		F	SE OF KING CITY; III AT KING CITY.
12/22/1954	21-12-28	36.00	120.60	D	3.8			
01/07/1955	14-50-22	34.40	119.60	D	3.0			
01/18/1955	13-30--?	36.20	121.85	D			F	IV REPORTED FELT AT BIG SUR.
02/05/1955	07-10-19	35.80	121.40	C	3.3			WEST OF SAN SIMEON.
02/27/1955	03-17-51	36.25	120.83	C	2.9		F	EAST OF KING CITY; IV IN PRIEST VALLEY.
03/02/1955	03-30--?	36.00	120.70	D			F	IV REPORTED FELT IN INDIAN VALLEY.
03/02/1955	15-59-01	36.00	120.93	B	4.8		F	18 MI. SE OF KING CITY; FELT OVER 7000 SQ. MI. OF W CENTRAL CALIF. USCGS MAG. 5.1. VI AT ADELAIDA, BRYSON, INDIAN VALLEY, SAN ARDO, SAN LUCAS, AND TEMPLETON.
03/02/1955	20-02-53	36.00	120.93	B	3.7			AFTERSHOCK OF QUAKE AT 15-59-01.
03/05/1955	08-46-36	36.10	121.10	D	2.0			SOUTH OF KING CITY.
03/06/1955	10-47-32	35.92	120.90	D	3.2			SOUTHEAST OF KING CITY.
04/04/1955	20-56-56	36.08	121.00	C	3.2			SOUTHEAST OF KING CITY.
04/27/1955	09-28-08	35.90	121.20	D	2.8			SOUTHWEST OF KING CITY.
05/14/1955	20--?--?	36.35	121.85	D			F	IV REPORTED FELT AT BIG SUR AND SANTA CRUZ.
05/16/1955	18-22-52	35.92	120.58	C	3.0			SOUTHWEST OF COALINGA.
05/30/1955	09-38-29	36.25	121.25	C	3.0			WEST OF KING CITY.
05/31/1955	01-45-53	36.40	121.25	B	3.0			NORTH OF KING CITY.
06/13/1955	14-55-12	36.30	121.30	C	2.5			NORTH OF KING CITY.
06/19/1955	05-36-33	35.62	121.10	C	2.5			SOUTHEAST OF SAN SIMEON.
07/06/1955	11-29-18	36.50	121.42	C	3.4			SOUTH OF HOLLISTER.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/06/1955	13-18-53	36.50	121.50	D	2.7			SOUTH OF HOLLISTER.
07/28/1955	12-07-52	36.50	121.40	D	2.6			SOUTH OF HOLLISTER.
09/21/1955	18-06-52	36.50	121.00	D	3.3			NORTH OF KING CITY.
10/22/1955	07-04-18	36.22	120.33	C	4.2		F	V AT AND 14 MI. NW OF COALINGA.
11/02/1955	19-40-06	36.00	120.92	A	5.2		F	55 MI. NNW OF SAN LUIS OBISPO; FELT OVER 7000 SQ. MI. OF COASTAL W CENTRAL CALIF. VI AT ADELAIDA RD. (14 MI. W OF PASO ROBLES), BRYSON, KING CITY, PASO ROBLES, SAN ARDO, SAN LUCAS, AND SAN MIGUEL.
11/18/1955	09-03-30	35.90	120.50	D	2.9			SOUTHWEST OF COALINGA.
11/19/1955	07-20--?	34.50	119.65	D			F	REPORTED FELT AT SANTA BARBARA.
11/19/1955	10-59-41	36.03	120.90	C	3.3			SOUTHEAST OF KING CITY.
11/21/1955	21-14-18	36.10	119.90	D	3.5			
12/11/1955	20-10-38	36.27	120.72	C	3.5			NORTHWEST OF COALINGA.
12/16/1955	14-43-11	36.03	120.87	C	3.8		F	SOUTHWEST OF KING CITY; FELT AT ATASCADERO, PASO ROBLES, AND SAN MIGUEL.
12/29/1955	13-33-17	36.45	121.25	C	3.4			NORTH OF KING CITY.
02/14/1956	22-15-08	36.50	121.10	D	2.8			SOUTHWEST OF LLANADA.
03/15/1956	15-26-11	36.50	121.20	D	2.6			SOUTHEAST OF HOLLISTER.
04/03/1956	09-26-02	36.45	121.23	B	2.7			SOUTH OF HOLLISTER.
04/10/1956	11-24-21	36.43	121.48	C	2.9			SOUTHEAST OF MONTEREY.
04/10/1956	20-53-21	36.30	121.00	D	2.9			NORTHEAST OF KING CITY.
05/01/1956	15-06-33	36.50	121.00	D	2.5			SOUTH OF HOLLISTER.
05/04/1956	08-16-14	35.75	121.07	B	3.1			NORTHEAST OF SAN SIMEON.
05/04/1956	08-16-16	35.95	120.93	D	3.5			
05/15/1956	10-45--?	34.90	120.40	D			F	REPORTED FELT AT SANTA MARIA.
06/11/1956	-?-48-37	36.00	120.97	C	3.2			SOUTHEAST OF KING CITY.
06/15/1956	23-42-03	36.30	121.80	D	2.8			SOUTH OF MONTEREY.
07/09/1956	23-15--?	35.10	120.50	D			F	III REPORTED FELT NEAR HUASNA.
07/23/1956	08-03-48	36.30	121.30	D	4.7		F	NW OF KING CITY; FELT OVER 4000 SQ. MI. OF COASTAL CENTRAL CALIF. V AT BIG SUR, CHUALAR, GONZALES, GREENFIELD, 7.5 MI. S OF HOLLISTER, KING CITY, PASO ROBLES, SAN BENITO, AND SAN JUAN BAUTISTA.
07/23/1956	08-20-37	36.50	121.40	D	3.1			AFTERSHOCK OF QUAKE AT 08-03-48.
07/31/1956	-?-40-43	34.15	119.60	C	3.2			
07/31/1956	17-25--?	35.10	120.50	D			F	IV REPORTED FELT AT HUASNA.
08/09/1956	-?-08-49	34.37	119.80	B	4.0		F	OFF SANTA BARBARA; IV AT LOS PRIETOS RANGER STATION.
08/10/1956	23-24-03	35.90	121.30	D	3.0			SOUTHWEST OF KING CITY.
08/20/1956	05-10-33	36.48	121.48	B	3.2		F	NEAR GONZALES; IV AT PINNACLES NATIONAL MONUMENT.
09/15/1956	-?-34-37	36.30	120.30	D	2.7			NORTH OF COALINGA.
10/10/1956	20-02-24	34.70	121.00	D	3.8			
11/12/1956	10-13--?	36.30	120.10	C	3.3			
11/16/1956	03-23-09	35.95	120.47	B	5.0		F	SW OF COALINGA; FELT OVER 8000 SQ. MI. FROM HOLY CITY TO BETTERAVIA TO FIREBAUGH. VI AT KING CITY, MEE RANCH (LONOAK), AND SAN LUCAS.
11/19/1956	13-53-53	35.98	120.57	C	3.3		F	SOUTHWEST OF COALINGA; III AT ADELAIDA (15 MI. WEST OF PASO ROBLES).
11/20/1956	03-42-44	34.70	120.50	C	3.6		F	IV AT LOS ALAMOS; III FELT AT 07-42--?, 11/21/1956.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/11/1956	10-56-53	35.88	120.47	C	4.1			NEAR PARKFIELD.
12/28/1956	13-39-37	35.90	121.10	D	2.6			NORTHEAST OF SAN SIMEON.
01/01/1957	09-25--?	35.50	120.65	D			F	REPORTED FELT AT ATASCADERO.
01/29/1957	21-19-53	35.87	122.12	C	4.9		F	OFF COAST NW OF SAN SIMEON; FELT OVER 5000 SQ. MI. OF COASTAL CENTRAL CALIF. V AT BIG SUR, CAMBRIA, CARMEL VALLEY, HARMONY, KING CITY, LUCIA, MARINA, AND SEASIDE, AND IV GENERALLY FROM MOSS LANDING TO 20 MI. W OF COALINGA TO SAN LUIS OBISPO.
02/03/1957	07-57-12	34.50	121.20	C	3.9			
02/08/1957	04-45-38	36.50	121.20	D	2.8			NORTH OF KING CITY.
02/08/1957	21-20--?	36.50	122.00	D			F	SHARP SHOCK FELT MONTEREY PEN. (BSSA).
02/09/1957	08-10--?	35.50	120.65	D			F	IV REPORTED FELT AT ATASCADERO.
02/14/1957	-?-31-30	35.10	119.80	D	2.4			
02/14/1957	10-30-27	36.00	120.60	C	3.6			
02/16/1957	11-43-50	34.30	119.53	C	3.5			
03/09/1957	14-38-28	34.70	119.60	C	2.9			
03/09/1957	14-59-21	34.70	119.60	C	2.4			
04/05/1957	-?-40--?	34.75	120.25	D			F	IV REPORTED FELT AT LOS ALAMOS.
06/21/1957	20-46-42	35.10	120.90	D	3.7		F	OFF COAST; FELT AT SAN LUIS OBISPO AND MORRO BAY.
07/02/1957	09-18-22	34.37	119.88	B	3.4		F	W OF SANTA BARBARA; FELT AT SANTA BARBARA.
07/02/1957	12-59-05	34.37	119.88	B	3.3			
07/02/1957	13-58-28	34.37	119.88	B	3.2			
07/21/1957	01-29-20	36.43	121.22	B	3.1			NORTH OF KING CITY.
08/03/1957	09-31-22	36.25	120.88	C	2.5			EAST OF KING CITY.
08/18/1957	03-05-25	34.47	120.13	C	3.4			
08/18/1957	11-08-23	34.47	120.13	C			F	N OF GAVIOTA; FELT AT CACHUMA RESERVOIR.
08/21/1957	07-36-54	36.47	121.52	C	3.6			NORTHWEST OF KING CITY.
08/28/1957	01-13-57	34.58	121.00	C	3.5			
09/12/1957	21-36--?	35.50	121.00	D			F	II FELT AT P G AND E PLANT, MORRO BAY.
09/21/1957	06-54-26	36.40	121.10	D	2.8			NORTH OF KING CITY.
09/21/1957	15-32--?	35.50	121.00	D			F	II FELT AT P G AND E PLANT, MORRO BAY.
09/25/1957	23-33-31	36.50	121.50	D	2.7			SOUTH OF HOLLISTER.
10/01/1957	12-55-57	36.47	121.23	C	3.3			SOUTHWEST OF LLANADA.
10/05/1957	14-42--?	34.75	120.25	D			F	IV REPORTED FELT AT LOS ALAMOS.
10/19/1957	-?-04-38	36.10	120.87	B	3.3			SOUTHEAST OF KING CITY.
10/28/1957	11-41-02	34.33	120.00	C	2.8			
11/05/1957	23-50-52	34.72	120.33	C	3.4			
11/18/1957	01-11-42	36.38	121.23	C	3.1			NORTHWEST OF KING CITY.
11/18/1957	07-26-32	36.50	121.70	D	3.3			SOUTHEAST OF MONTEREY.
12/31/1957	22-32-55	36.40	121.00	D	2.9			NORTHEAST OF KING CITY.
01/07/1958	17-13-16	35.70	120.80	D	3.0			NORTH OF SAN LUIS OBISPO.
01/18/1958	08-12--?	35.55	120.65	D			F	REPORTED FELT AT PASO ROBLES.
01/21/1958	21-22-08	36.40	120.50	D	2.9			NORTHWEST OF COALINGA.
01/23/1958	07-06-46	34.38	119.58	B	2.6		F	E OF SANTA BARBARA; IV AT SANTA BARBARA.
01/28/1958	07-12-54	36.50	121.10	D	2.2			SOUTHWEST OF LLANADA.
03/26/1958	13-12-30	36.20	120.30	D	2.4			NEAR COALINGA.
03/27/1958	20-26-14	35.90	121.50	D	2.8			NORTHWEST OF SAN SIMEON.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/31/1958	17-38-23	36.50	121.10	D	2.7			SOUTHWEST OF LLANADA.
04/10/1958	08-32-33	36.45	121.12	C	2.9			SOUTHWEST OF LLANADA.
06/05/1958	17-12-50	36.40	121.10	D	3.1			NORTH OF KING CITY.
06/15/1958	07-02-33	36.50	121.38	C	2.9			FORESHOCK OF QUAKE AT 07-05-34.
06/18/1958	07-05-34	36.50	121.38	C	3.3			SOUTH OF HOLLISTER.
06/21/1958	01-03-31	36.40	120.40	D	2.1			SOUTHWEST OF FRESNO.
07/02/1958	17-56-26	36.50	121.30	D	2.8			SOUTHWEST OF LLANADA.
08/08/1958	18-43-01	36.30	121.20	D	2.7			FORESHOCK OF 13-43-15 - RECORDS MIXED.
08/08/1958	13-43-15	36.30	121.20	D	3.9		F	NORTHWEST OF KING CITY; IV AT BIG SUR.
08/18/1958	05-30-42	35.80	121.30	D	3.4			NEAR SAN SIMEON.
09/01/1958	11-31-42	36.10	120.80	D	3.2			SOUTHEAST OF KING CITY.
09/21/1958	07-24-55	36.35	121.12	C	4.0		F	NORTH OF KING CITY; VI AT SAN BENITO; ALSO FELT AT SOLEDAD.
09/21/1958	14-23-01	36.50	121.05	C	2.7			SOUTHWEST OF LLANADA.
10/03/1958	04-25-51	34.37	119.50	B	3.7		F	FROM CARPINTERIA TO GOLETA.
10/10/1958	13-05-16	35.93	120.50	B	4.5		F	SOUTHWEST OF COALINGA; FELT OVER AN AREA OF APPROXIMATELY 3500 SQ. MI. OF THE SOUTHWEST-CENTRAL REGION OF CALIFORNIA - APPEARS TO HAVE BEEN FELT MORE STRONGLY AT PARKFIELD THAN ELSEWHERE; V AT ADELAIDA, CAMP ROBERTS, COALINGA, HARMONY, LONE PINE INN, OILFIELD, PARKFIELD, PASO ROBLES, AND SAN ARDO.
10/15/1958	16-16-44	35.50	121.20	D	3.2			NEAR SAN SIMEON.
11/06/1958	20-11-57	36.08	120.88	C	3.1			SOUTHEAST OF KING CITY.
11/16/1958	09-34-04	34.50	119.83	C	4.0		F	NW OF SANTA BARBARA; FELT OVER 600 SQ. MI. FROM SANTA YNEZ TO VENTURA; V AT CARPINTERIA, GOLETA, AND SANTA BARBARA.
11/27/1958	06-04-26	36.37	121.15	C	3.9		F	WEST OF LLANADA; FELT SLIGHTLY AT CARMEL.
11/27/1958	13-39-01	36.20	120.80	D	3.1			EAST OF KING CITY.
12/15/1958	14-58-49	36.20	120.40	D	3.0			NEAR COALINGA.
12/15/1958	15-24-01	36.20	120.40	D	3.0		F	NEAR COALINGA; IV AT COALINGA.
12/30/1958	01-34-15	35.92	119.80	C	3.2			
01/11/1959	05-18-26	36.20	120.80	D	2.5			WEST OF COALINGA.
02/07/1959	05-51-02	36.10	120.00	D	3.0			SOUTHEAST OF KING CITY.
02/27/1959	21-35-01	36.25	120.75	C	3.1			SOUTHEAST OF LLANADA.
03/13/1959	02-44-27	35.80	120.30	D	2.5			SOUTH OF COALINGA.
03/14/1959	02-43-41	35.70	121.30	D	3.6			WEST OF SAN SIMEON.
03/20/1959	05-12-09	36.48	121.17	B	2.9			SOUTHWEST OF LLANADA.
03/25/1959	05-34-17	34.25	119.58	C	2.5		F	SANTA BARBARA CHANNEL; IV AT CARPINTERIA.
04/08/1959	07-41-57	36.37	121.20	B	3.4			NORTH OF KING CITY.
04/09/1959	14-03-11	36.38	121.15	C	2.5			NORTH OF KING CITY.
04/21/1959	09-36-23	36.40	120.40	D	3.0			NORTH OF COALINGA.
04/21/1959	12-31-10	36.10	121.10	D	2.2			NEAR KING CITY.
04/22/1959	19-04-25	36.20	120.90	D	2.6			NEAR KING CITY.
05/13/1959	14-28-10	36.48	121.03	C	2.6			SOUTHWEST OF LLANADA.
05/14/1959	01-34-09	36.50	121.20	D	2.4			SOUTHWEST OF LLANADA.
05/20/1959	10-15-55	36.30	120.40	D	2.6			NORTHWEST OF COALINGA.
06/01/1959	03-47-24	36.50	121.23	C	2.4			SOUTHWEST OF LLANADA.
06/20/1959	15-01-17	36.50	121.30	D	2.9			SOUTH OF VINEYARD.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/21/1959	09-24-07	34.32	119.67	B	3.3			
07/18/1959	01-11-47	36.50	121.30	D	2.5			SOUTH OF VINEYARD.
08/05/1959	03--?-34	35.95	120.48	C	3.5		F	SOUTHEAST OF COALINGA (NEAR PARKFIELD; FELT STRONGEST AT PARKFIELD; IV FELT AT PASO ROBLES).
09/05/1959	05-45-34	36.50	121.70	D	3.8			SOUTHWEST OF VINEYARD.
10/01/1959	04-35-35	34.43	120.57	B	4.5		F	OFF POINT CONCEPTION; VI AT GAVIOTA PASS AND V AT GAVIOTA, GOLETA, AND LOMPOC.
10/01/1959	05-52-55	34.20	119.50	C	3.2			
10/11/1959	02-03-09	36.45	121.12	C	4.1		F	SOUTHWEST OF LLANADA; FELT AT SALINAS.
10/24/1959	23-12-54	36.47	121.40	C	3.2			SOUTH OF HOLLISTER.
10/25/1959	03-33-13	36.50	121.20	D	2.4			SOUTHEAST OF VINEYARD.
10/25/1959	03-34-02	36.50	121.32	C	3.0			SOUTH OF VINEYARD.
10/26/1959	09-56-01	36.40	121.10	D	3.0			SOUTHEAST OF VINEYARD.
11/25/1959	09-28-22	35.20	121.20	D	3.5			SOUTH OF KING CITY.
11/26/1959	07-02-05	36.40	121.40	D	2.7			SOUTH OF VINEYARD.
12/11/1959	05-55-26	35.60	120.60	D	3.5			SOUTHEAST OF VINEYARD.
12/25/1959	20-38-28	36.00	120.60	D	3.1			SOUTHEAST OF VINEYARD.
12/29/1959	14-53-08	35.75	120.30	C	3.5		F	NEAR CHOLAME; FELT AT PASO ROBLES.
01/02/1960	22-51-48	35.40	121.20	D	4.0			NW OF SAN LUIS OBISPO.
01/04/1960	12-18-20	36.20	120.70	D	3.2			WEST OF COALINGA.
02/14/1960	08-34-30	35.80	121.70	D	2.8			WEST OF SAN SIMEON.
02/25/1960	06-34-31	36.50	121.20	D	2.7			SOUTHWEST OF LLANADA.
02/28/1960	02-55-32	34.33	119.95	C	3.1			
03/21/1960	20-46-39	36.50	120.73	C	2.5			SOUTHEAST OF LLANADA.
03/26/1960	21-39-21	36.22	121.00	C	2.7			EAST OF KING CITY.
03/29/1960	11-46-42	36.50	121.10	C	2.4			SOUTHEAST OF VINEYARD.
03/31/1960	08-35-09	36.40	121.20	D	2.6			SOUTHEAST OF VINEYARD.
04/02/1960	13-02-10	35.97	120.33	C	2.7			SOUTH OF COALINGA.
04/02/1960	19-01-12	36.20	120.60	D	3.4			WEST OF COALINGA.
04/09/1960	08-01-14	36.50	121.13	B	3.6			SOUTHEAST OF HOLLISTER.
05/04/1960	09-44-32	36.42	120.72	C	3.4			SOUTHEAST OF LLANADA.
05/15/1960	06-07-23	36.43	121.27	C	2.5			SOUTH OF VINEYARD.
06/11/1960	17-39-48	36.30	120.90	D	3.7			SOUTHEAST OF VINEYARD, DIABLO RANGE.
06/19/1960	19-51-20	36.20	121.90	D	2.6			SOUTHWEST OF BIG SUR.
06/24/1960	18-13-12	36.45	121.22	B	3.5			SOUTHEAST OF VINEYARD.
07/14/1960	03-22-23	35.60	120.40	D	3.0			NORTHEAST OF SAN LUIS OBISPO.
07/20/1960	-?-59-36	35.80	119.80	D	2.8			NORTHEAST OF SAN LUIS OBISPO.
07/30/1960	02-16-29	36.43	120.28	C	2.5			SOUTHWEST OF FRESNO.
08/09/1960	08-59-47	36.20	120.20	D	3.2			EAST OF COALINGA.
08/10/1960	03-03-50	36.47	121.40	D	3.2			SOUTH OF VINEYARD.
08/26/1960	08-57-24	38.33	121.13	C	3.0			SOUTHEAST OF VINEYARD.
09/10/1960	01-18-22	36.47	121.05	C	2.7			SOUTHEAST OF HOLLISTER.
09/10/1960	20-49-12	36.45	121.28	D	2.8			SOUTHWEST OF LLANADA.
10/08/1960	-?-02-29	36.50	121.67	C	3.0			SOUTHWEST OF VINEYARD.
11/03/1960	07-13-40	36.43	121.07	C	2.7			SOUTH-SOUTHWEST OF LLANADA.
11/18/1960	04-36-44	36.38	121.20	C	3.0			NORTH-NORTHWEST OF KING CITY.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
12/01/1960	14-23-49	34.33	119.85	B	3.2			OFF SANTA BARBARA.
12/15/1960	08-28-08	36.40	121.30	D	3.0			SOUTH OF HOLLISTER.
12/27/1960	03-57-55	36.00	121.10	D	3.3			SOUTH OF KING CITY.
01/06/1961	20-46-36	35.80	120.20	D	3.4			SE OF PARKFIELD.
02/02/1961	12-31--?	36.35	121.20	D	2.2			NORTHWEST OF KING CITY.
02/21/1961	15-46-58	34.37	119.53	C	2.8			SE OF SANTA BARBARA.
03/14/1961	04-15--?	36.40	121.20	D	3.4			SOUTHEAST OF VINEYARD.
03/29/1961	16--?-11	36.50	121.50	D	2.5			SOUTH OF VINEYARD.
04/07/1961	12-21-19	36.20	120.40	D	2.9			NORTH OF COALINGA.
04/08/1961	04-55-26	36.00	121.20	D	2.7			SOUTH OF KING CITY.
04/08/1961	09-29-47	36.10	120.43	B	3.4			NEAR COALINGA.
04/08/1961	12-52-16	36.12	120.43	C	2.7			AFTERSHOCK OF QUAKE AT 09-29-47.
04/11/1961	09-08-11	36.00	120.10	D	2.7			SOUTHEAST OF KING CITY.
04/12/1961	04-59-08	35.92	120.50	C	2.6			SOUTHWEST OF COALINGA.
04/19/1961	18-16-35	36.40	121.58	C	3.3			SOUTHEAST OF MONTEREY.
05/25/1961	14-19-05	36.33	121.00	C	3.4			NORTHEAST OF KING CITY.
05/25/1961	14-19-35	36.33	121.00	B	3.4			NORTHEAST OF KING CITY.
06/01/1961	06-47-20	36.33	121.32	B	2.7			NORTHWEST OF KING CITY.
06/01/1961	14-11-30	36.45	121.20	C	2.6			NORTH OF KING CITY.
06/18/1961	12-50-59	36.18	120.83	C	2.1			EAST OF KING CITY.
06/25/1961	13-15-26	36.48	121.35	C	3.6		F	SOUTH OF HOLLISTER; FELT IN HOLLISTER AREA. INTENSITY IV 7.5 MI. SOUTH OF HOLLISTER AT HARRIS RANCH.
06/26/1961	11-30-22	35.77	122.00	C	2.5			OFF SAN SIMEON COAST.
07/22/1961	18-01-55	36.40	121.20	C	4.0		F	NORTHEAST OF PARAISO; FELT AT PINNACLES NATIONAL MONUMENT (ABOUT 25 MI. SOUTHEAST OF HOLLISTER).
07/31/1961	-?-07-09	35.82	120.37	C	4.7		F	SAN LUIS OBISPO; FELT OVER AN AREA OF 5000 SQ. MI. OF WEST CENTRAL CALIFORNIA. INTENSITY V AT ATASCADERO, CHOLAME, CRESTON, PARKFIELD, SAN LUIS OBISPO, AND TEMPLETON.
08/01/1961	06-12-54	36.43	120.85	C	3.1			SOUTH OF LLANADA.
08/17/1961	17-14-45	36.33	120.95	B	3.1			NORTHEAST OF KING CITY.
09/14/1961	15-12-20	34.32	119.63	C	2.7			
09/14/1961	15-14-38	34.32	119.63	C	2.8			
09/27/1961	02-02-06	36.33	121.25	C	2.7			EAST OF PARAISO.
09/29/1961	15-39-58	36.33	120.88	B	2.4			SOUTH OF LLANADA.
10/12/1961	06-31-11	35.80	121.30	D	2.3			NORTH OF SAN SIMEON.
10/29/1961	11-47-33	36.33	120.92	C	2.0			EAST OF PARAISO.
11/05/1961	10-43-57	36.03	120.10	D	2.0			EAST OF LLANADA.
11/29/1961	04-49-03	35.15	120.13	C	3.0			SOUTHEAST OF KING CITY.
12/06/1961	03-27-30	36.43	121.85	B	2.4			SOUTH OF MONTEREY.
12/14/1961	07-28-44	36.48	121.08	B	2.1			SOUTHWEST OF LLANADA.
01/04/1962	03-56-10	36.40	121.40	C	3.0	11		NORTHWEST OF KING CITY.
01/31/1962	08-33-15	34.88	120.68	C	3.6			
02/01/1962	06-37-57	34.88	120.68	C	4.5		F	WEST OF GUADALUPE; FELT OVER AN AREA OF 3000 SQ. MI. V AT ARROYO GRANDE, AVILA BEACH, CASMALIA, GROVER CITY, GUADALUPE, HALCYON, OCEANO, POINT ARGUELLO, AND SHELL
02/01/1962	07-58-12	34.38	120.68	C	3.7			
02/04/1962	11-43-34.1	36.42	121.27	C	3.2	12		

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
02/07/1962	13--?-70	34.30	122.10	D	3.9			
03/05/1962	07-44-01	34.60	121.60		4.5		F	OFF COAST NEAR LOMPOC; V AT MORRO BAY AND PISMO BEACH.
03/06/1962	03-40-22	34.60	121.60	D	3.6			
03/10/1962	08-07-21	34.60	121.60	D	4.2			OFF COAST NEAR LOMPOC.
03/10/1962	13-40-48	34.60	121.60	D	4.0			OFF COAST NEAR LOMPOC.
03/10/1962	15-24-21	34.60	121.60	D	3.5			
03/12/1962	21-32-09	34.60	121.60	D	3.9			
03/23/1962	22-10-18	34.28	120.20	C	2.9			
03/24/1962	03-38-41.8	36.20	119.78	B	3.4	19		SOUTH OF FRESNO.
04/02/1962	03-06-03.2	36.25	120.10	B	3.7	16	F	EAST OF COALINGA; V IN TEHACHAPI.
04/15/1962	08-41-02.3	36.42	120.62	B	4.7	23	F	SOUTHEAST OF LLANADA; V AT IDRIA.
05/04/1962	20-52-32	35.27	119.55	B	2.8			
05/05/1962	-?-55-20	34.20	121.50	D	3.3			
09/03/1962	17-53-33.1	36.47	121.07	C	2.6	8	F	SOUTHWEST OF LLANADA; FELT IN HOLLISTER.
09/11/1962	01-34-31	36.03	121.23	B	3.3	16		SOUTHWEST OF KING CITY.
09/16/1962	18-12-35	34.48	119.68	B	4.0		F	NEAR SANTA BARBARA; V AT LOS PRIETOS.
09/16/1962	18-17-09	34.48	119.68	C	2.2			
09/16/1962	18-31-17	34.52	119.77	B	2.9			
09/21/1962	05-07-18	34.47	119.58	B	3.0			
09/29/1962	19-47-32	34.47	119.70	B	2.9			
10/13/1962	17-49-39.5	36.35	120.42	B	3.7	17		NORTHEAST OF PRIEST.
12/15/1962	-?-40-20.9	36.47	120.63	B	2.9	13		NORTH OF PRIEST.
01/09/1963	06-04-25.7	35.98	120.35	B	3.2	14	F	SE OF PRIEST; III AT WHEELER RIDGE.
02/09/1963	02-52-14.5	35.98	121.69	C	2.8	8		OFF COAST S OF BIG SUR.
02/12/1963	03-44-30.9	36.50	121.32	B	2.6	10		S OF VINEYARD.
02/22/1963	15-56-21.9	35.11	121.44	C	3.3	15		OFF COAST, SW OF MORRO BAY.
02/22/1963	15-56-36.0	35.67	120.83	D	3.6			
04/04/1963	01--?-58	35.80	121.50		2.5	6		NW OF SAN SIMEON.
04/10/1963	01-38-56.8	36.42	121.05		2.9	11		SW OF LLANADA.
04/11/1963	14-02-31.8	36.20	120.87		2.9	13		NW OF PRIEST.
04/20/1963	16-37-33.0	36.38	120.96		3.0	14		SOUTH OF LLANADA.
05/10/1963	10-17-57.1	36.37	120.98		2.5	9		SOUTH OF LLANADA.
06/01/1963	05-19--0.2	34.33	119.54	B	2.0			
07/02/1963	12--?-24.9	34.86	119.80	C	2.0			
07/04/1963	03-20-41.0	34.77	120.02	C	3.2			
07/06/1963	23-32-30.4	34.78	120.63	B	3.3			
08/15/1963	21-02-32.2	35.97	121.02		3.6	15		NEAR JOLON; FORESHOCK OF FOLLOWING--
08/15/1963	21-21-32.1	35.91	121.06		3.9	18	F	NEAR JOLON; FELT AT HARRIS RANCH.
08/16/1963	08-12-13.6	36.06	121.01		3.2	10		NEAR JOLON; AFTERSHOCK OF PRECEDING.
09/06/1963	03-54-34	36.22	121.48		2.6	8		WEST OF PARAISO.
11/01/1963	14-05-56.0	35.56	120.23		3.4	9		EAST OF ATASCADERO.
11/01/1963	14-06--0.4	35.75	120.47	C	3.2			
11/18/1963	07-31-38.5	36.22	120.30	C	3.5		F	IV 15 MI. NE OF SAN MIGUEL.
11/18/1963	10-54-45.4	36.38	120.32		2.7	11		NE OF COALINGA.
11/19/1963	03-33-09.2	36.42	121.03		2.9	10		SW OF LLANADA.
12/12/1963	17-10-48.5	34.98	119.51	C	3.1			
02/10/1964	05-47-25.0	35.75	120.94		3.9	19		NE OF PASO ROBLES.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
03/20/1964	13-15-51.0	36.40	121.03		2.6	7		SW OF LLANADA.
04/28/1964	15-01-48.3	36.23	121.08		2.8	7		NEAR KING CITY.
05/07/1964	17-53-58.3	36.43	120.54		2.5	5		N OF PRIEST.
06/06/1964	11-47-39.0	34.63	121.40	D	4.3			
06/20/1964	09-21-51.4	34.13	120.67	C	3.1			
07/24/1964	07-09-35.9	36.47	121.18		2.9	15		NE OF PARAISO.
08/30/1964	03-41-10.4	36.29	121.94		2.9	7		OFF COAST NW OF POINT SUR.
09/12/1964	01-45-53.5	36.08	120.49		3.1	10		SE OF PRIEST.
10/17/1964	23-43-22.6	36.21	120.92		3.3	14		NW OF PRIEST.
11/08/1964	01-19-19.0	36.00	120.00		4.0	15		E OF AVENAL.
11/08/1964	13-45-51.1	36.34	121.32		3.1	16		NEAR PARAISO.
11/18/1964	01-47-34.0	35.98	121.13		2.7	8		SW OF KING CITY.
11/25/1964	12-49-41.8	36.21	120.78		2.8	15		NW OF PRIEST.
12/05/1964	13-55-57.5	36.02	121.08		2.6	12		W OF SAN ARDO.
12/11/1964	03-35-38.8	34.24	119.76	B	3.5			
12/25/1964	11-21-13.2	35.97	121.18		2.6	5		N OF LAKE NACIMIENTO.
12/27/1964	18-58-59.4	36.46	121.06		2.6	7		SW OF LLANDA.
01/13/1965	04-20-48.2	36.45	120.58		2.6	9		NE OF PRIEST.
01/26/1965	08-34-30.7	35.72	120.54		3.0	12		SE OF PRIEST.
01/26/1965	08-36-36.6	35.92	120.27	C	3.1			
01/26/1965	08-38-16.4	36.04	120.26	C	3.1			
02/21/1965	18-39-18.3	35.67	120.43		3.1	12		E OF PASO ROBLES.
03/28/1965	02-32-21.0	36.20	120.40		3.5			(USCGS)
04/06/1965	20-49-24.4	35.95	121.46		2.5	7		N OF SAN SIMEON.
04/08/1965	01-05-40.6	36.03	121.40		3.0	10		N OF SAN SIMEON.
04/09/1965	12-50-19.3	36.03	120.64		3.0	10		S OF PRIEST.
04/18/1965	03-58-52.4	36.50	121.23		2.7	7		NEAR PINNACLES NATIONAL MONUMENT
04/24/1965	07-29-47.1	34.91	120.14	C	3.6			
05/12/1965	17-55-08.7	35.49	121.17		3.0	6		SW OF SAN SIMEON.
06/07/1965	15-06-47.6	36.50	121.13		2.5	10		NEAR PINNACLES NATIONAL MONUMENT.
06/20/1965	02-56-43.5	36.33	120.37		2.7	11		N OF COALINGA.
06/30/1965	15-21-27.7	36.35	120.71		2.5	9		N OF PRIEST.
07/23/1965	05-31-52.7	35.71	121.23		3.4	13		N OF SAN SIMEON.
07/24/1965	15-25-57.4	36.36	120.98		2.5	7		SW OF LLANADA.
08/01/1965	06-47-27.3	36.23	120.85		2.5	7		NW OF PRIEST.
08/01/1965	13-28-32.9	36.23	120.84		2.5	6		AFTERSHOCK OF 06-47-27.3.
08/13/1965	07-36-08.4	36.46	121.08		2.6	9		SW OF LLANADA.
08/13/1965	13-46-16.5	34.35	119.63	B	3.7		F	IV AT CARPINTERIA AND SANTA BARBARA.
08/13/1965	21-28-51.8	36.48	121.13		2.4	8		W OF LLANADA.
08/15/1965	23-06-52.5	36.00	120.20		4.0		F	AT PAICINES.
08/21/1965	20-09-35.4	36.46	121.07		2.5	8		SW OF LLANADA.
09/06/1965	18--?-57.8	35.96	120.36	C	3.4			
09/12/1965	08-50-05.5	36.49	121.12		2.5	7		W OF LLANADA.
09/19/1965	15-42-07.8	35.98	120.34	C	4.8		F	V AT ARMONA, AVENAL, CHOLAME, KETTLEMAN CITY, AND STRATFORD.
10/22/1965	02-29-22	36.00	121.70		2.7	6		OFF COAST, W OF KING CITY.
12/02/1965	22-29-13.0	36.20	121.68		2.8	9		W OF PARAISO.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
01/28/1966	01-49-47.4	35.83	120.45		3.0		F	PARKFIELD SEQUENCE; MC EVILLY, ET AL, (1967) THE PARKFIELD, CALIFORNIA EARTHQUAKE OF 1966, BULL. SEISM. SOC. AM.
02/01/1966	-?-20-44.3	36.03	120.57		2.9			PARKFIELD SEQUENCE - SEE 01/28/1966 AT 01-49-47.4.
02/14/1966	-?-24-03.9	36.02	120.57		2.4			PARKFIELD SEQUENCE - SEE 01/28/1966 AT 01-49-47.4.
02/25/1966	01-34-38.0	36.05	120.63		2.4			PARKFIELD SEQUENCE - SEE 01/28/1966 AT 01-49-47.4.
03/31/1966	21-38-45.2	36.05	120.60		2.5			PARKFIELD SEQUENCE - SEE 01/28/1966 AT 01-49-47.4.
04/05/1966	20-44-58.7	36.24	120.85		2.7	9		10 KM NW OF PRIEST (UC BERKELEY SEISMOGRAPH STATION (SS)).
04/12/1966	15-31-39.8	36.07	120.70		2.3			PARKFIELD SEQUENCE.
05/11/1966	17-37-01.1	35.98	120.57		2.3			PARKFIELD SEQUENCE.
05/23/1966	08-07-37.6	36.02	120.57		2.5			PARKFIELD SEQUENCE.
05/23/1966	08-11-07.0	36.02	120.57		2.2			PARKFIELD SEQUENCE.
05/27/1966	15-36-03.7	35.98	120.49		2.7			PARKFIELD SEQUENCE.
06/18/1966	16-32-17.6	35.96	120.53		2.0			PARKFIELD SEQUENCE.
06/20/1966	23-19-18.8	36.33	120.96		2.8	9		NE OF KING CITY.
06/24/1966	21-42-50.4	36.50	120.85		3.1	10		SE OF LLANADA.
06/28/1966	01--?-31.5	35.95	120.52		3.1		F	PARKFIELD SEQUENCE; FELT AT CHOLAME, PARKFIELD, VALLETON, AND WORK RANCH.
06/28/1966	01-14-55	35.95	120.50		1.8			PARKFIELD SEQUENCE.
06/28/1966	04-08-55.2	35.97	120.50		5.1			PARKFIELD SEQUENCE FIRST MAIN SHOCK (FELT REPORTS FOR THE 2 MAIN SHOCKS ARE NOT SEPARATED.) FELT OVER 20,000 SQ. MI., MINOR SURFACE FAULTING ALONG SAN ANDREAS FAULT FROM PARKFIELD TO CHOLAME (20 MI.), MAXIMUM DISPLACEMENT 4 IN. VII AT CHOLAME AND PARKFIELD, VI AT ANNETTE, BITTERWATER VALLEY, COALINGA, HIDDEN VALLEY RANCH, PASO ROBLES, SAN LUIS OBISPO, SAN MIGUEL, SHAFTER, SHANDON, SLACK CANYON, VALLETON, WAITI RANCH, AND WORK RANCH, AND V AT ADELAIDA, ALPAUGH, ARROYO GRANDE, ATASCADERO, AVILA BEACH, BAKERSFIELD, BAYWOOD PARK, BRYSON, BURREL, BUTTONWILLOW, EARLIMART, FELLOWS, FRAZIER PARK, GREENFIELD, HARMONY, INDIAN VALLEY, KETTLEMAN CITY, KING CITY, LAPANZA, LOST MARICOPA, MEE RANCH, MORRO BAY, MOSS LANDING, MUSICK, NIPOMO, OCEANO, OLD RIVER, PANOCHE, PINE CANYON, PISMO BEACH, POZO, PRIEST VALLEY, SAN ARDO, SAN JOAQUIN, SAN LUCAS, SAN SIMEON, SIMMLER, STRATFORD, TEMPLETON, AND VANDENBURG A.F.B.
06/28/1966	04-09-53	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-18-34.0	35.95	120.53		2.6		F	PARKFIELD SEQUENCE - FELT AT CANTUA CREEK AND SOQUEL.
06/28/1966	04-26-13.4	35.95	120.50		5.5		F	PARKFIELD SEQUENCE - SECOND MAIN SHOCK.
06/28/1966	04-26-28	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-26-34	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-27-37	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-27-58	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-28-19	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-28-36	35.95	120.50		4.5			PARKFIELD SEQUENCE.
06/28/1966	04-28-46	35.95	120.50					PARKFIELD SEQUENCE.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/28/1966	04-29-13	35.95	120.50					PARKFIELD SEQUENCE.
06/28/1966	04-31-55	35.95	120.50		3.0			PARKFIELD SEQUENCE.
06/28/1966	04-32-50	35.95	120.50		3.5		F	PARKFIELD SEQUENCE - FELT AT CANTUA CREEK, CHOLAME, AND HERNANDEZ.
06/28/1966	04-34-59.1	35.81	120.40		3.0			PARKFIELD SEQUENCE.
06/28/1966	04-39-08.1	35.95	120.50		3.0		F	PARKFIELD SEQUENCE - FELT AT PARKFIELD AND WORK RANCH.
06/28/1966	04-42-33.6	35.83	120.38		2.4			PARKFIELD SEQUENCE.
06/28/1966	04-43-54.8	35.95	120.57		2.7			PARKFIELD SEQUENCE.
06/28/1966	04-46-22	35.95	120.50		3.0			PARKFIELD SEQUENCE.
06/28/1966	04-51-43	35.95	120.50		2.4			PARKFIELD SEQUENCE.
06/28/1966	05--?-59.5	35.85	120.40		3.1			PARKFIELD SEQUENCE.
06/28/1966	05-03-44.7	35.88	120.45		2.4			PARKFIELD SEQUENCE.
06/28/1966	05-09-48.3	35.83	120.13		2.5			PARKFIELD SEQUENCE.
06/28/1966	05-12-42.5	35.92	120.47		2.9			PARKFIELD SEQUENCE.
06/28/1966	05-17-05	35.95	120.50		2.1			PARKFIELD SEQUENCE.
06/28/1966	05-21-05	35.95	120.50		2.0			PARKFIELD SEQUENCE.
06/28/1966	05-29-14.9	35.92	120.48		2.1			PARKFIELD SEQUENCE.
06/28/1966	05-37-04.6	35.88	120.44		2.5			PARKFIELD SEQUENCE.
06/28/1966	05-40-19.4	35.94	120.48		2.7			PARKFIELD SEQUENCE.
06/28/1966	05-45-59.1	35.75	120.33		3.2			PARKFIELD SEQUENCE.
06/28/1966	05-48-26	35.95	120.50		2.2			PARKFIELD SEQUENCE.
06/28/1966	05-51-34.0	35.86	120.44		2.1			PARKFIELD SEQUENCE.
06/28/1966	05-52-06	35.95	120.50		2.3			PARKFIELD SEQUENCE.
06/28/1966	05-52-58	35.95	120.50		2.4			PARKFIELD SEQUENCE.
06/28/1966	05-56--?	35.95	120.50		2.1			PARKFIELD SEQUENCE.
06/28/1966	06-11-03.5	35.81	120.35		2.6			PARKFIELD SEQUENCE.
06/28/1966	06-32-17.9	35.94	120.52		3.4		F	PARKFIELD SEQUENCE - FELT AT CHOLAME, COALINGA, AND PARKFIELD.
06/28/1966	06-35-11.4	35.80	120.38		3.0			PARKFIELD SEQUENCE.
06/28/1966	06-39-31.2	35.90	120.47		2.2			PARKFIELD SEQUENCE.
06/28/1966	07-01-03.8	35.92	120.48		2.2			PARKFIELD SEQUENCE.
06/28/1966	07-33-52.7	35.90	120.45		2.7			PARKFIELD SEQUENCE.
06/28/1966	07-41-43	35.95	120.50		2.3			PARKFIELD SEQUENCE.
06/28/1966	07-45-48.3	35.90	120.47		3.0		F	PARKFIELD SEQUENCE - FELT AT CHOLAME AND PARKFIELD.
06/28/1966	08-14-48.6	35.83	120.42		2.4			PARKFIELD SEQUENCE.
06/28/1966	08-47-52.4	35.85	120.42		2.0			PARKFIELD SEQUENCE.
06/28/1966	08-54-49.5	35.92	120.50		2.3			PARKFIELD SEQUENCE.
06/28/1966	08-59-52.3	35.85	120.42		2.5			PARKFIELD SEQUENCE.
06/28/1966	09-31-26.5	35.77	120.35		2.4			PARKFIELD SEQUENCE.
06/28/1966	09-35-54.3	35.77	120.36		2.2			PARKFIELD SEQUENCE.
06/28/1966	09-56-09.7	35.83	120.40		2.5			PARKFIELD SEQUENCE.
06/28/1966	10-15-53.3	35.92	120.53		2.1			PARKFIELD SEQUENCE.
06/28/1966	10-20-16.4	35.85	120.42		2.3			PARKFIELD SEQUENCE.
06/28/1966	10-23-22.8	35.55	120.42		2.0			PARKFIELD SEQUENCE.
06/28/1966	10-23-22.8	35.94	120.48		2.5			PARKFIELD SEQUENCE.
06/28/1966	10-46-22.9	35.94	120.50		2.0			PARKFIELD SEQUENCE.
06/28/1966	11-15-13.9	35.85	120.42		2.0			PARKFIELD SEQUENCE.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/28/1966	11-28-41.4	35.85	120.38		2.0			PARKFIELD SEQUENCE.
06/28/1966	11-30-14.0	35.90	120.47		2.2			PARKFIELD SEQUENCE.
06/28/1966	12-31-52.1	35.94	120.48		2.5			PARKFIELD SEQUENCE.
06/28/1966	12-52-22.0	35.97	120.53		2.3			PARKFIELD SEQUENCE.
06/28/1966	13-48-22	35.97	120.53		2.7			PARKFIELD SEQUENCE.
06/28/1966	14-13-09.3	35.94	120.48		2.6			PARKFIELD SEQUENCE.
06/28/1966	14-21-36.3	35.94	120.48		2.2			PARKFIELD SEQUENCE.
06/28/1966	14-51-53.6	35.90	120.47		2.3			PARKFIELD SEQUENCE.
06/28/1966	18-12-19.4	35.92	120.50		2.3			PARKFIELD SEQUENCE.
06/28/1966	18-22-32.4	35.92	120.50		2.0			PARKFIELD SEQUENCE.
06/28/1966	18-54-55.3	35.88	120.45		2.5			PARKFIELD SEQUENCE.
06/28/1966	19-59-37.8	35.92	120.47		2.8			PARKFIELD SEQUENCE.
06/28/1966	20--?-38.7	35.92	120.48		2.5			PARKFIELD SEQUENCE.
06/28/1966	20-46-56.4	35.77	120.40		3.1		F	PARKFIELD SEQUENCE - FELT AT BAR B RANCH AND WORK RANCH.
06/28/1966	22-01-13.9	35.85	120.44		2.0			PARKFIELD SEQUENCE.
06/28/1966	22-37-56.7	35.88	120.42		2.0			PARKFIELD SEQUENCE.
06/28/1966	23-57-22.3	35.77	120.35		2.5			PARKFIELD SEQUENCE.
06/29/1966	-?-17-32.6	35.85	120.44		2.3			PARKFIELD SEQUENCE.
06/29/1966	02-19-39.9	35.92	120.52		3.6		F	PARKFIELD SEQUENCE - FELT AT CHOLAME, PARKFIELD, AND WORK RANCH.
06/29/1966	04-06-40.3	35.92	120.53		2.8			PARKFIELD SEQUENCE.
06/29/1966	07-28-59.4	35.92	120.48		2.3			PARKFIELD SEQUENCE.
06/29/1966	08-55-52.4	35.88	120.45		2.9			PARKFIELD SEQUENCE.
06/29/1966	09-20-50.1	35.78	120.36		2.5			PARKFIELD SEQUENCE.
06/29/1966	10-13-44.0	35.97	120.50		2.3			PARKFIELD SEQUENCE.
06/29/1966	10-56-58.8	35.75	120.33		3.0			PARKFIELD SEQUENCE.
06/29/1966	12-30-09.0	35.94	120.50		2.4			PARKFIELD SEQUENCE.
06/29/1966	13-11-59.7	35.82	120.38		3.1		F	PARKFIELD SEQUENCE - FELT AT CHOLAME AND PARKFIELD.
06/29/1966	15-18-38.9	35.95	120.33		2.0			PARKFIELD SEQUENCE.
06/29/1966	15-34-22.2	35.92	120.48		2.3			PARKFIELD SEQUENCE.
06/29/1966	16-03-30.1	35.86	120.45		2.1			PARKFIELD SEQUENCE.
06/29/1966	17-10-28.3	35.82	120.36		2.0			PARKFIELD SEQUENCE.
06/29/1966	19-53-25.9	35.95	120.53		5.0		F	PARKFIELD SEQUENCE - FELT AT ADELAIDA, BITTERWATER, CHOLAME, COALINGA, FRESNO, MEE RANCH, MORRO BAY, SAN LUIS OBISPO, SAN MIGUEL, SANTA MARGARITA, SHANDON, AND WORK RANCH.
06/29/1966	20-44-40.0	35.74	120.28		2.5			PARKFIELD SEQUENCE.
06/29/1966	23-48-12.0	35.74	120.28		2.3			PARKFIELD SEQUENCE.
06/30/1966	01-17-36.1	35.86	120.45		4.1			PARKFIELD SEQUENCE.
06/30/1966	03-36-16.8	35.92	120.47		2.6			PARKFIELD SEQUENCE.
06/30/1966	05-04-12.9	35.88	120.45		2.0			PARKFIELD SEQUENCE.
06/30/1966	06-07-21.5	35.94	120.48		2.4			PARKFIELD SEQUENCE.
06/30/1966	06-23-32.4	35.90	120.47		2.1			PARKFIELD SEQUENCE.
06/30/1966	07-37-12.1	35.90	120.47		2.0			PARKFIELD SEQUENCE.
06/30/1966	08-01-38.4	35.90	120.47		2.9			PARKFIELD SEQUENCE.
06/30/1966	11-07-55.1	35.78	120.33		2.8			PARKFIELD SEQUENCE.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/30/1966	13-26-05.7	35.78	120.35		2.3			PARKFIELD SEQUENCE.
06/30/1966	13-29-56.6	35.86	120.40		2.0			PARKFIELD SEQUENCE.
06/30/1966	13-40-50.9	35.83	120.38		2.1			PARKFIELD SEQUENCE.
06/30/1966	16-05-02.7	35.97	120.50		2.3			PARKFIELD SEQUENCE.
06/30/1966	19-06-17.5	35.86	120.42		2.1			PARKFIELD SEQUENCE.
07/01/1966	09-41-21.9	35.94	120.52	3.2			F	PARKFIELD SEQUENCE - FELT AT WORK RANCH.
07/02/1966	12-08-34.8	35.79	120.33		3.7		F	PARKFIELD SEQUENCE - FELT AT PARKFIELD.
07/02/1966	12-16-15.8	35.81	120.35		3.4		F	PARKFIELD SEQUENCE - FELT AT PARKFIELD.
07/02/1966	12-25-06.8	35.80	120.35		3.1		F	PARKFIELD SEQUENCE - FELT AT PARKFIELD.
07/05/1966	18-54-54.5	35.92	120.48		3.0		F	PARKFIELD SEQUENCE - FELT AT PARKFIELD.
07/25/1966	22-49-39	36.40	120.30		2.5	4		NE OF COALINGA.
07/27/1966	08-12- 0.2	35.90	120.48		3.0			PARKFIELD SEQUENCE.
08/03/1966	12-39-05.8	35.80	120.38		3.4		F	PARKFIELD SEQUENCE; V AT CHOLAME, PARKFIELD, AND WORK RANCH.
08/04/1966	-?-54-24.5	35.74	121.35		3.0	8		NW OF SAN SIMEON.
08/07/1966	17-03-24.9	35.94	120.55		3.0			PARKFIELD SEQUENCE.
08/19/1966	22-51-20.1	35.90	120.45		3.3			PARKFIELD SEQUENCE.
09/07/1966	-?-20-50.5	35.83	119.94		3.2	9		SE OF COALINGA.
09/18/1966	15-09-55.7	35.74	120.35		3.1			PARKFIELD SEQUENCE.
10/27/1966	12-06-03.9	35.94	120.50		3.8		F	PARKFIELD SEQUENCE; V AT ATASCADERO, AVENAL, COALINGA, PARKFIELD, SAN MIGUEL, TEMPLETON, AND WORK RANCH.
11/05/1966	13-31-31.2	35.94	120.50		3.3			PARKFIELD SEQUENCE.
11/18/1966	23-39-42.3	35.75	120.33		3.3			PARKFIELD SEQUENCE.
12/30/1966	10-23-48	36.47	120.40		2.5	4		N OF COALINGA.
01/08/1967	23-03-50.9	35.90	120.40		2.8	8		35 KM SE OF PRIEST (UC BERKELEY SS).
01/09/1967	23-18-59.5	35.86	120.10		3.1	9		SE OF COALINGA.
02/01/1967	13-55-54.1	35.70	120.25		3.0	8		NE OF SAN LUIS OBISPO.
02/26/1967	15-17-53.9	36.40	121.06		2.5	9		SW OF LLANADA.
03/13/1967	21-59-48.4	36.00	120.61		3.1	8	F	15 KM S OF PRIEST (UC BERKELEY SS). IV AT SAN MIGUEL; FELT AT INDIAN VALLEY AND RANCHITO CANYON.
03/21/1967	02-24-28.3	36.21	120.85		2.8	8		17 KM NW OF PRIEST (UC BERKELEY SS).
03/23/1967	11-39-56.4	36.16	120.18		3.0	5		20 KM E OF COALINGA.
04/13/1967	09-06-42.5	36.15	120.80		2.7	8		13 KM W OF PRIEST (UC BERKELEY SS).
05/17/1967	14-16-52.2	35.95	120.73		3.0	6		30 KM S OF PRIEST (UC BERKELEY SS).
06/03/1967	20-10-53.0	35.71	121.48		2.6	7		OFF COAST NW OF SAN SIMEON.
06/06/1967	06-11-38.5	35.81	120.43		3.0	10	F	40 KM SE OF PRIEST (UC BERKELEY SS); IV AT WORK RANCH; FELT IN INDIAN VALLEY, SOUTHERN MONTEREY COUNTY, AND VINEYARD CANYON.
06/13/1967	12-54-10.7	35.81	121.50		3.3	10		OFF COAST, 35KM NW OF SAN SIMEON.
07/24/1967	07-08-52.9	35.96	120.50		3.7	9		PARKFIELD AREA.
07/28/1967	14-44-40.1	35.75	121.38		3.0	6		NEAR SAN SIMEON.
08/01/1967	22-14-13.0	35.75	121.40		2.7	6		NW OF SAN SIMEON.
08/08/1967	18-11-20.3	36.42	120.42		2.5	7		N OF COALINGA.
08/12/1967	18-57-40.4	35.80	120.45		4.1	18	F	PARKFIELD AREA; V AT ESTRELLA AREA, HOG CANYON ROAD TO PARKFIELD, AND SHANDON, AND IV AT CHOLAME.

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
08/12/1967	23-21-07.8	36.11	120.80		2.8	6		SE OF KING CITY.
08/12/1967	23-22-05.3	36.13	120.76		2.5	7		SE OF KING CITY.
08/17/1967	23-12-02.7	35.91	121.50		2.6	5		NW OF SAN SIMEON.
08/25/1967	02-28-14.4	35.81	121.27		2.7	6		NW OF SAN SIMEON.
08/25/1967	16-35-27.8	36.05	120.00		3.2	7		SE OF COALINGA.
08/25/1967	16-40-50.2	36.01	119.95		3.0	7		SE OF COALINGA.
08/31/1967	18-10-40.4	35.86	121.35		2.8	7		NW OF SAN SIMEON.
09/09/1967	21-35-05.6	35.81	121.63		2.4	3		OFF SHORE SAN SIMEON.
10/14/1967	12-02-43.6	36.50	120.61		2.7	5		NEAR MT. CIERVO.
10/21/1967	12-05-21.8	35.83	120.46		3.1	7		PARKFIELD AREA.
10/25/1967	23-05-30.5	35.73	121.45		2.6	4		NEAR SAN SIMEON.
11/11/1967	22-10-06.8	36.50	120.81		3.3	9		S OF PANOCHE VALLEY.
11/11/1967	22-33-47.5	36.48	120.78		2.8	6		S OF PANOCHE VALLEY.
11/12/1967	07-11-20.4	36.48	120.80		2.6	7		S OF PANOCHE VALLEY.
11/14/1967	-?--?-51.7	35.78	120.53		3.1	3		PARKFIELD.
11/25/1967	15-27-43.4	36.46	121.06		2.5	6		BEAR VALLEY.
12/21/1967	05-13-11.3	35.36	120.85		2.6	3		S OF SAN SIMEON.
12/21/1967	19-08-53.8	35.91	119.53		3.1	5		NW OF DELANO.
12/21/1967	23-58-60.2	35.93	120.56		3.0	3		PARKFIELD.
12/31/1967	23-48-13.5	35.75	120.45		4.3	3	F	PARKFIELD AREA; V AT CRESTON, PARKFIELD, SALINAS DAM, SAN MIGUEL, SHANDON, TEMPLETON, AND WORK RANCH.
02/03/1968	19-07-26.4	35.73	121.25		2.8	5		NEAR SAN SIMEON.
02/23/1968	20-20-57.9	35.86	121.31		2.5	7		EAST OF HOLLISTER.
03/25/1968	11-32-07.4	36.37	120.70		3.6	8	F	SE OF LLANADA; MAXIMUM INTENSITY V.
03/28/1968	04-53-26.5	36.36	120.19		3.1	5	F	SE OF COALINGA; FELT AT AVENAL - INTENSITY IV.
04/14/1968	06-20-54.6	36.18	121.65		2.5	6		SE OF MONTEREY.
04/23/1968	15-09-14.9	35.52	120.82		3.4	7		SE OF SAN SIMEON.
04/27/1968	14-32-37.4	36.22	120.83		2.7	7		NW OF PRIEST (UC BERKELEY SS).
04/28/1968	06-31-32.9	35.46	120.83		3.5	7		NW OF SAN LUIS OBISPO.
05/31/1968	07-07-37.9	35.80	120.60		3.0	5		S OF COALINGA.
06/11/1968	11-43-28.1	35.90	121.70		3.3	9		OFFSHORE, NW OF SAN SIMEON.
06/22/1968	12-50-50.1	36.43	121.04		2.9	9		S OF LLAN
07/03/1968	17-52-52	35.80	121.50		2.5	7		NW OF SAN SIMEON.
07/29/1968	04-27-51.9	36.38	120.69		2.7	9		N OF PRIEST (UC BERKELEY SS).
07/29/1968	05-29-19.9	36.37	120.70		2.8	9		N OF PRIEST (UC BERKELEY SS).
07/31/1968	-?-49-25.4	36.37	120.70		2.9	9		N OF PRIEST (UC BERKELEY SS).
08/19/1968	16-30-18.2	36.40	121.91		3.3	9		S OF CARMEL.
09/01/1968	21-56-24.4	36.45	121.02		2.7	8		E OF PINNACLES NATIONAL MONUMENT.
11/06/1968	08-58-23.2	35.88	120.45		2.8	10	F	NEAR PARKFIELD; FELT NEAR SAN MIGUEL.
11/10/1968	04-06-03.9	35.70	121.18		3.2	9		NEAR SAN SIMEON.
11/17/1968	01-03-47.0	36.29	120.94		3.0	6		NEAR KING CITY.
12/11/1968	12-19-52.4	35.81	120.48		3.0	10		NEAR PARKFIELD.
12/16/1968	01-14-10.9	36.17	120.85		2.7	7		W OF PRIEST (UC BERKELEY SS).
01/09/1969	09-42-47.2	35.94	120.57		3.8	7	F	CHOLAME VALLEY; FELT IN PARKFIELD AND SLACK CANYON - MAXIMUM INTENSITY V.
02/04/1969	-?-45-25	36.40	120.38		3.0	4		NORTH OF COALINGA.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
06/19/1969	07-05-08	36.12	119.58		3.5	8	F	NEAR TULARE; FELT IN CORCORAN, DINUBA, HANFORD, IVANHOE, LEMON COVE, STRATHMORE, AND TIPTON. MAXIMUM INTENSITY IV.
06/24/1969	14-25-37	36.42	120.13		3.0	4		SOUTHWEST OF FRESNO.
07/16/1969	04-06-35	35.83	120.28		3.2	7		15 KM SOUTHEAST OF PARKFIELD.
09/06/1969	13-44-45	35.30	121.10		3.8	10	F	50 KM WEST OF SAN LUIS OBISPO.
09/16/1969	03-32-24	36.18	120.80		2.5	8		13 KM WEST OF PRIEST (UC BERKELEY SS).
10/02/1969	06--?-58.9	36.32	120.32		3.3	10		10 KM NORTH OF COALINGA.
11/17/1969	20-49-10.4	36.43	121.05		4.4	10	F	NNE OF KING CITY; FELT IN MONTEREY - SWAYED BUILDINGS IN SALINAS
11/19/1969	06-23-50	36.45	121.52		4.2	8	F	GONZALES AND SALINAS VALLEY; FELT IN SALINAS AND SANTA CRUZ - RATTLED WINDOWS IN MONTEREY.
11/26/1969	-?-06-59	36.48	120.60		2.5	6		50 KM NORTHEAST OF KING CITY.
11/30/1969	15-11-54	35.30	120.90		2.5	10		20 KM EAST OF KING CITY; 2 SMALL FORESHOCKS RECORDED.
12/10/1969	13-25-31	35.75	120.40		3.5	7		40 KM SOUTH OF COALINGA.
12/14/1969	19-07-57	35.92	120.68		3.2	9		20 KM NORTH OF PASO ROBLES.
01/29/1970	02-49-12.9	36.11	120.99		2.5	6		20 KM SOUTHWEST OF KING CITY.
02/01/1970	21-19-45.7	36.41	121.08		2.6	12		30 KM EAST OF PARAISO.
02/08/1970	-?-14-13.3	36.40	120.97		2.7	13		25 KM SOUTH OF LLANADA.
02/09/1970	16--?-46.1	35.77	120.35		3.1	16		60 KM SOUTH OF PRIEST (UC BERKELEY SS).
02/14/1970	15-44-58.0	36.09	120.64		2.8	14		5 KM SOUTH OF PRIEST (UC BERKELEY SS).
04/18/1970	13-16-53.4	36.49	120.01		3.0	15		35 KM SOUTHWEST OF FRESNO.
04/21/1970	22-29-25.9	35.66	120.43		3.0	8		65 KM SOUTH OF PRIEST (UC BERKELEY SS).
04/23/1970	03-25-18.9	35.97	121.45		2.5	10		25 KM SOUTHWEST OF KING CITY.
05/27/1970	10-42-19.3	35.99	120.91		3.4	8		40 KM SOUTHWEST OF PRIEST (UC BERKELEY SS).
07/20/1970	23-24-55	35.95	121.57		2.5	5		8 KM SOUTH OF LOPEZ POINT - OFFSHORE.
07/21/1970	05-24-16.1	35.99	121.57		2.5	5		5 KM SOUTHEAST OF LOPEZ POINT.
08/05/1970	06-47-36.4	35.82	119.94		2.9	8		KETTLEMAN HILLS.
08/05/1970	16-51-45.7	36.23	121.69		3.0	11		25 KM SOUTHWEST OF PARAISO.
08/13/1970	05-06-19.8	36.17	121.70		3.7	11		20 KM WEST OF LOPEZ POINT.
09/05/1970	11-29-11	36.20	120.10		3.1	4		EAST-NORTHEAST OF COALINGA.
09/10/1970	23-45-59	36.40	120.50		3.2	11		30 KM NORTHWEST OF COALINGA.
09/11/1970	15-20-08	35.98	120.05		3.3	9		8 KM EAST OF AVENAL.
09/16/1970	18-22-10.7	35.96	121.27		2.6	7		NEAR MILPITAS.
10/07/1970	17-57-06.3	36.30	121.40		2.5	9		30 KM NORTHWEST OF KING CITY.
12/01/1970	06-05-59	35.38	121.13		3.3	7	F	25 KM WEST OF MORRO BAY; INTENSITY V AT BRYSON - NO DAMAGE.
12/12/1970	22-29-20	35.65	121.55		2.5	6		30 KM WEST OF SAN SIMEON.
01/02/71	06-27-37.5	35°55.1'	120°32.2'		3.0			10 km NW of Parkfield
01/16/71	05-33-27.8	36°00'	120°12'		3.1			Kettleman Hills
01/26/71	21-53-53	35°12'	120°42'		3.0			Near San Luis Obispo.
01/31/71	12-22-49.5	35°55.6'	120°30.6'		3.0			NW of Parkfield; sharp, rapid jolting at Shandon.
04/05/71	01-40-34.2	36°24.8'	120°59.0'		3.0			20 km SE of Pinnacles National Monument.
04/19/71	09-35-58.8	36°13.7'	120°50.3'		3.0			25 km E of King City.
04/29/71	02-13-15.7	36°30.3'	120°32.5'		3.0			40 km NW of Coalinga.
06/20/71	12-41-39.8	35°3'	120°20'		3.4			Near Cholame.
07/06/71	09-24-35	35°34'	121°35'		3.0			SW of San Simeon.

TABLE 2.5-1

MM/DD/YY	HR/MN/SE	NORTH LAT	WEST LONG	QUALITY	MAG.	STA. REC.	FELT	MAXIMUM INTENSITY - COMMENTS
07/21/71	09-14-26.2	36°13.7'	120°50.8'		3.2			Near Coalinga.
08/06/71	20-03-16.3	36°00.8'	120°02.2'		3.0			Near Coalinga.
10/06/71	14-43-30.6	35°51.3'	120°22.5'		3.5			S of Coalinga; intensity IV at Cholame, Parkfield, and Shandon.
10/21/71	22-09-45.4	35°58.8'	120°50.2'		3.7			SE of King City; intensity V at San Ardo (small objects shifted) and intensity IV at Jolon, King City, Lockwood, Pine Canyon, and San Lucas.
11/07/71	14-03-30.4	35°31.2'	119°50.2'		4.0			SE of Coalinga.
11/18/71	04-03-52.4	36°14.5'	120°50.6'		3.4			NE of King City.
11/30/71	09-45-42.8	36°03.6'	119°53.4'		3.0			SE of Coalinga.

END OF SELECTED EARTHQUAKES

END OF QUAKES PROGRAM FOR SELECTION OF EARTHQUAKES

TABLE 2.5-2

SUMMARY, REVISED EPICENTERS OF REPRESENTATIVE SAMPLES OF EARTHQUAKES OFF THE COAST OF CALIFORNIA NEAR SAN LUIS OBISPO

Date	Event Number	Lat. (Original Hypocenter / Revised Hypocenter)	Long. (Original Hypocenter / Revised Hypocenter)	Distance Hypocenter Moved, km	Error Ellipse km	Mag., M_L
May 27, 1935	1	35.370 35.621	120.960 121.639	66NW	7 x 14	3.0
Sept. 7, 1939	6	35.420 35.459	121.070 121.495	40W	8 x 8	3.0
Oct. 6, 1939	7	35.800 36.232	121.500 121.763	54NW	16 x 31	3.5
July 11, 1945	8	35.670 35.809	121.250 121.408	21NW	7 x 24	4.0
Mar. 23, 1947	12	35.150 34.577	121.300 121.137	66S	12 x 24	3.7
Mar. 27, 1947	15	35.000 34.739	121.000 120.896	32SW	20 x 20	4.2
Dec. 20, 1948	9	35.800 35.683	121.500 121.364	16SE	9 x 38	4.5
Dec. 31, 1948	10	35.670 35.598	121.400 121.226	17SE	8 x 29	4.6
Nov. 22, 1952 Bryson Earthquake	17	35.730 35.830 35.836	121.190 121.170 121.204	U.C. Berkeley Richter (1969) 12N	7 x 24	6.0
Mar. 13, 1954	21	35.000 34.960	120.690 120.490	19E	9 x 18	3.4
Mar. 5, 1955	23	35.600 35.863	121.400 121.149	38NE	15 x 29	3.3
June 21, 1957	25A	35.100 35.255	120.900 120.951	15NW	10 x 19	3.7
Jan. 2, 1960	26	35.400 35.778	121.190 121.066	44NE	15 x 29	4.0
Feb. 1, 1962	52	34.880 35.031	120.670 120.846	22NW	6 x 16	4.5

Date	Event Number	Original Hypocenter / Revised Hypocenter Lat.	Long.	Distance Hypocenter Moved, km	Error Ellipse km	Mag., M_L
Mar. 5, 1962	54	34.600 34.622	121.590 121.416	17E	8 x 10	4.5
Mar. 10, 1962	54A	34.600 34.667	121.590 121.372	22NE	6 x 20	4.2
Feb. 22, 1963	28	35.110 34.730	121.440 121.400	42S	7 x 28	3.3
Sept. 6, 1969	31	35.300 35.355	121.090 121.033	9NE	5 x 10	3.6
Oct. 22, 1969	56	34.830 34.649	121.340 121.471	23SW	14 x 50	5.4

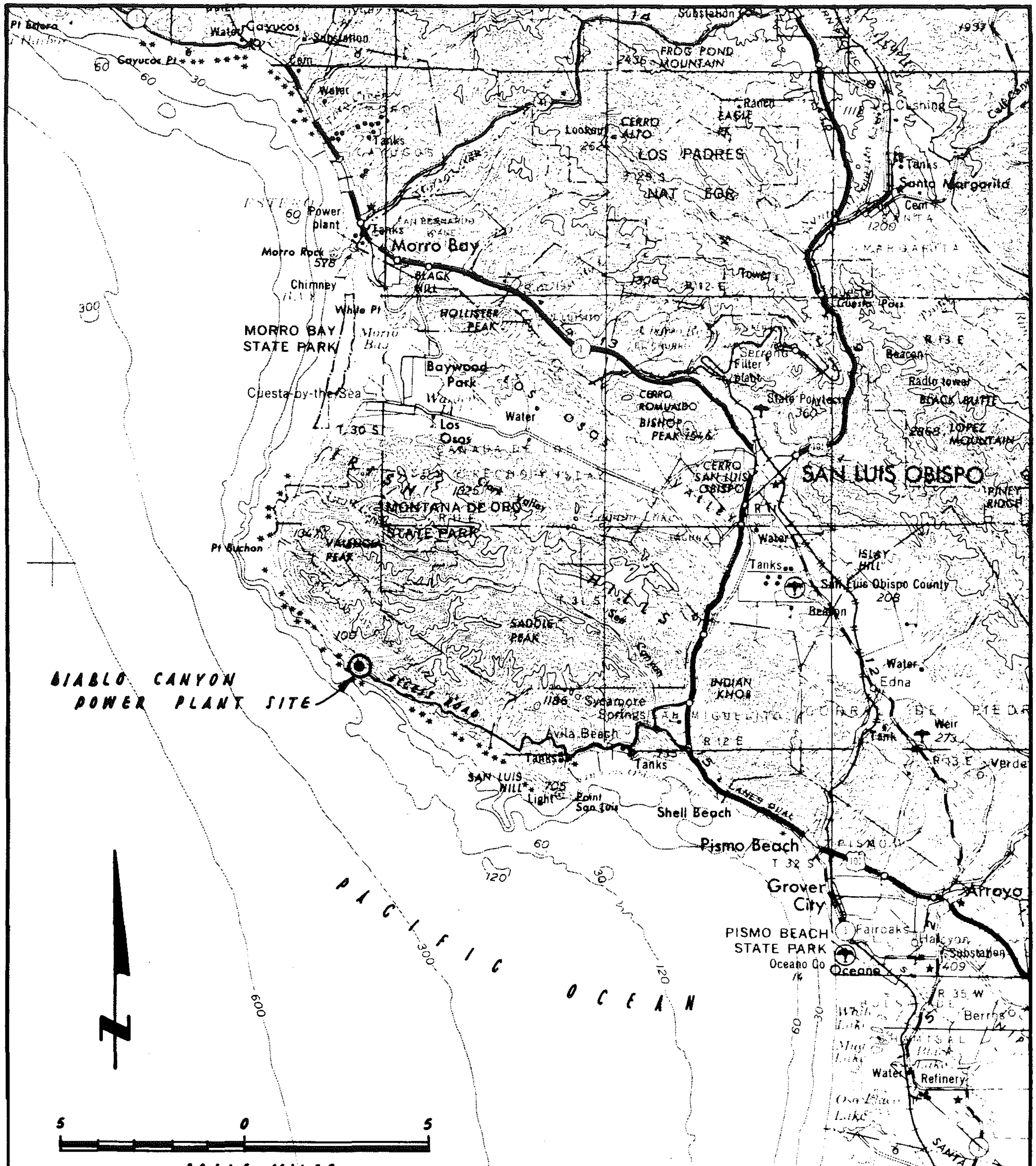
TABLE 2.5-3

DISPLACEMENT HISTORY OF FAULTS IN THE SOUTHERN COAST RANGES OF CALIFORNIA

Fault	Distance From Diablo Site, miles	Time of Principal Activity	Youngest Formation Cut By Fault	Oldest Formation Capping Fault
San Andreas	45	Mid-Tertiary - present		Currently active
Faults in ground between San Andreas and Sur-Nacimiento-Rinconada, La Panza, Cuyama, Red Hills, East Huasna	18-45	Tertiary	Pleistocene (possible Holocene) (Ref. 14)	Not Known
Sur-Nacimiento (zone)	18	Late Mesozoic, (Benioff-subduction zone)	Pleistocene (possible Holocene) (Ref. 14)	Late Quaternary terrace deposits (Ref. 11)
West Huasna-Suey	11	Late Tertiary	Post late-Miocene	Late Quaternary terrace deposits (Ref. 36)
Edna	4.5	Late Tertiary	Plio-Pleistocene (Paso Robles Fm)	Late Pleistocene (Ref. 20)
Miguelito	5	Late Tertiary	Early Pliocene (Miguelito Member of Careaga Fm) (Ref. 21)	Poss. capped by mid-Pliocene Squire Member of Careaga Fm; Plio-Pleistocene Paso Robles Fm

TABLE 2.5-3

Fault	Distance From Diablo Site, miles	Time of Principal Activity	Youngest Formation Cut By Fault	Oldest Formation Capping Fault
Faulting in the Mesozoic rocks near Pt. San Luis	4	Mesozoic	Mesozoic	Late Pleistocene (Ref. 20)
Unnamed faults near Pt. San Simeon	35	Probable Tertiary	Not known; possible Holocene	Not known
Offshore structural zone	4.5	Late Tertiary	Possible Holocene (Ref. 19) (northern part)	Holocene-upper Pliocene (Ref. 19) (southern part)
Faults in the Santa Maria Basin	40	Not known	Possible Pleistocene (orcutt Fm) (Ref. 23)	Pleistocene-Holocene



FSAR UPDATE

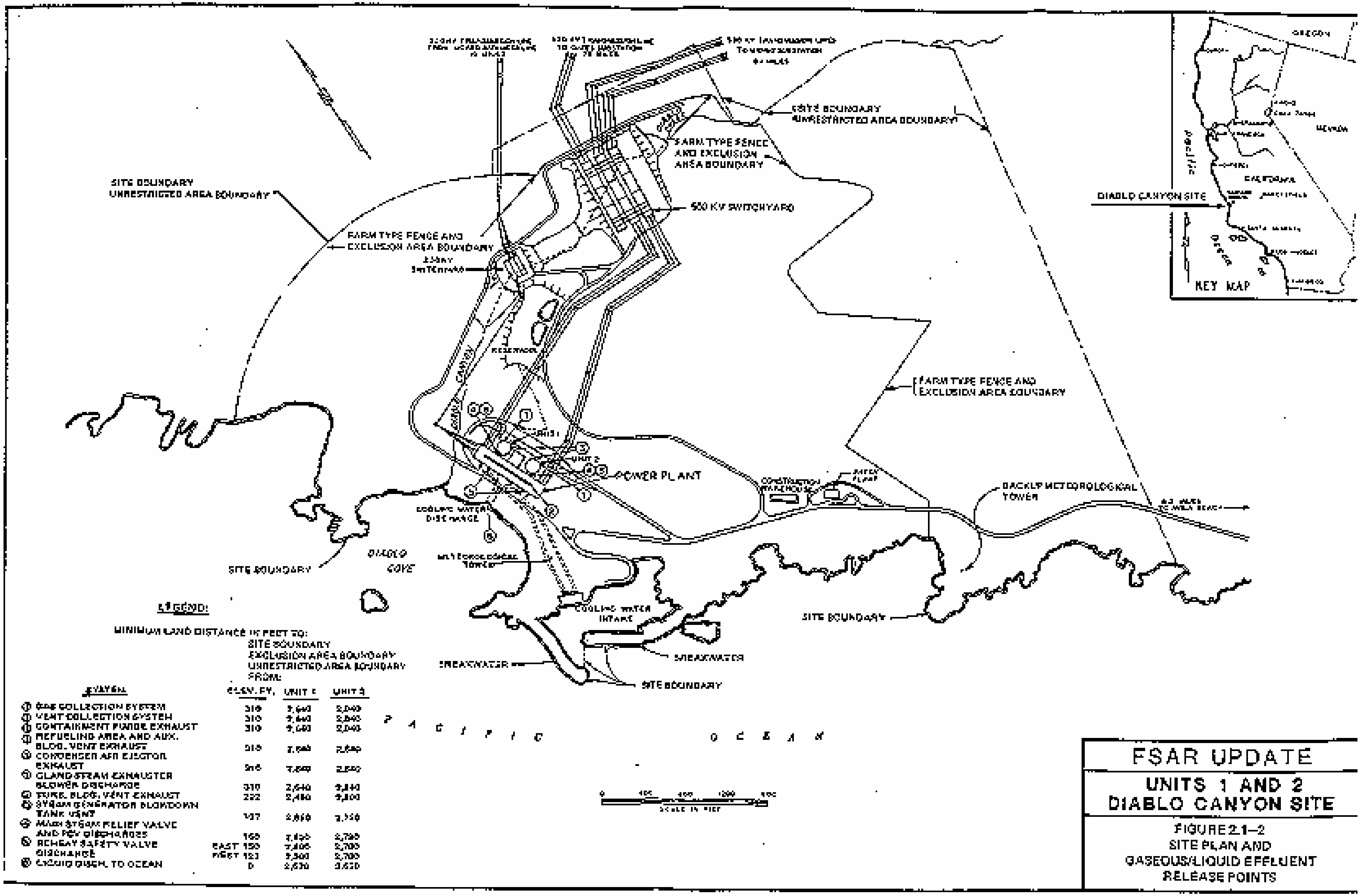
UNITS 1 AND 2 DIABLO CANYON SITE

FIGURE 2.1–1

SITE LOCATION MAP

(HISTORICAL)

063 4 3419



LEGEND:

MINIMUM LAND DISTANCE IN FEET TO:
SITE BOUNDARY
EXCLUSION AREA BOUNDARY
UNRESTRICTED AREA BOUNDARY
FROM:

SYSTEM	ELEV. FT.	UNIT 1	UNIT 2
① GAS COLLECTION SYSTEM	310	[illegible]	2,040
② VENT COLLECTION SYSTEM	310	[illegible]	2,040
③ CONTAINMENT PURGE EXHAUST	310	[illegible]	2,040
④ REFUELING AREA AND AUX. BLDG. VENT EXHAUST	310	[illegible]	[illegible]
⑤ CONDENSER AIR EJECTOR EXHAUST	310	[illegible]	[illegible]
⑥ GLAND STEAM EXHAUSTER BLOWER DISCHARGE	310	2,640	[illegible]
⑦ TURB. BLDG. VENT EXHAUST	232	2,480	[illegible]
⑧ STEAM GENERATOR BLOWDOWN TANK VENT	137	2,660	[illegible]
⑨ MAIN STEAM RELIEF VALVE AND PORV DISCHARGES	165	[illegible]	2,725
⑩ REHEAT SAFETY VALVE DISCHARGE	EAST 150	[illegible]	2,700
	WEST 121	[illegible]	2,700
⑪ LIQUID DISCH. TO OCEAN	0	2,630	[illegible]

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

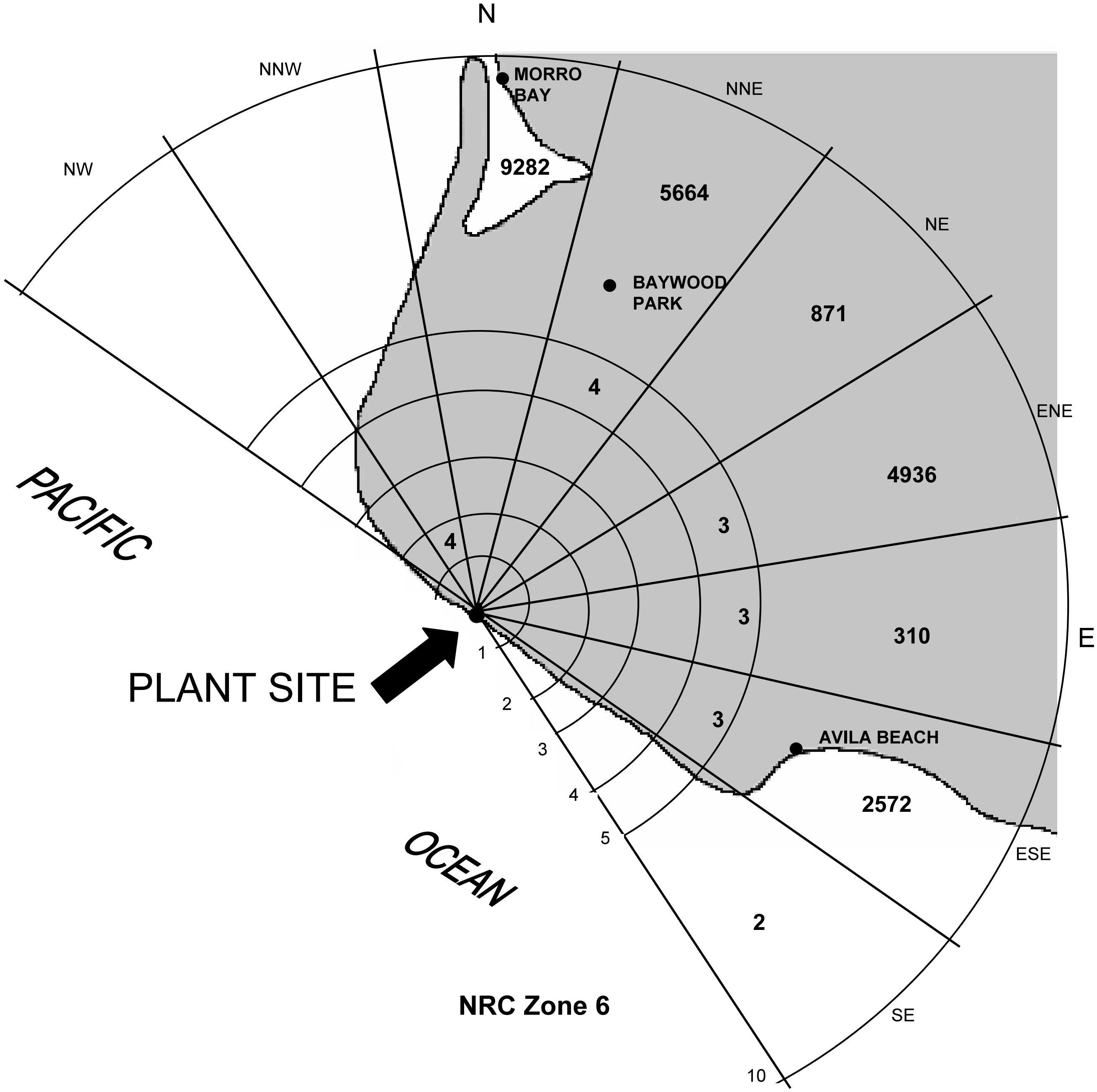
FIGURE 2.1–2
SITE PLAN AND
GASEOUS/LIQUID EFFLUENT
RELEASE POINTS

FSAR UPDATE

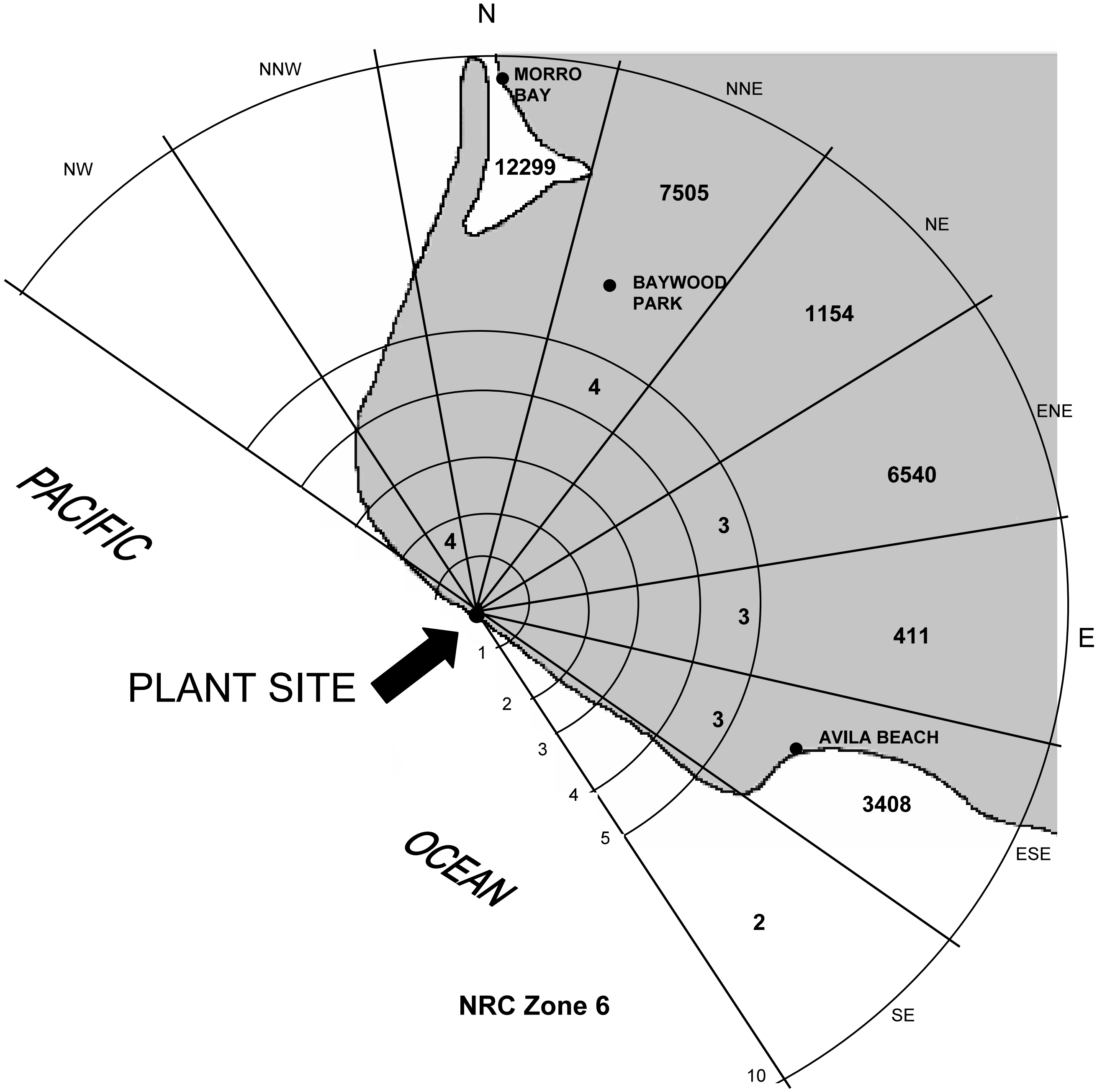
UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.1-3

AERIAL PHOTOGRAPH OF THE SITE
(HISTORICAL)



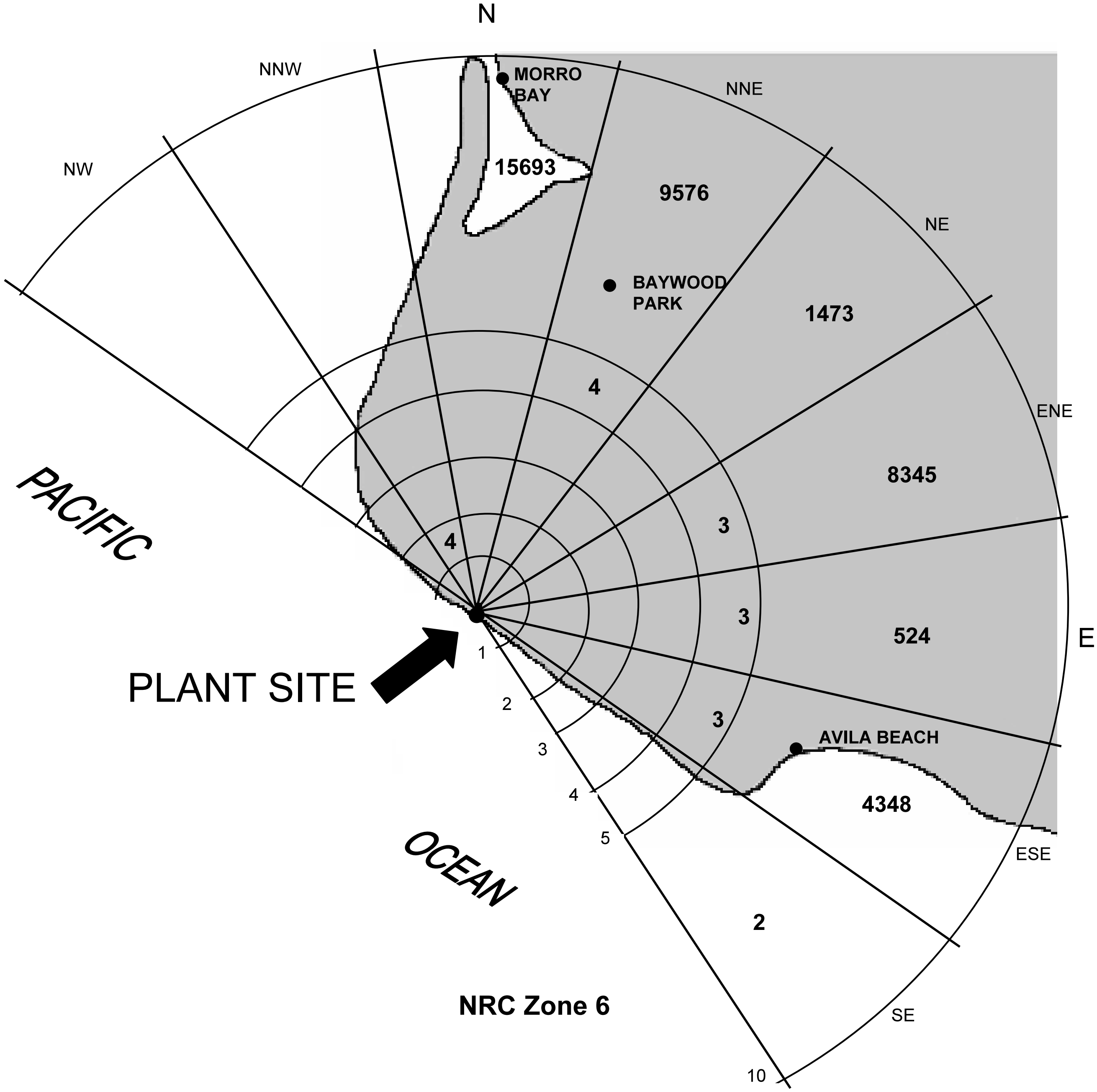
FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.1-4 POPULATION DISTRIBUTION 0 TO 10 MILES 2000 CENSUS (HISTORICAL)



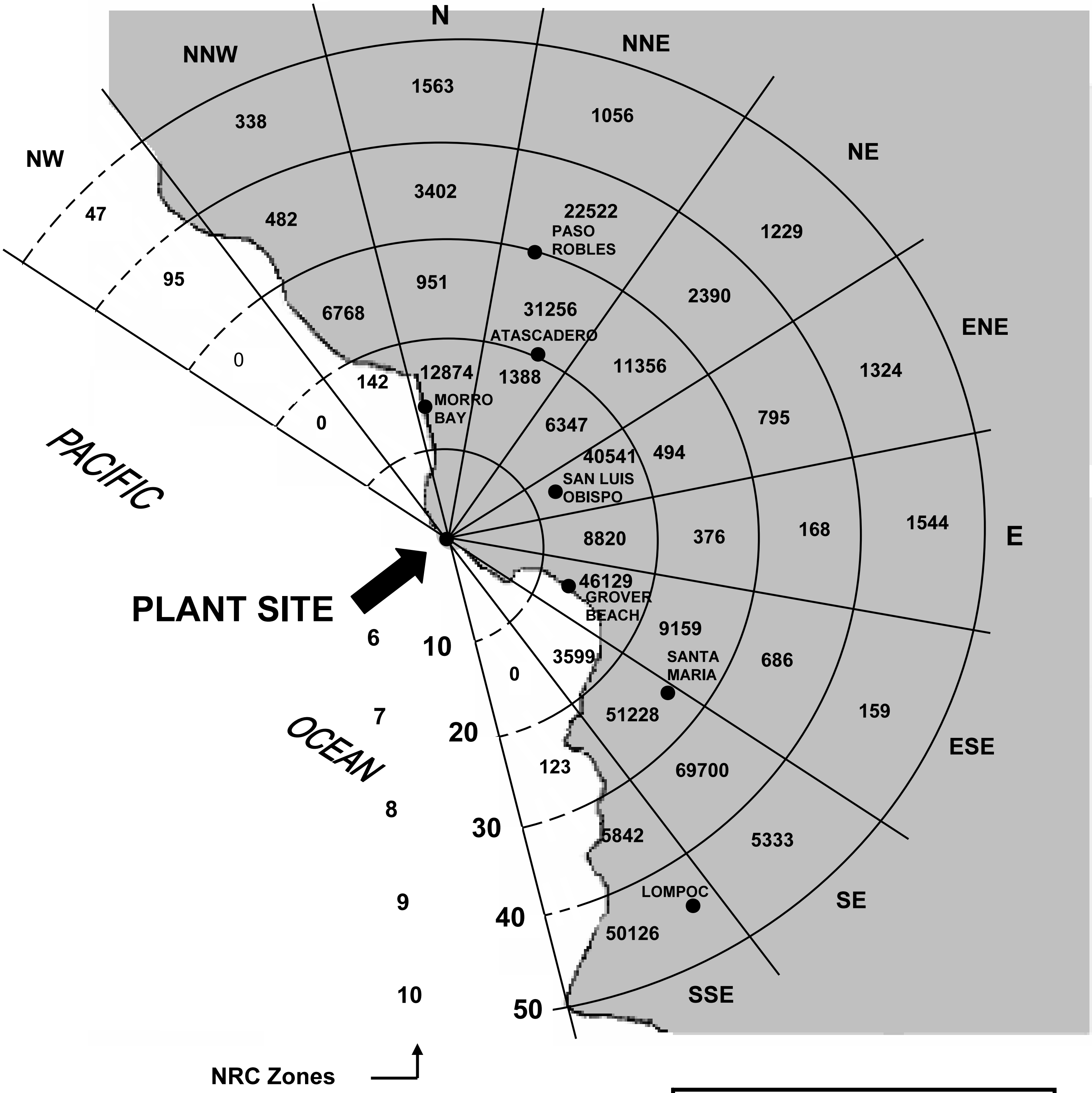
FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

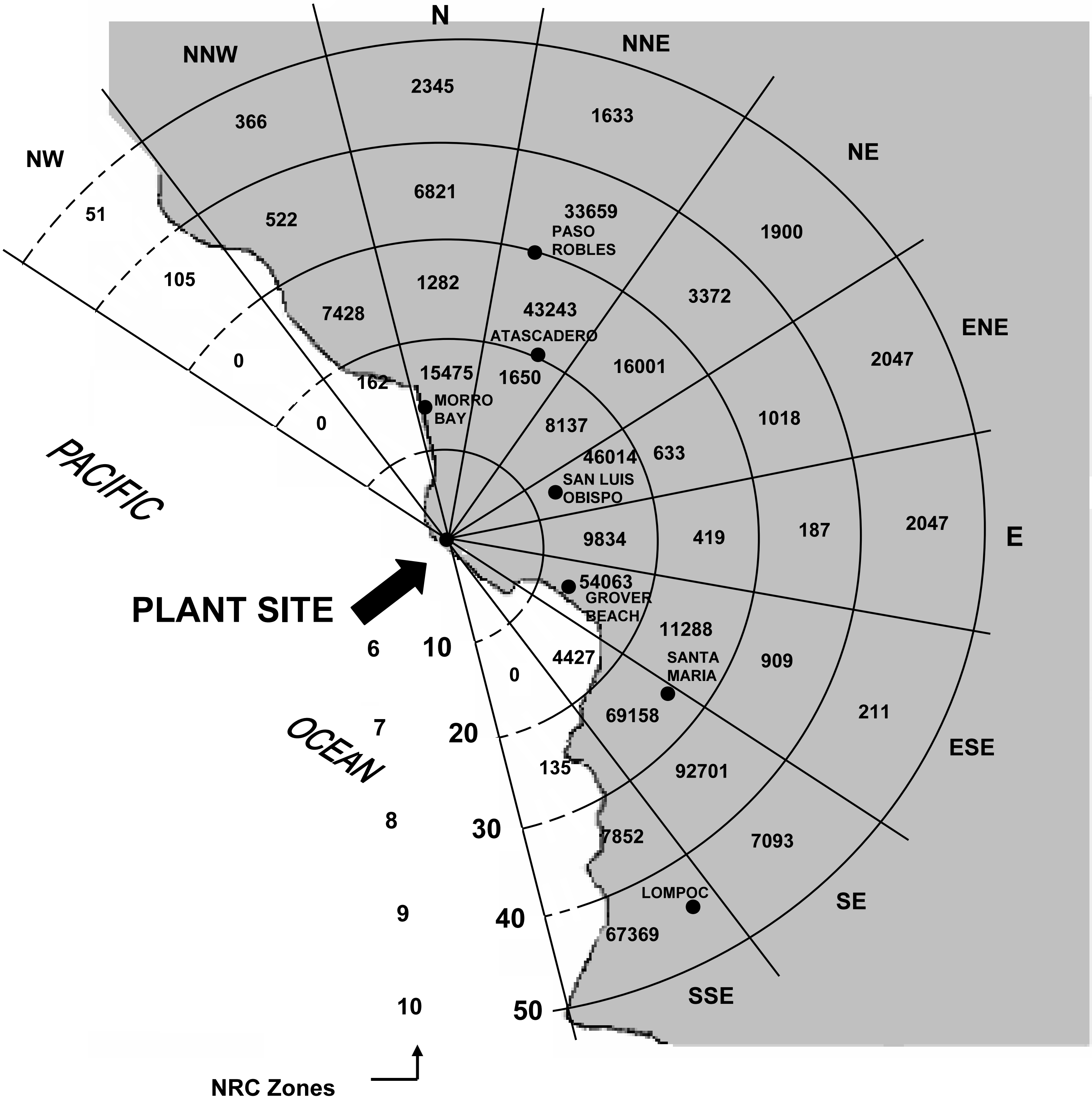
FIGURE 2.1-5
POPULATION DISTRIBUTION
0 TO 10 MILES
2010 PROJECTED
(HISTORICAL)



FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.1-6 POPULATION DISTRIBUTION 0 TO 10 MILES 2025 PROJECTED (HISTORICAL)



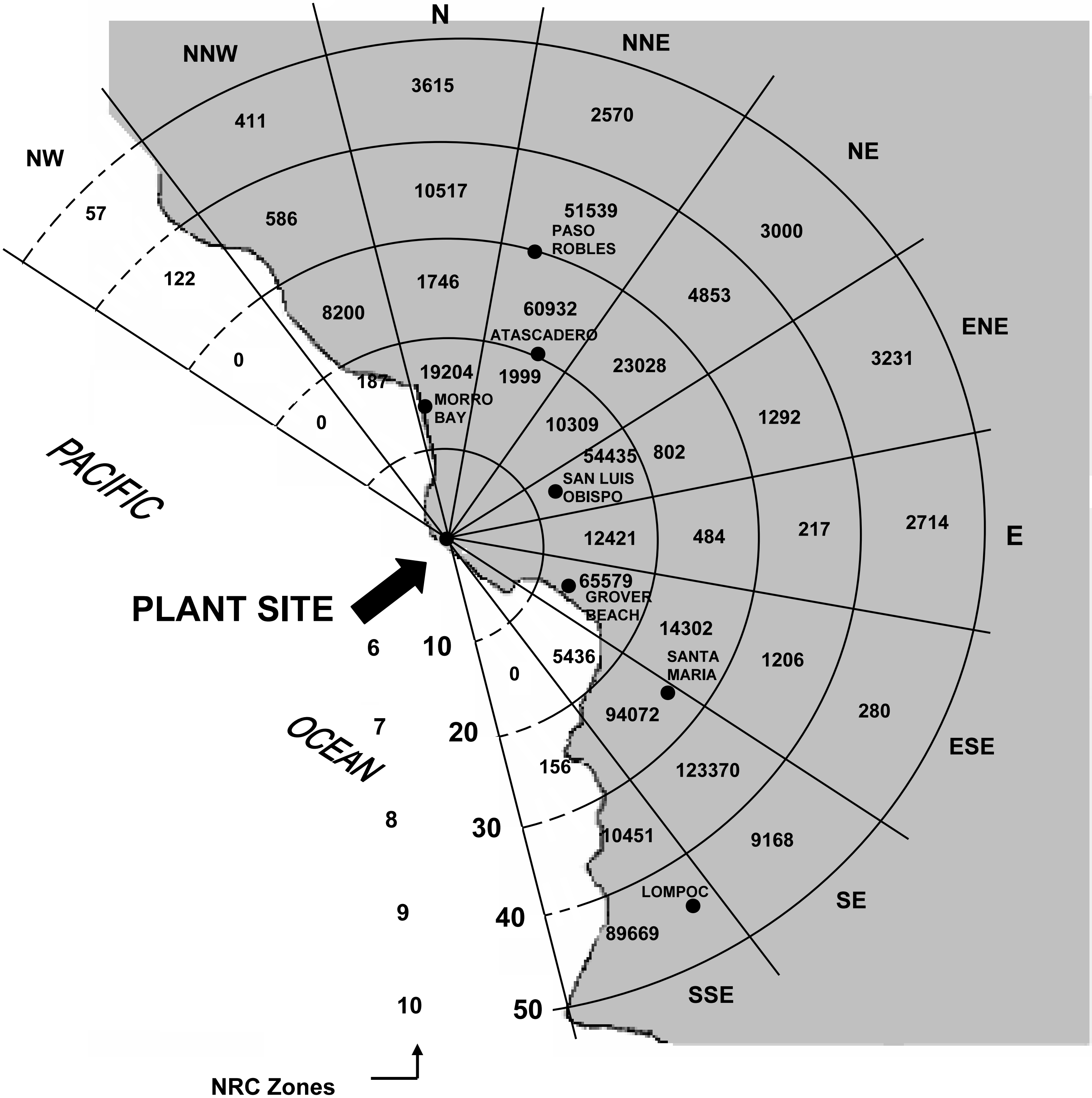
FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.1-7 POPULATION DISTRIBUTION 10 TO 50 MILES 2000 CENSUS (HISTORICAL)



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

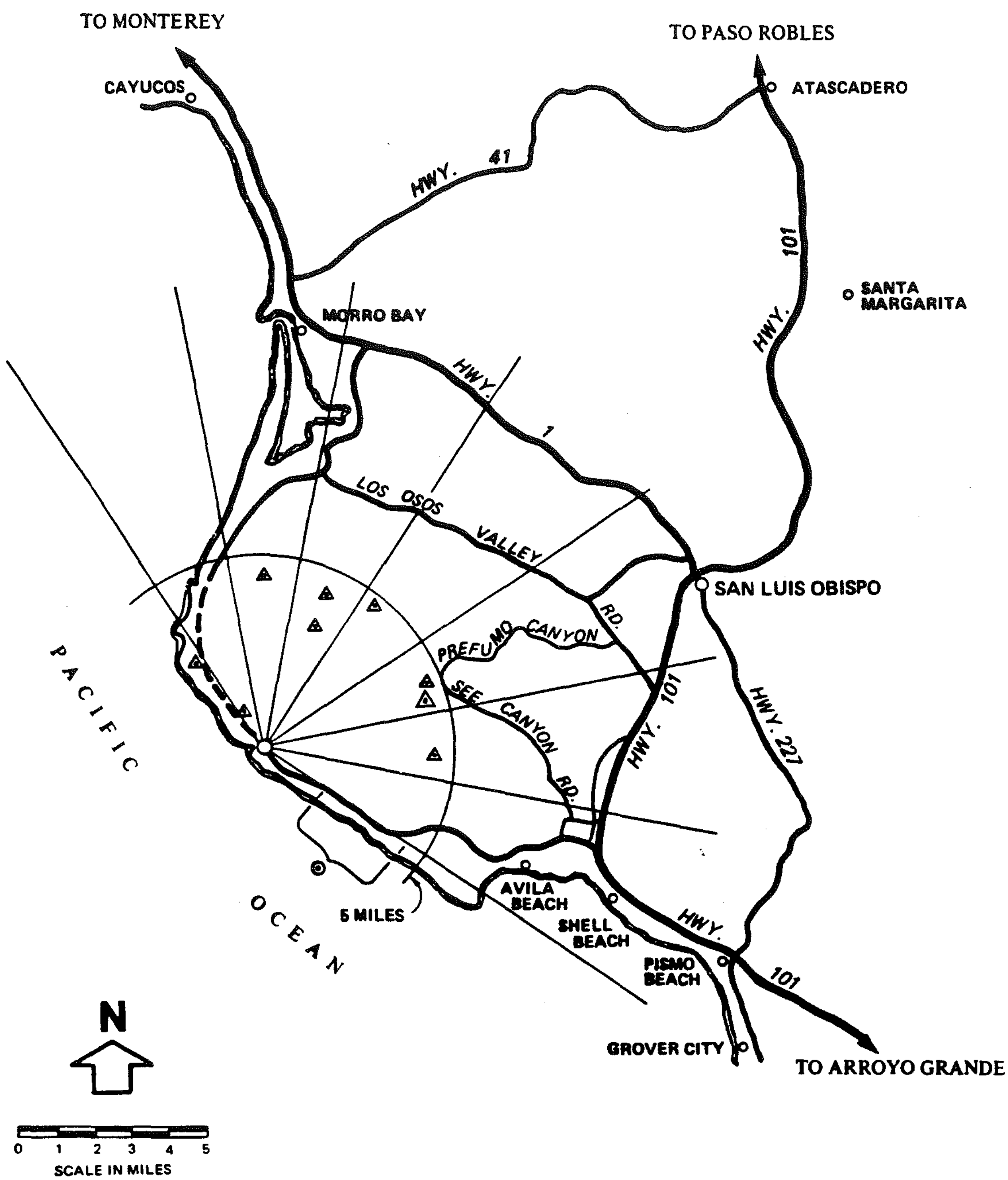
FIGURE 2.1-8
POPULATION DISTRIBUTION
10 TO 50 MILES
2010 PROJECTED
(HISTORICAL)



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.1-9
POPULATION DISTRIBUTION
10 TO 50 MILES
2025 PROJECTED
(HISTORICAL)



LEGEND

⊙ GARDENS OR FARM

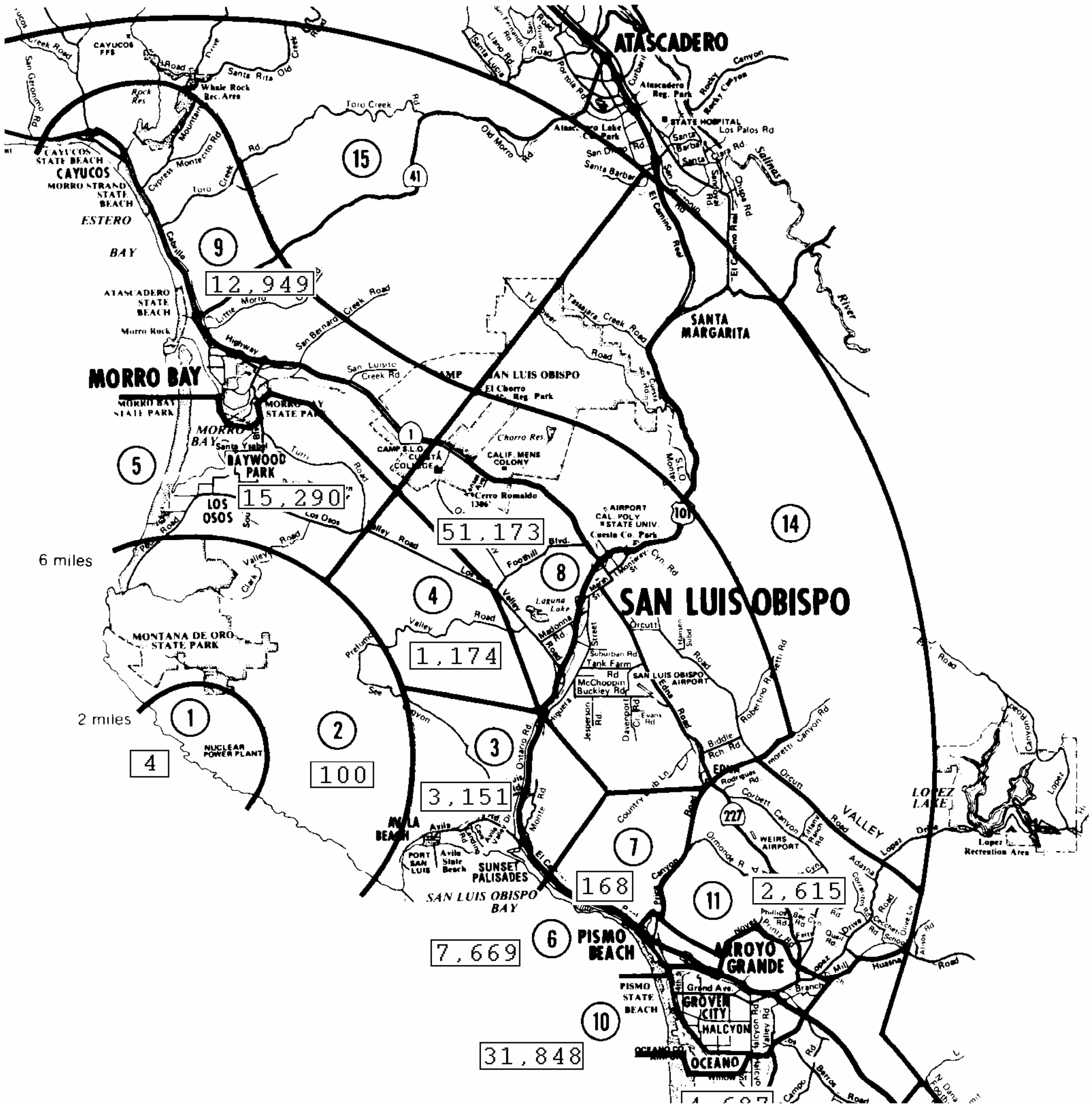
△ RESIDENCES

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

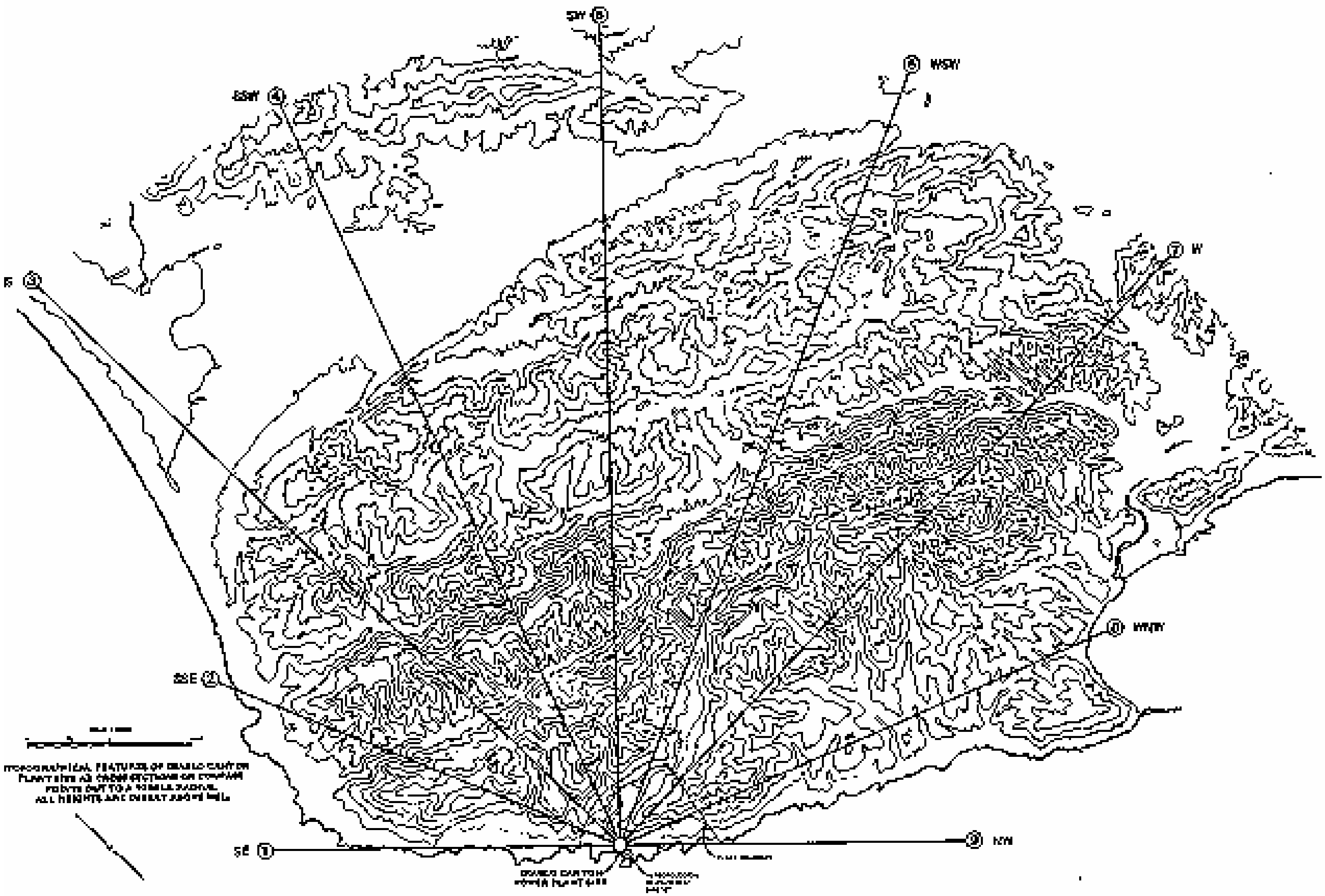
FIGURE 2.1–14.

1985 LAND USE CENSUS
(HISTORICAL)



FSAR Update
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.1-15 LOW POPULATION ZONE (HISTORICAL)

Revision 22 May 2015

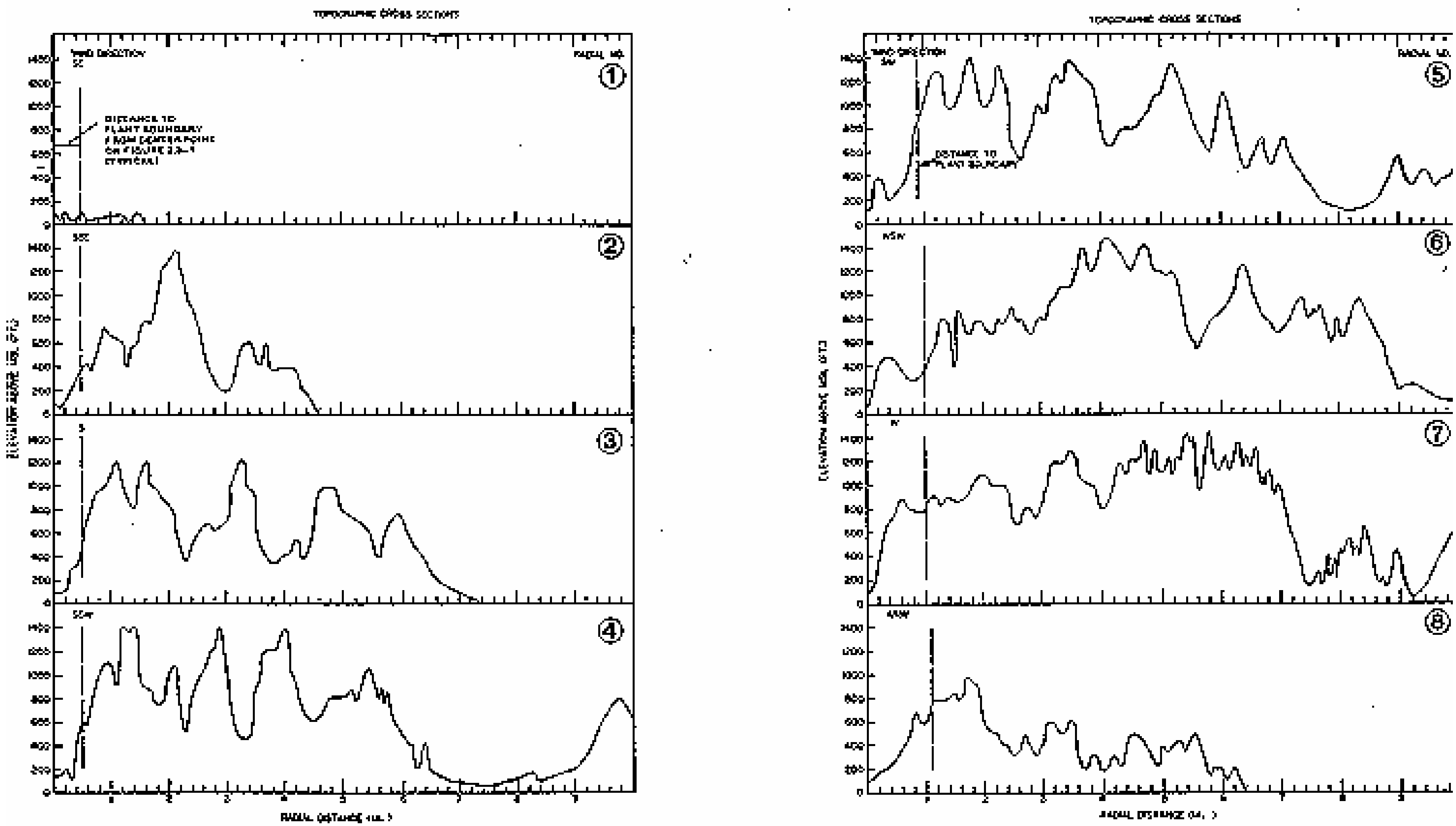


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.3-1

TOPOGRAPHICAL FEATURES AT CROSS SECTIONS TO A 10 MILE RADIUS (SECTIONS SHOWN ON FIGURE 2.3-2)



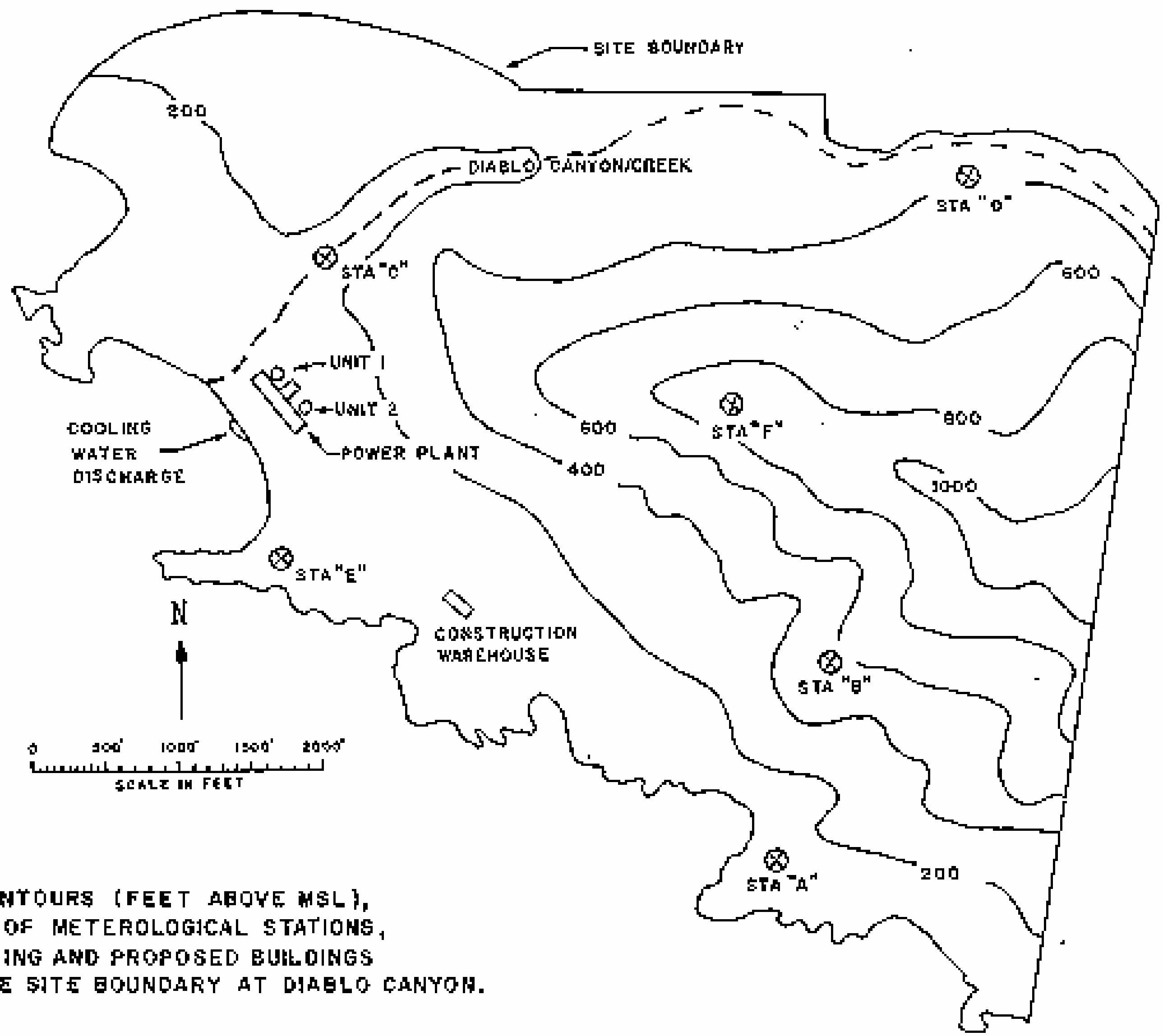
TOPOGRAPHICAL FEATURES OF DIABLO CANYON PLANT SITE AT CROSS SECTIONS ON COMPASS POINTS OUT TO A 10 MILE RADIUS. ALL HEIGHTS ARE IN FEET ABOVE MSL.

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.3-2

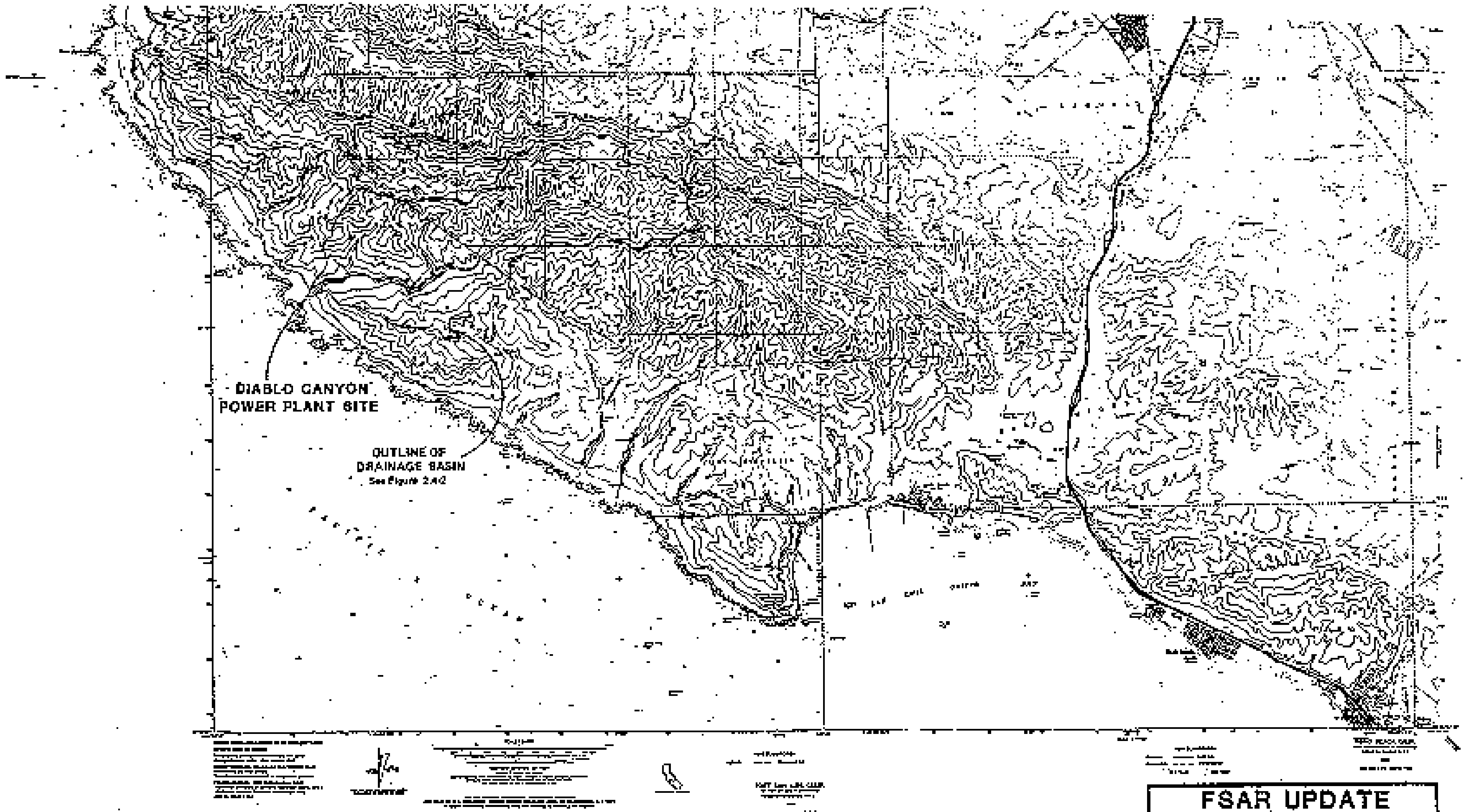
TOPOGRAPHICAL FEATURES AT CROSS SECTIONS TO A 10 MILE RADIUS (SECTIONS CUT ON FIGURE 2.3-1)



NOTE:
HEIGHT CONTOURS (FEET ABOVE MSL),
LOCATION OF METEROLOGICAL STATIONS,
AND EXISTING AND PROPOSED BUILDINGS
WITHIN THE SITE BOUNDARY AT DIABLO CANYON.

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.3-3 LOCATION OF METEROLOGICAL STATIONS WITHIN THE SITE BOUNDARY

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.3-4 LOCATION OF METEOROLOGICAL MEASUREMENT SITES AT DIABLO CANYON AND VICINITY

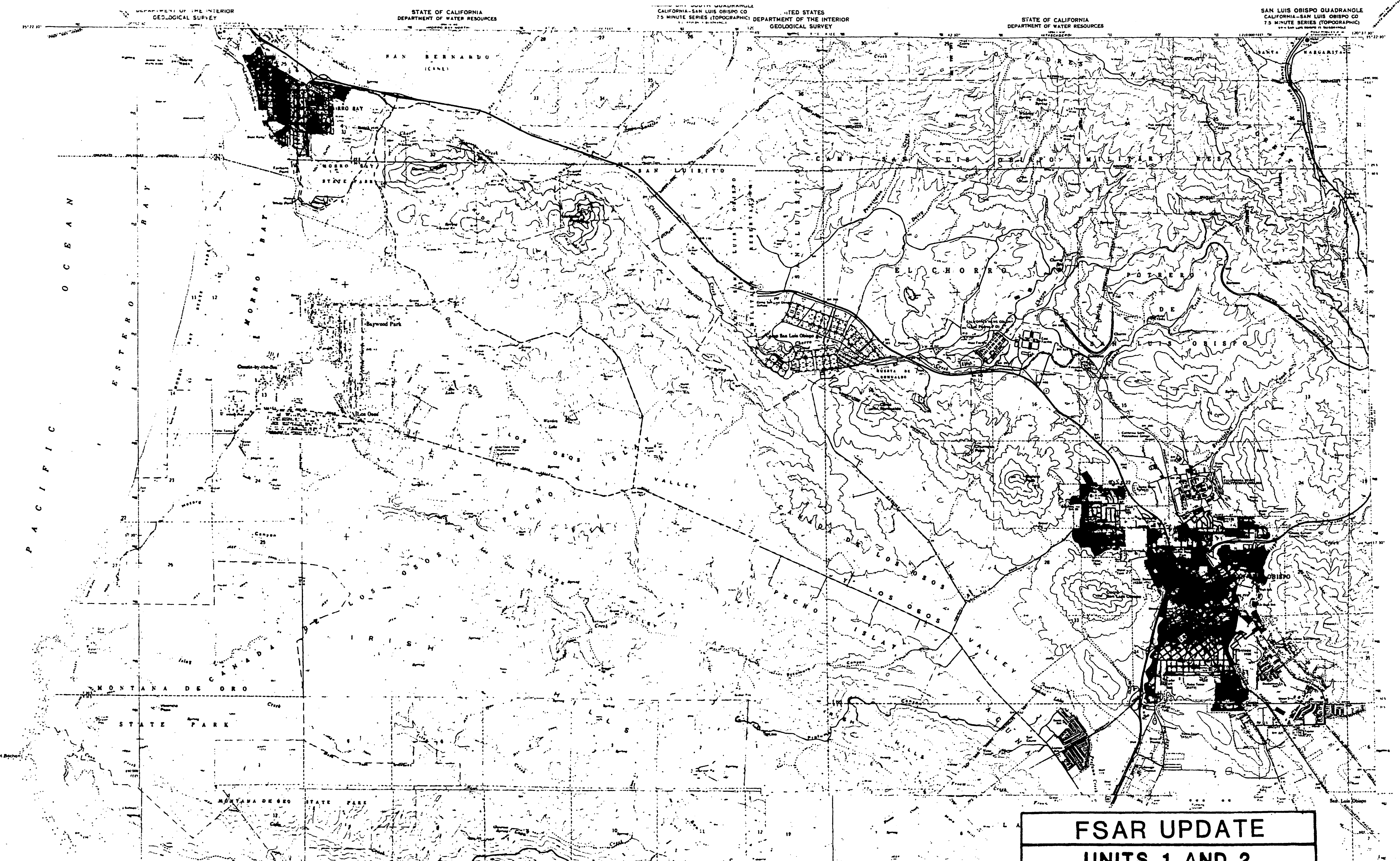


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.4-1

PLANT SITE LOCATION
DRAINAGE AND TOPOGRAPHY
(SHEET 1 OF 2)

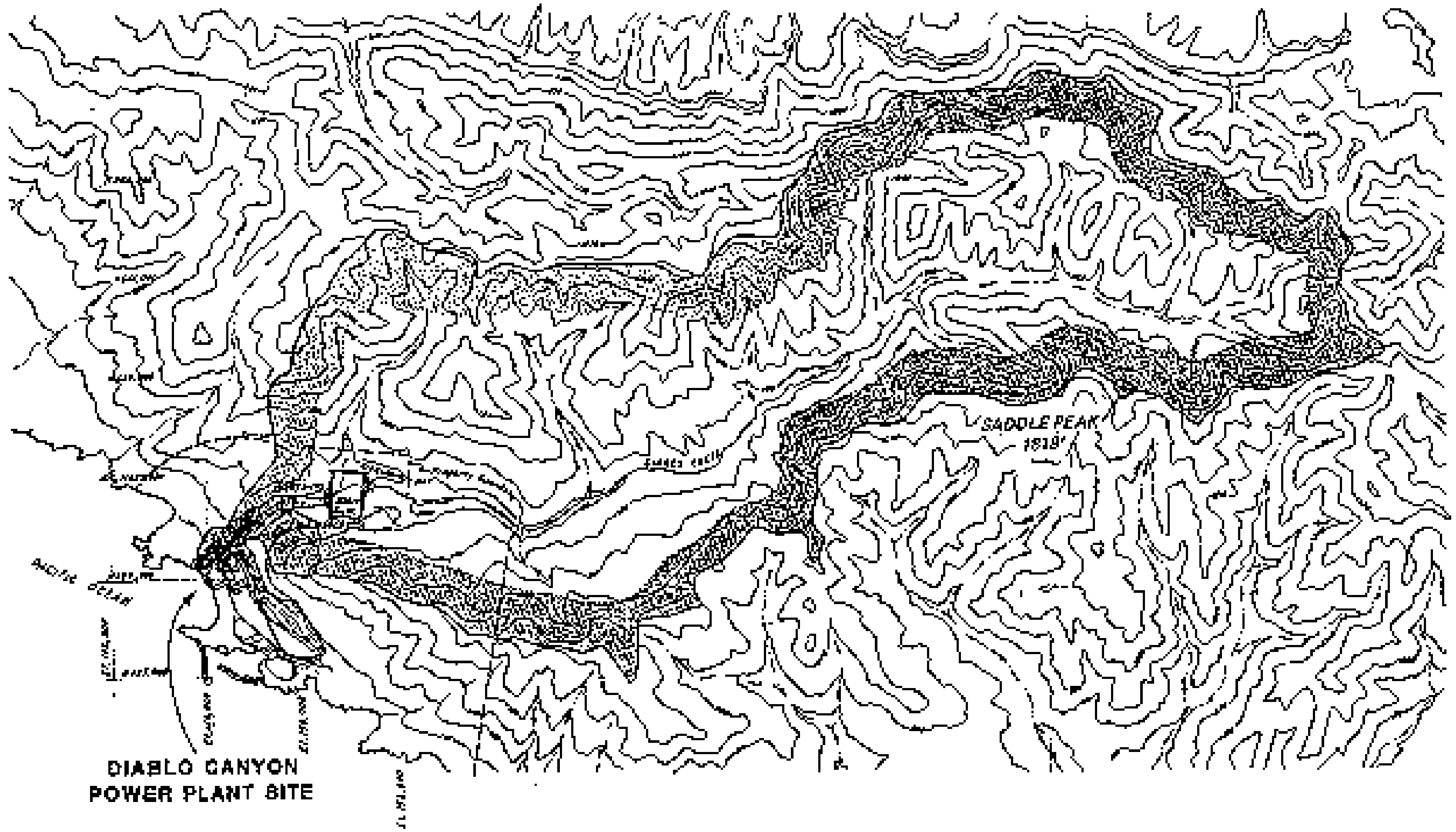


FSAR UPDATE

UNITS 1 AND 2 DIABLO CANYON SITE

FIGURE 2.4-1

PLANT SITE LOCATION
DRAINAGE AND TOPOGRAPHY
(SHEET 2 OF 2)



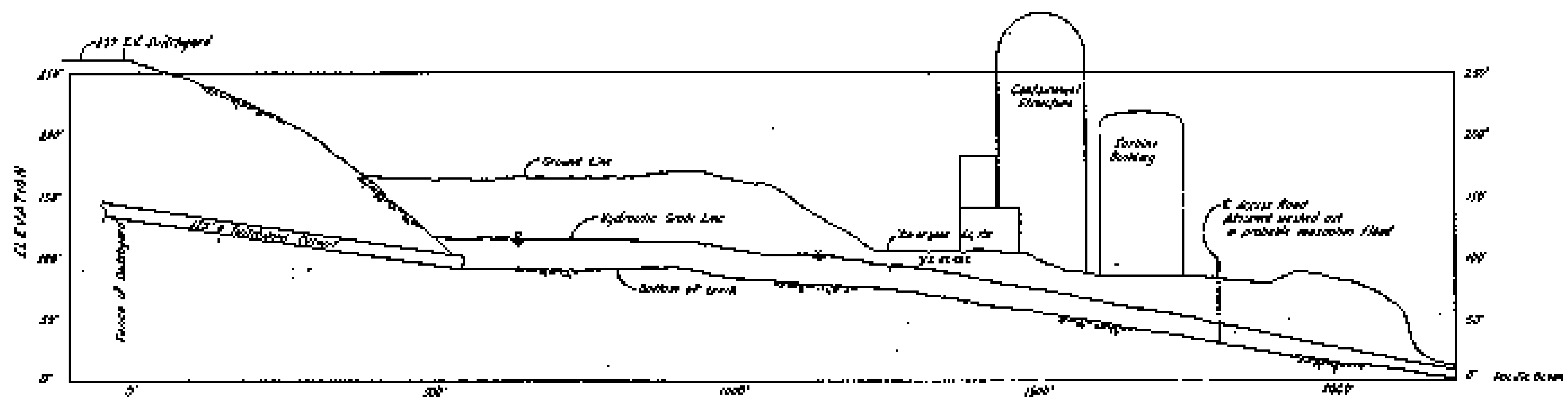
SURFACE DRAINAGE PLAN

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

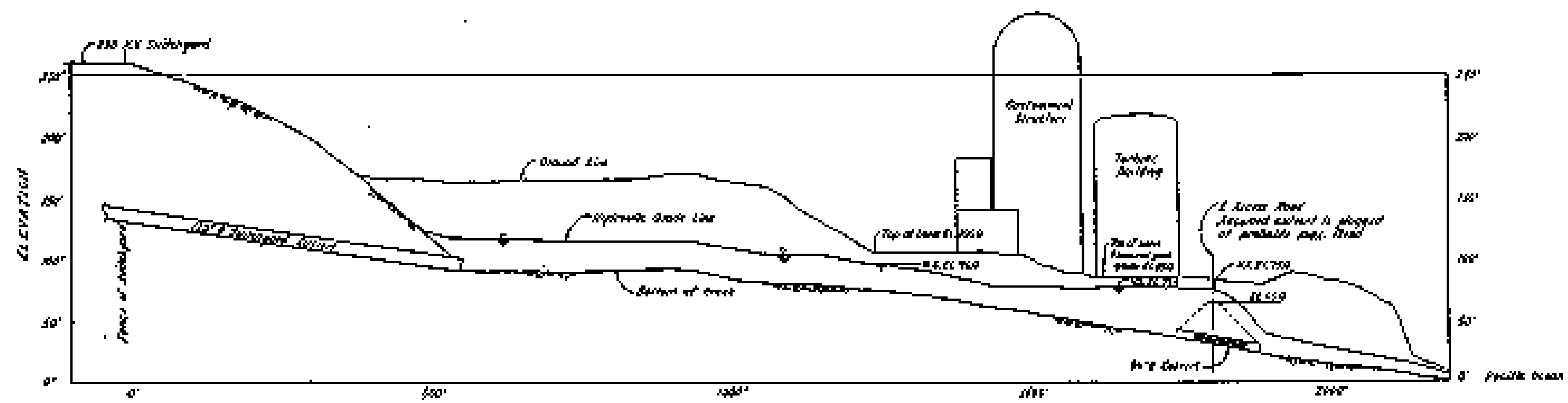
FIGURE 2.4-2

SURFACE DRAINAGE PLAN



PROFILE - DIABLO CREEK
FROM FOOT OF 230 KV SWITCHYARD TO PACIFIC OCEAN

NOTES:



PROFILE - DIABLO CREEK
FROM FOOT OF 230 KV SWITCHYARD TO PACIFIC OCEAN

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.4-3

DIABLO CREEK FROM FOOT OF 230 KV SWITCHYARD TO PACIFIC OCEAN

Revision 11 November 1996

DIABLO CANYON PROJECT JAN. 19-20, 1969 OPTIMIZATION
RUNOFF FROM LOS BERROS CREEK
RAIN FROM DIABLO & 250 TOWER

INPUT DATA

NHT	M	IPLOT	NCLRK
1	0	1	0

DA	TR	VAR1	VAR2	VAR3	VAR4	VAR5	VAR6	VAR7	QRECSN
15.00	60.	1.42	0.0	0.0	0.0	0.0	0.0	0.0	240.

FLAG1	FLAG2	FLAG3	FLAG4	FLAG5	FLAG6	FLAG7	FLGNH1	FLGNH2	RTIOR
0.	0.	0.	0.	0.	0.	0.	0.	0.	1.56

INPUT DATA

NP	VARNH1	VARNH2	STRTQ	NQQ	IQA	RQA	IQB	RQB	N
21	0.0	0.0	15.	66	0	0.0	0	0.0	1

MODIFIED INPUT DATA

NP	VARNH1	VARNH2	STRTQ	NQQ	IQA	RQA	IQB	RQB	N
21	0.50	0.50	15.	66	0	0.0	0	0.0	1.

DA	TR	VAR1	VAR2	VAR3	VAR4	VAR5	VAR6	VAR7	QRECSN
15.00	60.	1.42	1.00	1.50	0.0	2.00	0.03	0.30	240.

FLAG1	FLAG2	FLAG3	FLAG4	FLAG5	FLAG6	FLAG7	FLGNH1	FLGNH2	RTIOR
0.	0.	0.	0.	0.	0.	0.	0.	0.	1.56

OPTIMIZATION RESULTS

DA	TR	VAR1	VAR2	VAR3	VAR4	VAR5	VAR6	VAR7	QRECSN
15.00	60.	2.44	2.02	1.00	0.0	4.24	0.02	0.29	240.

FLAG1	FLAG2	FLAG3	FLAG4	FLAG5	FLAG6	FLAG7	FLGNH1	FLGNH2	RTIOR
0.	0.	0.	0.	0.	0.	0.	0.	0.	1.56

UNIT HGS.NO= 27 LAG= 2.595 CP= 0.366

366.	1031.	1367.	1274.	1039.	848.	691.	564.	460.	375.
306.	250.	204.	166.	135.	110.	90.	73.	60.	49.
40.	33.	27.	22.	18.	14.	12.			

NP	VARNH1	VARNH2	STARTQ	NQC	IQA	RQA	IQB	RQB	STRTK
21	0.14	0.59	15.	66	0	0.0	0	0.0	0.14

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.4-4
OPTIMIZATION OF FIT
DIABLO - LOS BERROS
(SHEET 1 OF 3)

LOS BERROS CREEK OPTIMIZATION

PERIOD	RAIN	LOSS	EXCESS	COMP Q	OBS Q
1	0.01	0.01	0.0	14.	15.
2	0.1[illegible]	0.10	[illegible]	14.	30.
3	0.02	0.02	0.0	13.	40.
4	0.08	0.06	0.0	13.	45.
5	0.07	0.07	0.0	12.	40.
6	0.27	0.14	0.13	41.	51.
7	0.14	0.09	0.05	154.	140.
8	0.48	0.05	0.4[illegible]	203.	190.
9	0.08	0.07	0.01	261.	234.
10	0.02	0.02	0.0	197.	150.
11	0.03	0.03	0.0	184.	118.
12	0.07	0.07	0.0	170.	115.
13	0.15	0.08	0.07	161.	112.
14	0.05	0.05	0.0	161.	150.
15	0.70	0.46	0.17	210.	215.
16	0.07	0.05	0.01	273.	300.
17	0.03	0.03	0.0	241.	322.
18	0.13	0.06	0.07	290.	450.
19	0.24	0.05	0.19	361.	502.
20	0.16	0.06	0.10	571.	575.
21	0.02	0.02	0.0	611.	600.
22				575.	505.
23				453.	426.
24				394.	335.
25				322.	341.
26				264.	329.
27				240.	290.
28				219.	265.
29				220.	246.
30				216.	235.
31				201.	220.
32				192.	210.
33				184.	197.
34				176.	187.
35				161.	178.
36				163.	170.
37				154.	162.
38				147.	158.
39				141.	152.
40				135.	145.
41				129.	141.
42				123.	135.
43				118.	128.
44				113.	122.
45				108.	118.
46				103.	114.
47				95.	109.
48				94.	105.
49				90.	100.
50				86.	95.
51				83.	90.
52				79.	84.
53				76.	80.
54				72.	74.
55				69.	70.
56				66.	65.
57				63.	60.
58				60.	55.
59				58.	50.
60				55.	45.
61				53.	46.
62				51.	35.
63				48.	30.
64				46.	28.
65				44.	25.
66				42.	20.
TOTAL	2.03	1.32	0.71	[illegible]	[illegible]

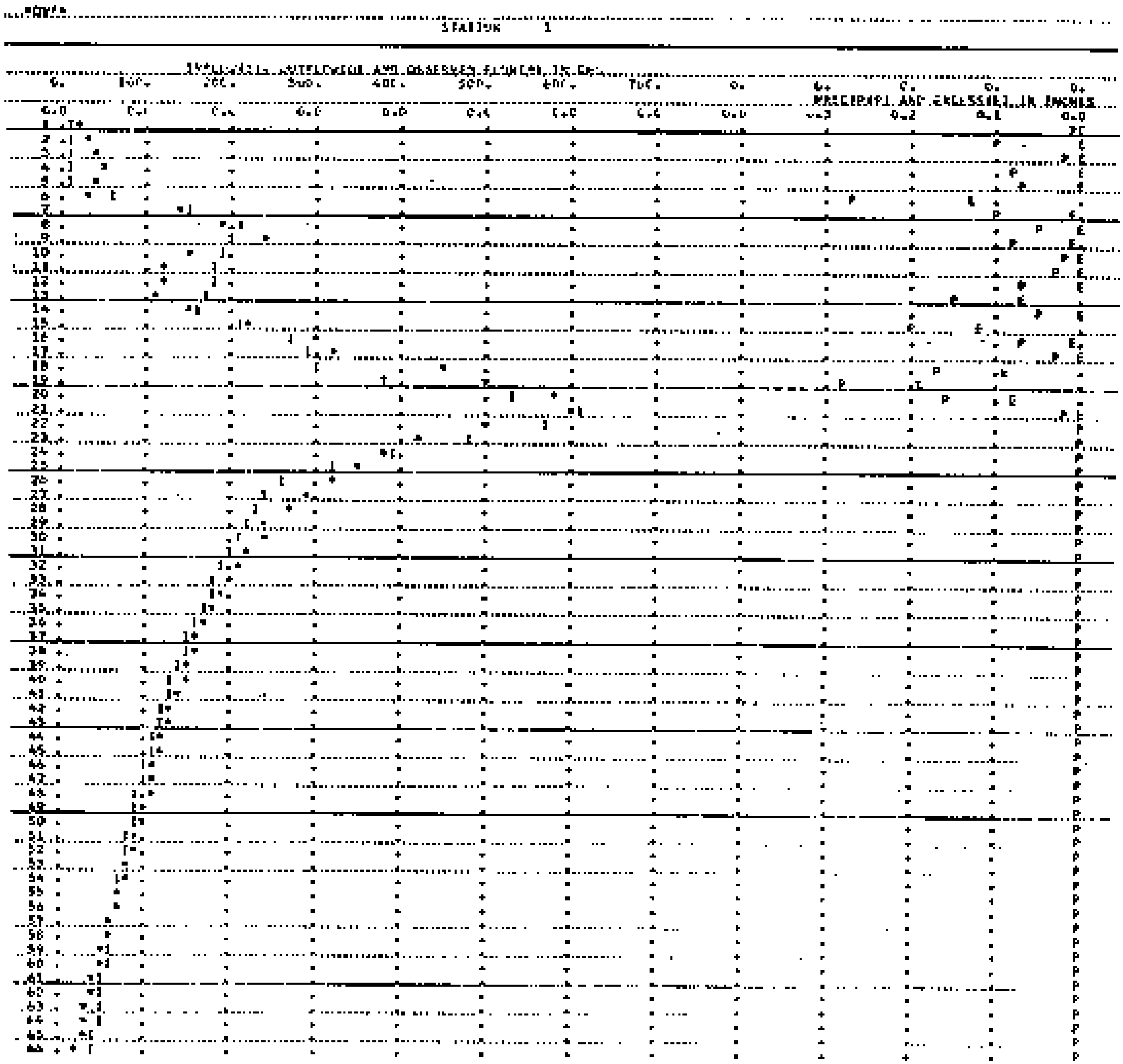
PEAK OBSERVED = 600.00 PEAK HR = 21.
PEAK COMPUTED = 611.03 PEAK HR = 21

COMPUTED VOL. = 593. AC.FT.
OBSERVED VOL. = 552. AC.FT.
RATIO OF VOL. FIT = 1.0[illegible]
RATIO OF PEAK FIT = 0.98
LOSS PERCENTAGE = 65.02

FSAR UPDATE

UNITS 1 AND 2 DIABLO CANYON SITE

FIGURE 2.4-4
OPTIMIZATION OF FIT
DIABLO – LOS BERROS
(SHEET 2 OF 3)



FSAR UPDATE

UNITS 1 AND 2 DIABLO CANYON SITE

FIGURE 2.4-4
OPTIMIZATION OF FIT
DIABLO - LOS BERROS
(SHEET 3 OF 3)

D E S I G N   F L O O D   H Y D R O G R A P H

DIABLO CANYON CREEK, 24HR PMP
LOSS RATES EQUAL 50% OF OPTIMIZED RESULTS
RAIN AND RAINFALL DISTRIBUTION FROM HMR 36

INPUT DATA - BASIN CHARACTERISTICS

DA	TR	VAR1	VAR2	VAR3	VAR4	VAR5	VAR6	VAR7	DSECSN
5.19	60.	0.96	2.02	1.00	0.0	8.48	0.01	0.29	240.

NP	NCLRX	VARNH1	VARNH2	STARTQ	STRTK	RTIOR
25	0	0.07	0.30	0.0	0.07	1.56

OUTPUT

UNIT HGR.NO. 11   LAG= 1.755   CP= 0.505

686.	1091.	644.	390.	224.	132.	78.	46.	27.	16.
9.									

DESIGN FLOOD HYDROGRAPH   DIABLO CANYON CREEK 24HR PMP

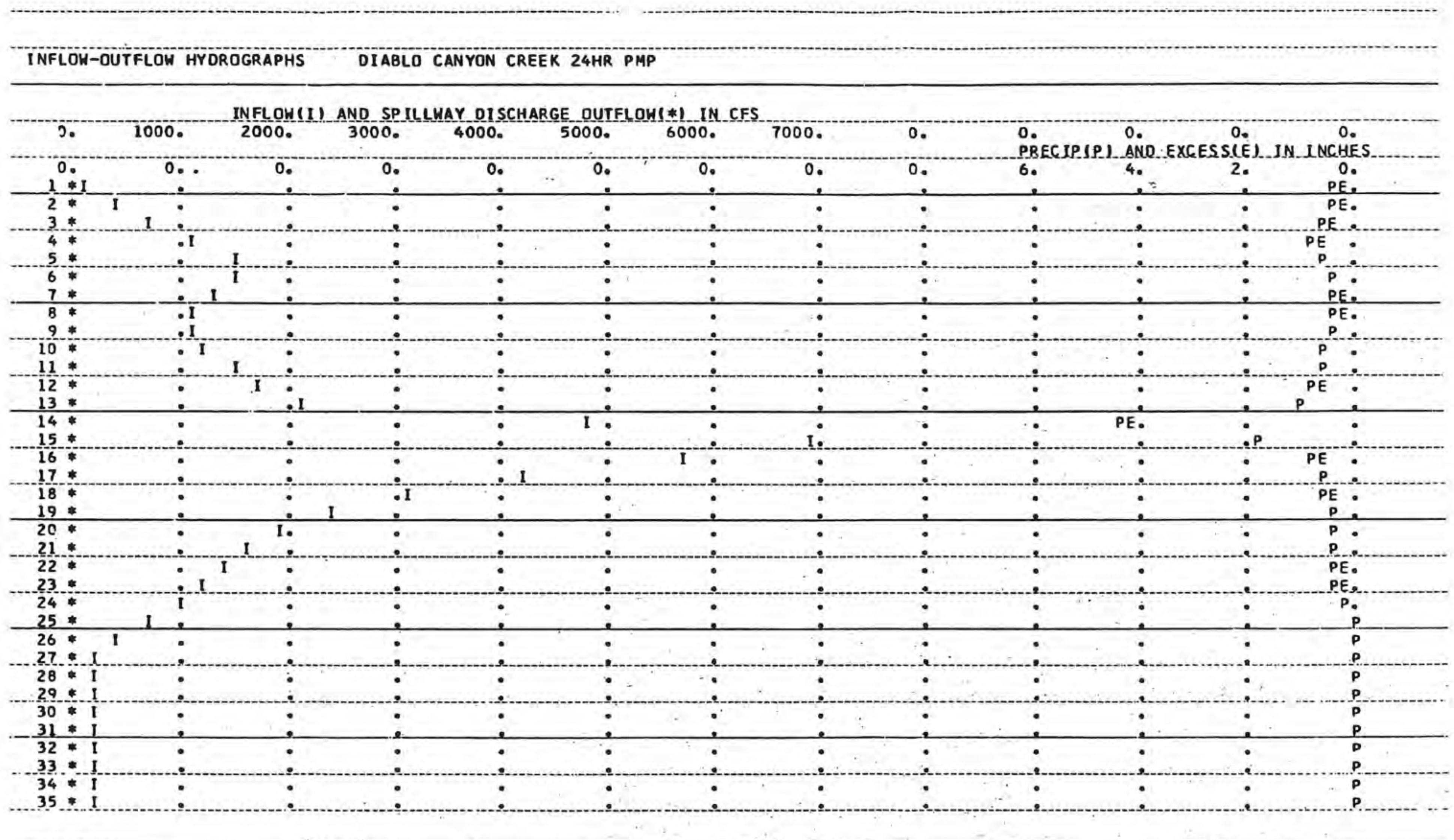
PERIOD	RAIN	LOSS	EXCESS	COMP Q
1	0.30	0.09	0.21	142.
2	0.30	0.07	0.23	383.
3	0.50	0.07	0.43	678.
4	0.70	0.07	0.63	1131.
5	0.60	0.06	0.54	1474.
6	0.40	0.05	0.35	1491.
7	0.30	0.05	0.25	1287.
8	0.30	0.05	0.25	1108.
9	0.40	0.05	0.35	1089.
10	0.60	0.05	0.55	1255.
11	0.60	0.05	0.55	1493.
12	0.70	0.06	0.64	1690.
13	1.00	0.06	0.94	2081.
14	4.30	0.09	4.21	4757.
15	1.80	0.07	1.73	6878.
16	0.70	0.05	0.65	5685.
17	0.60	0.05	0.55	4173.
18	0.50	0.05	0.45	3158.
19	0.40	0.04	0.36	2410.
20	0.40	0.04	0.36	1908.
21	0.40	0.04	0.36	1611.
22	0.30	0.04	0.26	1370.
23	0.30	0.04	0.26	1162.
24	0.20	0.04	0.16	972.
25				663.
26				381.
27				240.
28				230.
29				220.
30				210.
31				201.
32				192.
33				184.
34				176.
35				168.
TOTAL	16.60	1.34	15.26	52191.

PERCENT LOSS = 8.08 %
PERIOD OF MAXIMUM INFLOW = 15
MAXIMUM INFLOW = 6878. CFS
UNIT MAXIMUM INFLOW = 1325. CFS/SQ.MILE

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.4-5
DESIGN FLOOD HYDROGRAPH
(SHEET 1 OF 3)



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.4-5

DESIGN FLOOD HYDROGRAPH

(SHEET 2 OF 3)

Revision 11 November 1996

DEFINITION OF SYMBOLS

DA	– CONTRIBUTING DRAINAGE AREA IN SQUARE MILES
TR	– RAINFALL AND RUNOFF INTERVAL IN MINUTES
VAR 1	– CLARK'S TIME OF CONCENTRATION T_C IN HOURS
VAR 2	– RATIO OF CLARK'S R TO T_C
VAR 3	– SHAPE FACTOR FOR SYNTHETIC TIME-AREA CURVE
VAR 4	– RATIO OF IMPERVIOUSNESS OF DRAINAGE AREA
VAR 5	– RATIO OF K ON STRAIGHT LINE PORTION OF LOSS RATE CURVE TO K AT 10 INCHES MORE ACCUMULATED LOSS
VAR 6	– RECOVERY LOSS INDEX IN INCHES, SUBTRACTED FROM ACCUMULATED LOSS EVERY PERIOD
VAR 7	– EXPONENT OF RAIN IN LOSS COMPUTATION
QRECSN	– FLOW BELOW WHICH RECESSION RATES ARE MAINTAINED AS A MINIMUM
NP	– NUMBER OF OBSERVED PRECIPITATION PERIODS IN STORM
NCLRK	– INDICATOR, CALLS FOR THE NUMBER OF TIME-AREA ORDINATES
VAR NH1	– LOSS RATE INDEX – VALUE OF K ON STRAIGHT LINE PORTION OF LOSS RATE CURVE WHEN ACCUMULATED LOSS IS 1/2 OF STORM LOSS. ALSO KNOWN AS VAR 8
VAR NH2	– ACCUMULATED LOSS INCREMENT DURING INITIAL LOSS PERIOD. ADDS AN INCREMENT OF 0.2(VAR(NH2)) TO K WHEN ACCUMULATED LOSS IS ZERO, DECREASING TO ZERO WHEN ACCUMULATED LOSS IS VAR(NH2). ALSO KNOWN AS VAR 9
STRTQ	– FLOW IN CFS AT START OF FIRST TR PERIOD OF STORM
RTIOR	– RATIO OF RECESSION FLOW TO THAT 10 TR UNITS LATER
LAG	– SNYDER'S T_p IN HOURS
	– TIME IN HOURS FROM CENTER OF MASS OF EXCESS RAINFALL TO PEAK OF UNIT HYDROGRAPH
CP	– SNYDER'S C_p

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.4-5
DESIGN FLOOD HYDROGRAPH
DEFINITION OF SYMBOLS
(SHEET 3 OF 3)

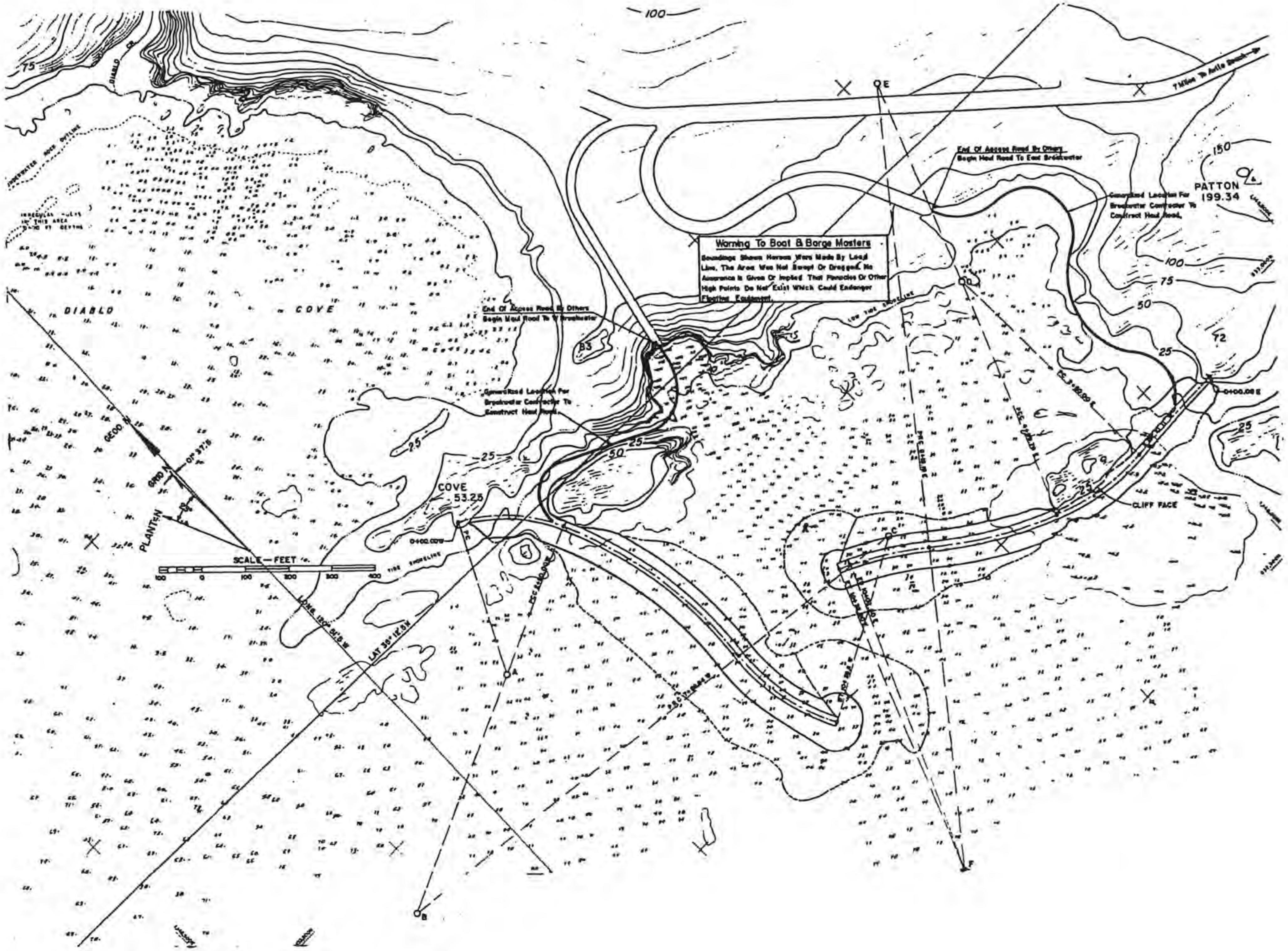
INTAKE BASIN BREAKWATERS FOR DIABLO CANYON SITE

PACIFIC GAS & ELECTRIC COMPANY

NOTES–

1. Soundings refer to Mean Lower Low Water Datum.
2. Contours shown above "Low Tide Shoreline" refer to Sea Level Datum and are from an aerial survey made before power plant excavation was started. Bidders shall ascertain by visit to the site, before submitting bids, what effect the site excavations have, if any, on the cost of the work to be done in building the breakwaters.
3. To convert elevations from one datum to the other, apply the following: $Elev_{MLLW} = Elev_{SLD} + 2.6$ Ft.
4. Coordinate positions shown refer to California Coordinate System, Zone 5, a Lambert Conformal Projection. Conversion of arbitrary Plant Grid System is made as follows:

$$N_L = 576{,}067.06 + .9205049\,N_P + .3907311\,D_P$$
$$D_L = 1{,}161{,}806.12 + .9205049\,D_P - .3907311\,N_P$$



FSAR UPDATE

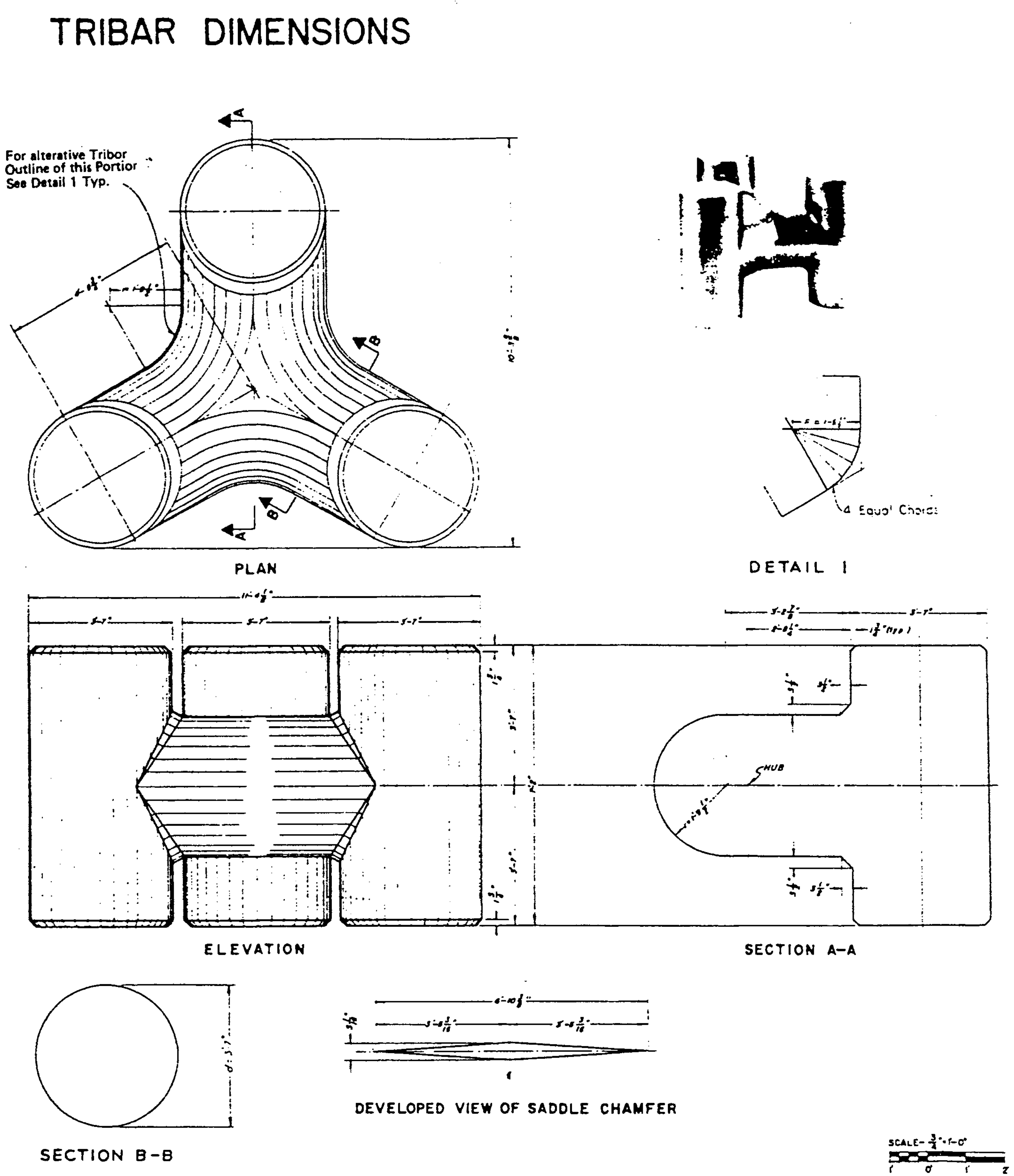
UNITS 1 AND 2 DIABLO CANYON SITE

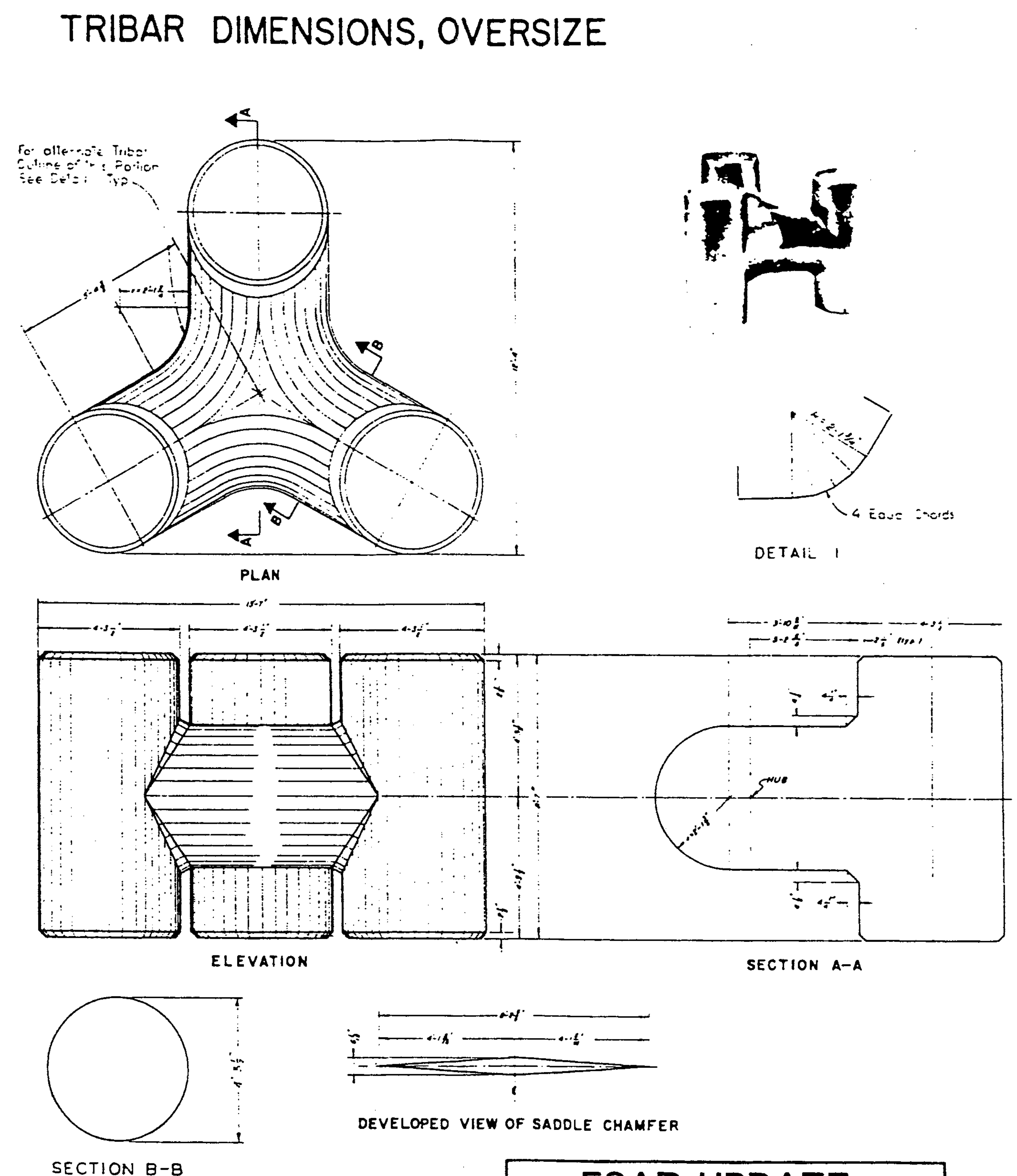
FIGURE 2.4–6
GENERAL LAYOUT OF BREAKWATERS
(Sheet 1 of 2 – SOUNDINGS)

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.4–6
GENERAL LAYOUT OF BREAKWATERS
(Sheet 2 of 2 – CONTOURS)





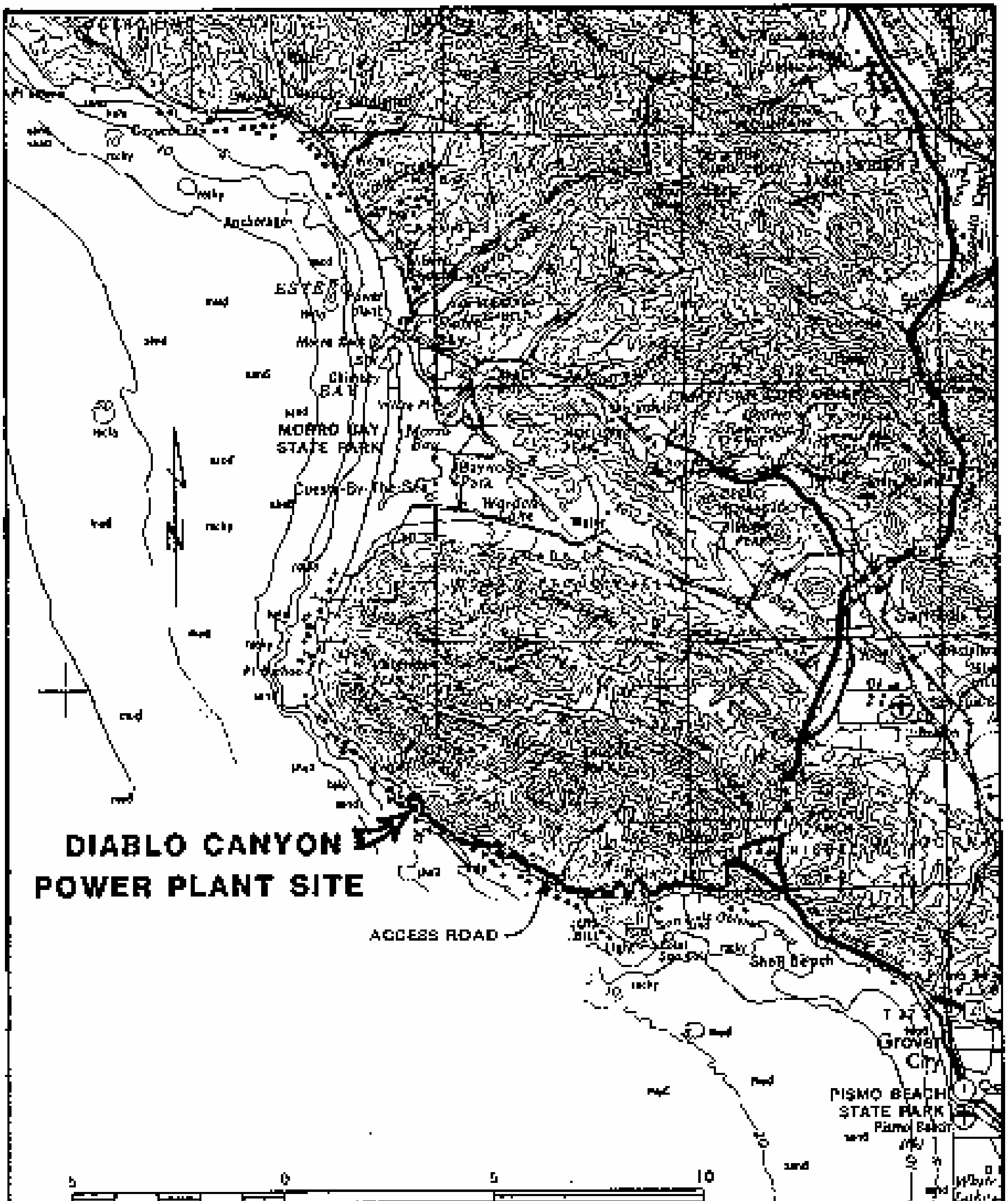
FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.4-9

DIMENSIONS FOR TRIBARS

Revision 11 November 1996

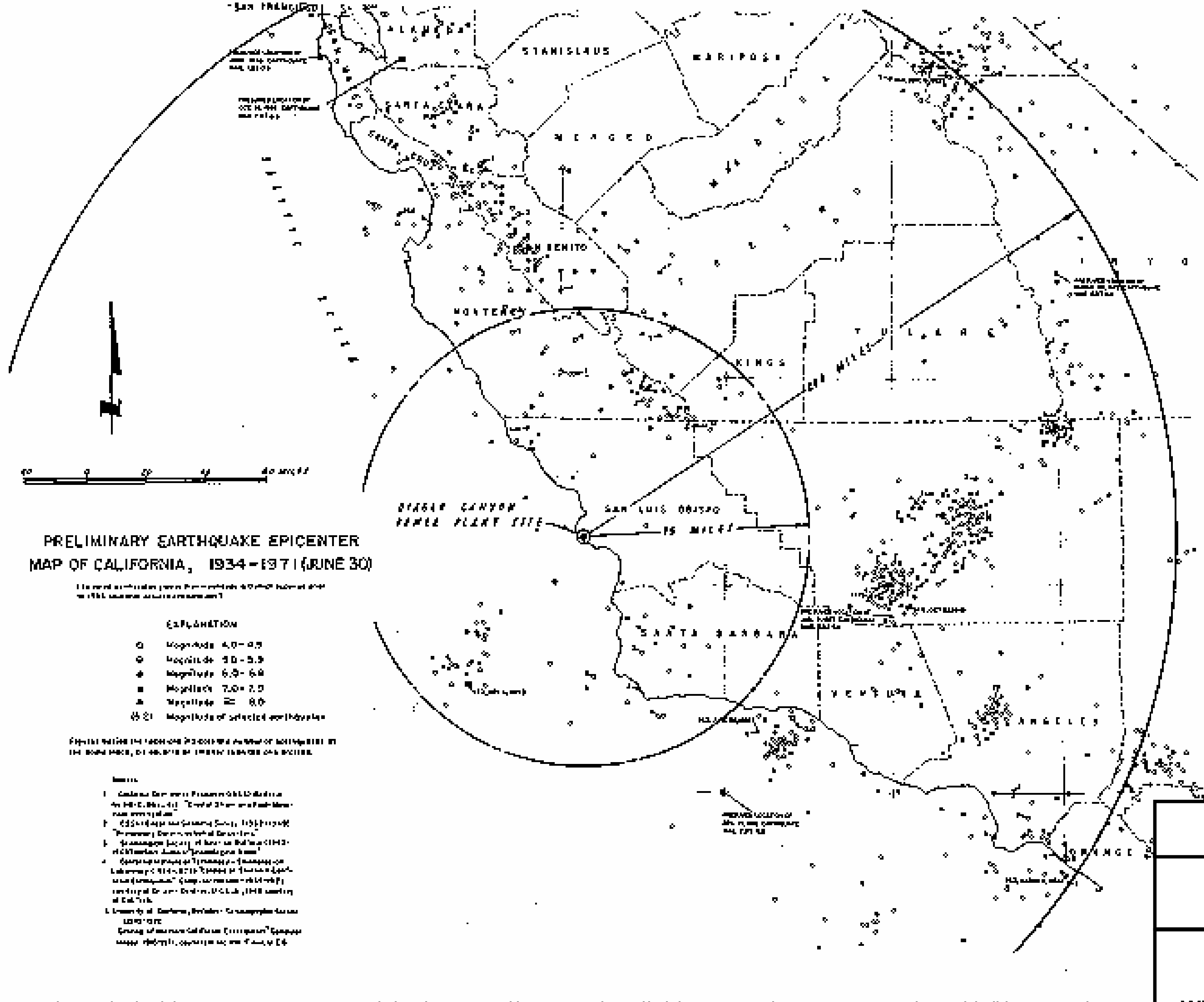


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-1

PLANT SITE LOCATION
AND TOPOGRAPHY

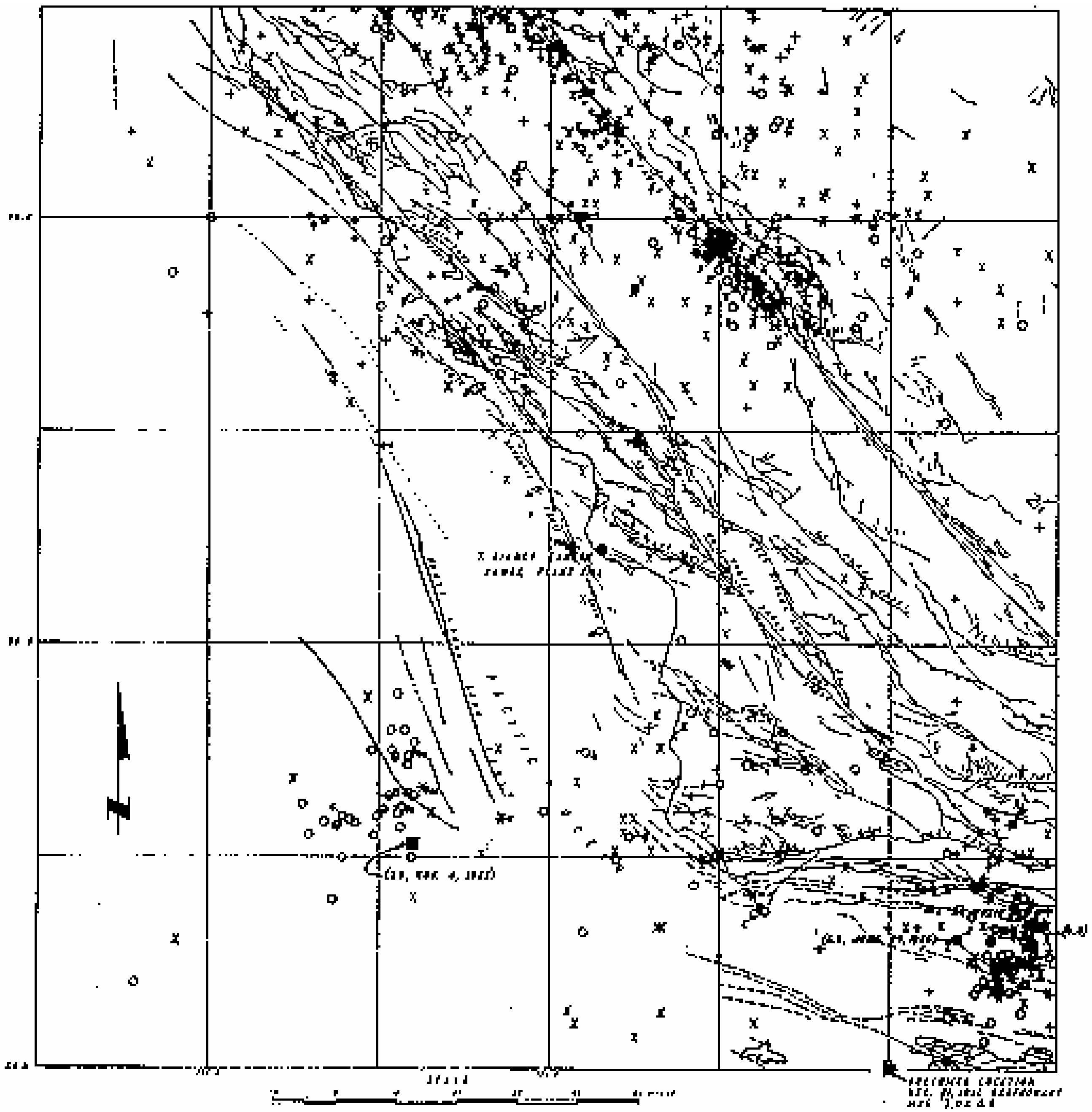


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-2
EARTHQUAKE EPICENTERS
WITHIN 200 MILES OF PLANT SITE

Revision 11 November 1996



EXPLANATION

EARTHQUAKE EPICENTER DATA

NOTES FOR REVISED FAULT AND EPICENTER DATA

A. FAULT DATA REVISED IN ACCORDANCE WITH NOTE 4, FIGURE 5, APPENDIX 2.5D

B. EPICENTERS OF 9 SELECTED EARTHQUAKES, RECOMPUTED BY S. W. SMITH (1974). EPICENTER–MAGNITUDE SYMBOL OF THESE EVENTS IS INDICATED BY HORIZONTAL DASHES. NUMBER SUBSCRIPT INDICATES EVENT NUMBER

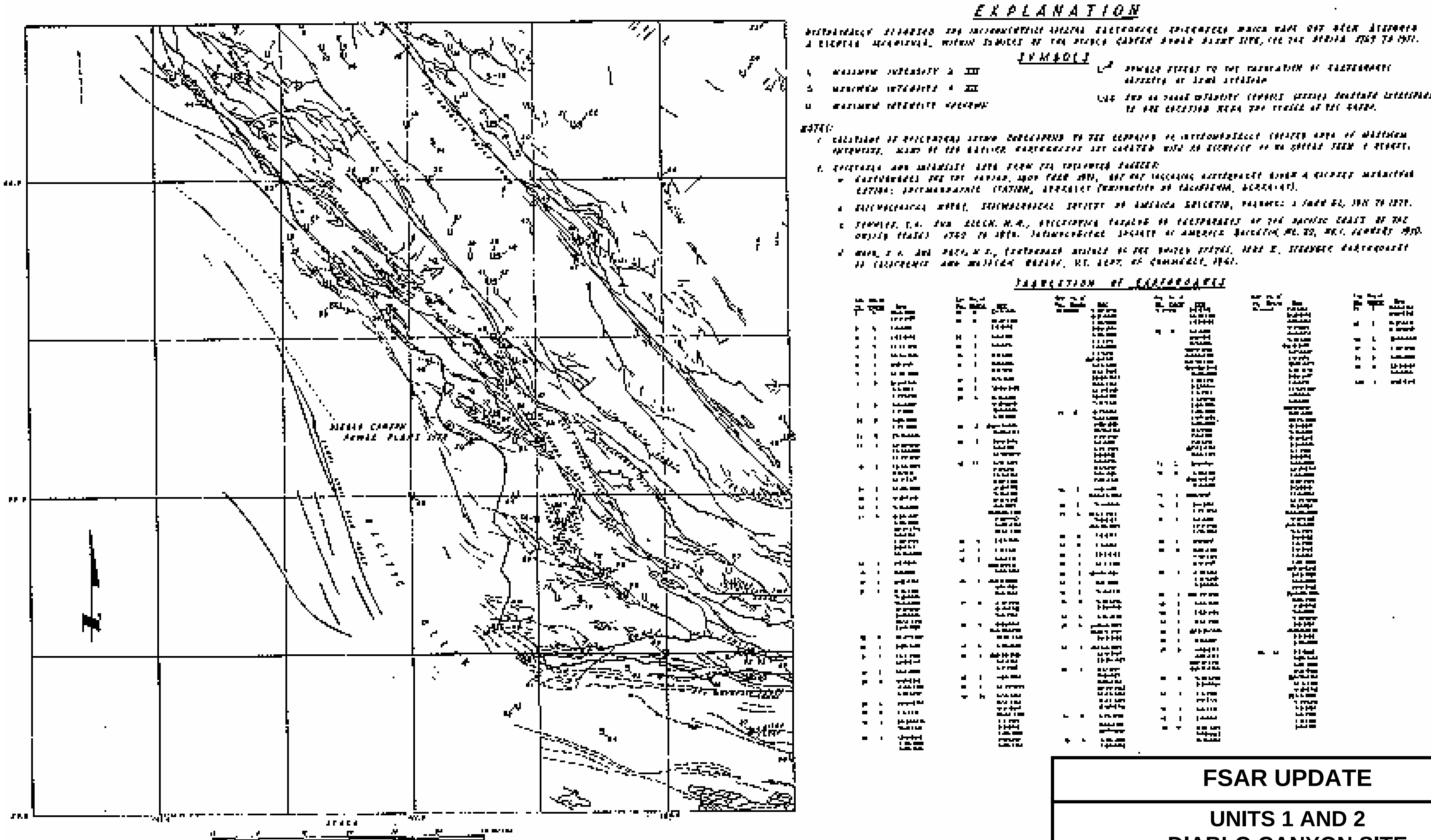
C. EPICENTERS MAGNITUDE ≥ 4.0 RECORDED AND LOCATED BY WILLIAM GAWTHROP (1975), AS DESCRIBED IN "PRELIMINARY REPORT ON A SHORT TERM SEISMIC STUDY OF THE SAN LUIS OBISPO REGION IN MAY, 1973."

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-3
FAULTS AND EARTHQUAKE EPICENTERS
WITHIN 75 MILES OF PLANT SITE
(FOR EARTHQUAKES WITH ASSIGNED MAGNITUDES)

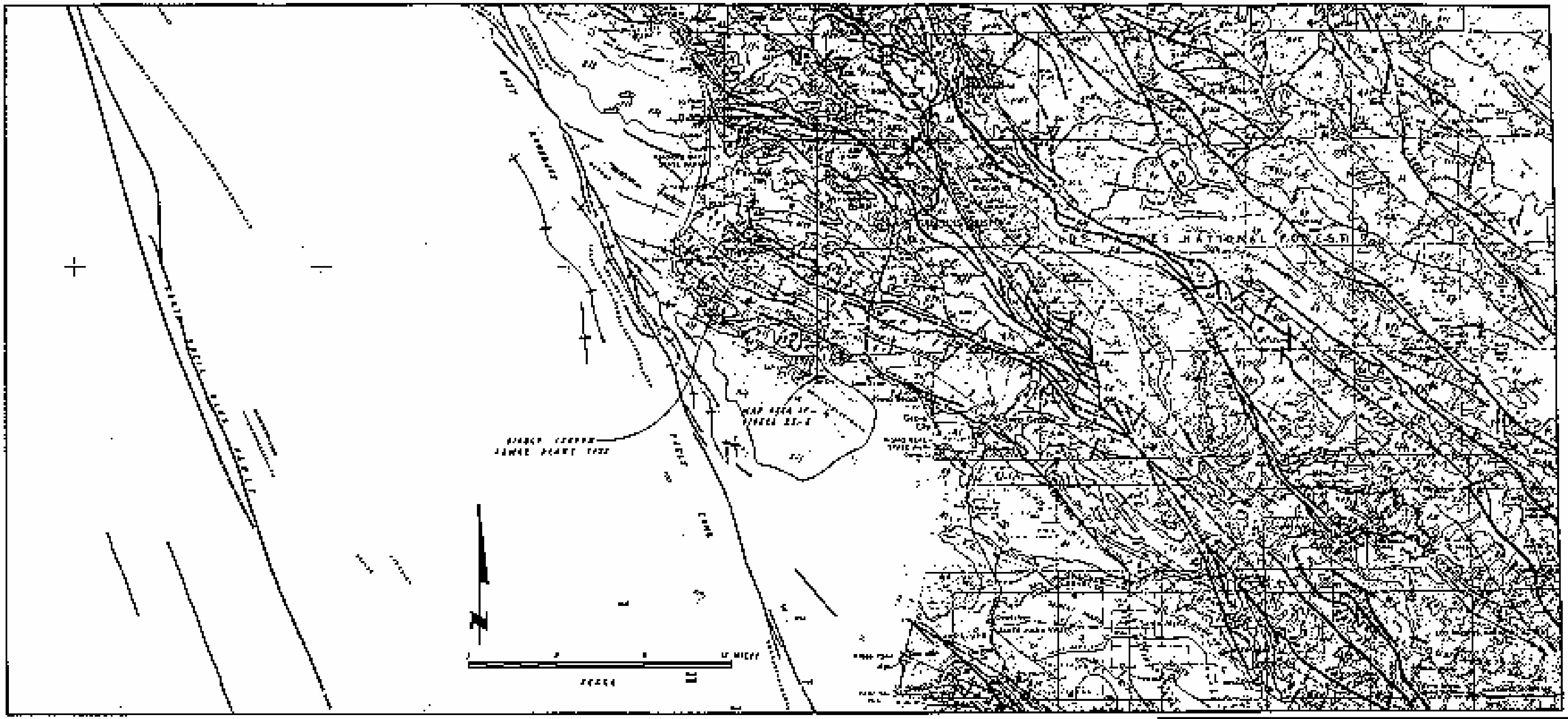
Revision 11 November 1996



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-4
FAULTS AND EARTHQUAKE EPICENTERS WITHIN 75 MILES OF PLANT SITE (FOR EARTHQUAKES WITH ASSIGNED INTENSITIES ONLY)



FSAR UPDATE

**UNITS 1 AND 2
DIABLO CANYON SITE**

**FIGURE 2.5-5
GEOLOGIC AND TECTONIC MAP OF
SOUTHERN COAST RANGES IN THE
REGION OF PLANT SITE
(SHEET 1 OF 2)**

EXPLANATION

GEOLOGIC UNITS

CENOZOIC

SEDIMENTARY DEPOSITS AND ROCKS

- **Q** HOLOCENE AND PLEISTOCENE NONMARINE (CHIEFLY ALLUVIAL) AND MARINE DEPOSITS, UNDIVIDED.
- **Qls** LANDSLIDE, WHERE MAPPED SEPARATELY.
- **Qs** DUNE SAND DEPOSITS, WHERE MAPPED SEPARATELY.
- **QPc** PLIO-PLEISTOCENE AND PLIOCENE NONMARINE.
- **P** PLIOCENE MARINE
- **M** MIOCENE MARINE
- **Φc** OLIGOCENE NONMARINE
- **E** EOCENE MARINE
- **Ep** PALEOCENE MARINE

VOLCANIC ROCKS

- **Tv** TERTIARY VOLCANIC ROCKS
- **Tvp** TERTIARY PYROCLASTIC ROCKS, (INCLUDING VOLCANIC MUDFLOW DEPOSITS).
- **Ti** TERTIARY INTRUSIVE ROCKS.

MESOZOIC

SEDIMENTARY AND METAMORPHIC ROCKS

- **Ku** UPPER CRETACEOUS MARINE ROCKS
- **Kl** LOWER CRETACEOUS MARINE ROCKS
- **KJf** FRANCISCAN ASSEMBLAGE (PREDOMINANTLY SEDIMENTARY AND META-SEDIMENTARY ROCKS. INCLUDES FRANCISCAN MELANGE)
- **J** JURASSIC MARINE ROCKS (INCLUDES KNOXVILLE FORMATION)
- **m** METAMORPHIC ROCKS OF PRE-TERTIARY AGE, UNDIVIDED.

METAVOLCANIC ROCKS

- **Mzv** MESOZOIC METAVOLCANIC ROCKS (INCLUDING FRANCISCAN VOLCANIC ROCKS).

PLUTONIC ROCKS

- **grMz** MESOZOIC GRANITIC ROCKS
- **um** MESOZOIC ULTRAMAFIC ROCKS

INCLUDED IN "CORE COMPLEXES"

SYMBOLS

GEOLOGIC CONTACT, DASHED WHERE APPROXIMATE OR WHERE QUATERNARY DEPOSITS ARE INVOLVED.

FAULT (SOLID LINE WHERE LOCATION IS WELL DEFINED; DASHED LINE WHERE APPROXIMATE OR INFERRED; DOTTED WHERE CONCEALED.

ANTICLINAL AXIS – WITH PLUNGE INDICATED. (SOLID LINE WHERE LOCATION IS WELL DEFINED; DASHED WHERE APPROXIMATE OR INFERRED; DOTTED WHERE CONCEALED.

SYNCLINAL AXIS – WITH PLUNGE INDICATED. (SOLID LINE WHERE LOCATION IS WELL DEFINED; DASHED WHERE APPROXIMATE OR INFERRED; DOTTED WHERE CONCEALED.

NOTES

1. MAP BASE FROM PARTS OF SAN LUIS OBISPO AND SANTA MARIA SHEETS, U.S. GEOLOGICAL SURVEY 1:250,000 SCALE TOPOGRAPHIC MAP SERIES

2. GEOLOGIC DATA FROM:

   "GEOLOGIC MAP OF CALIFORNIA" 1959, SCALE 1:250,000; PRELIMINARY, UNPUBLISHED, CALIFORNIA DIVISION OF MINES AND GEOLOGY. (COMPILED BY C.W. JENNINGS)

   LOCALLY SUPPLEMENTED WITH DATA FROM:

   HEADLEE, L.A., 1965, GEOLOGY OF THE COASTAL PORTION OF THE SAN LUIS RANGE, SAN LUIS OBISPO COUNTY, CALIF., UNPUBLISHED M.S. THESIS, UNIV. SOUTHERN CALIF.

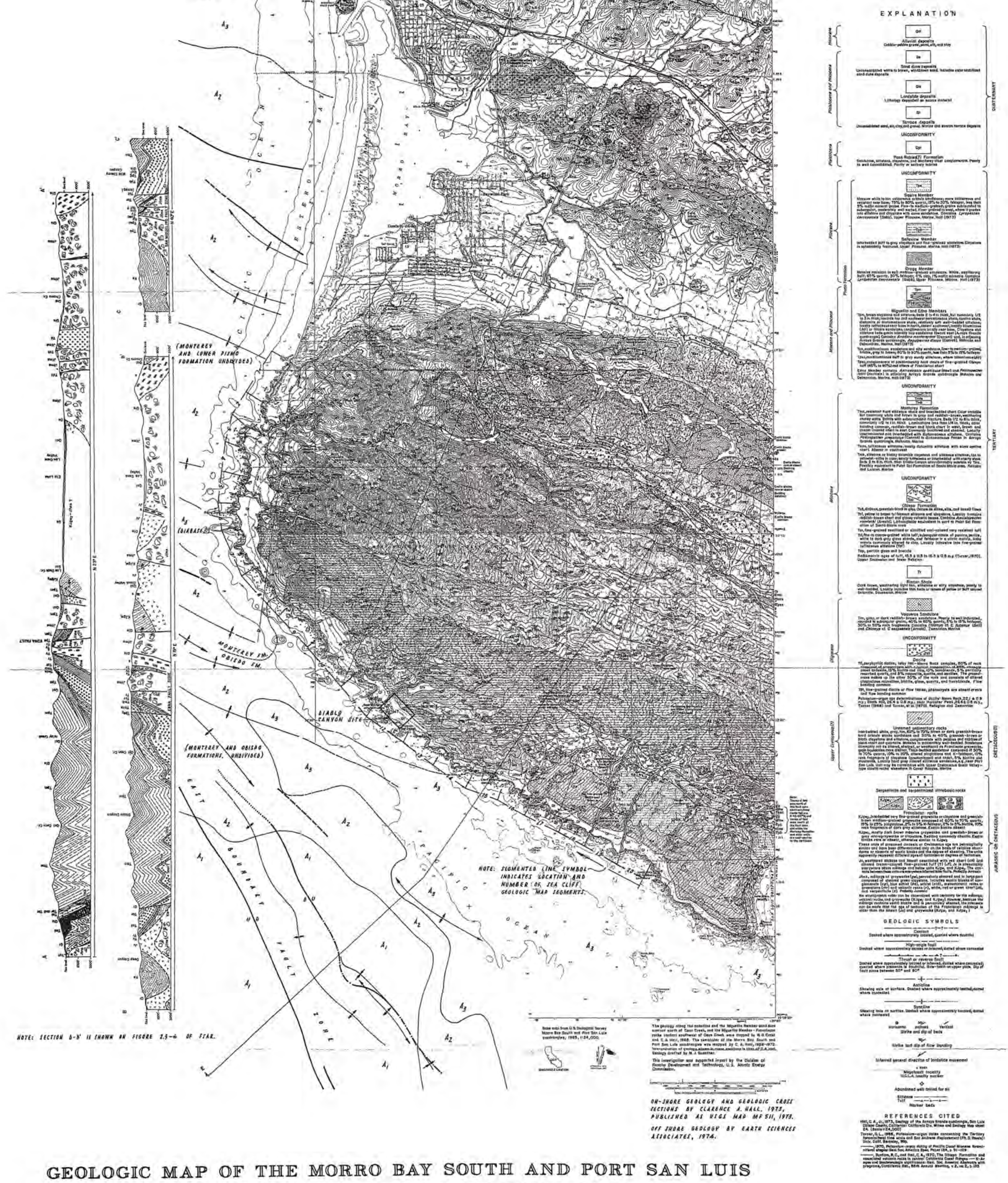
   JAHNS, R.H., 1966, GEOLOGY OF THE DIABLO CANYON POWER PLANT SITE, SAN LUIS OBISPO COUNTY, CALIFORNIA; 1967 SUPPLEMENTARY REPORTS I AND II; 1968 SUPPLEMENTARY REPORT III, UNPUBLISHED REPORTS TO THE PACIFIC GAS AND ELECTRIC COMPANY.

   HALL, C.A., 1973, U.S.G.S. MAP MF 511,

   EARTH SCIENCES ASSOCIATES, 1974, OFFSHORE INVESTIGATION.

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-5 GEOLOGIC AND TECTONIC MAP OF SOUTHERN COAST RANGES IN THE REGION OF PLANT SITE (SHEET 2 OF 2)

(SHEET 2 OF 2)



GEOLOGIC MAP OF THE MORRO BAY SOUTH AND PORT SAN LUIS QUADRANGLES, SAN LUIS OBISPO COUNTY, CALIFORNIA

EXPLANATION FOR GEOLOGIC MAP OF THE OFFSHORE AREA

MAP UNITS

MAP ACOUSTIC UNIT	STRATIGRAPHIC EQUIVALENT
A_1	UPPER PART OF PISMO FORMATION (ABOVE MIGUELITO AND EDNA NUMBERS).
A_2	MONTEREY FORMATION AND LOWER PART OF PISMO FORMATION (MIGUELITO AND EDNA NUMBERS), INCLUDES OBISPO FORMATION IN THE AREA OFFSHORE FROM DIABLO CANYON.
A_3	FRANCISCAN AND GREAT VALLEY SEQUENCE BASEMENT ROCKS, UNDIVIDED. LOCALLY MAY INCLUDE SOME OBISPO FORMATION.

MAP SYMBOLS

CONTACT, DASHED WHERE POORLY CONTROLLED.

FAULT, DASHED WHERE POORLY CONTROLLED.

BURIED FAULT

ANTICLINE

SYNCLINE

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5 – 6
GEOLOGIC MAP OF THE MORRO BAY SOUTH AND PORT SAN LUIS QUADRANGLES, SAN LUIS OBISPO COUNTY, CALIFORNIA, AND ADJACENT OFFSHORE AREA.
Revision 11 November 1996

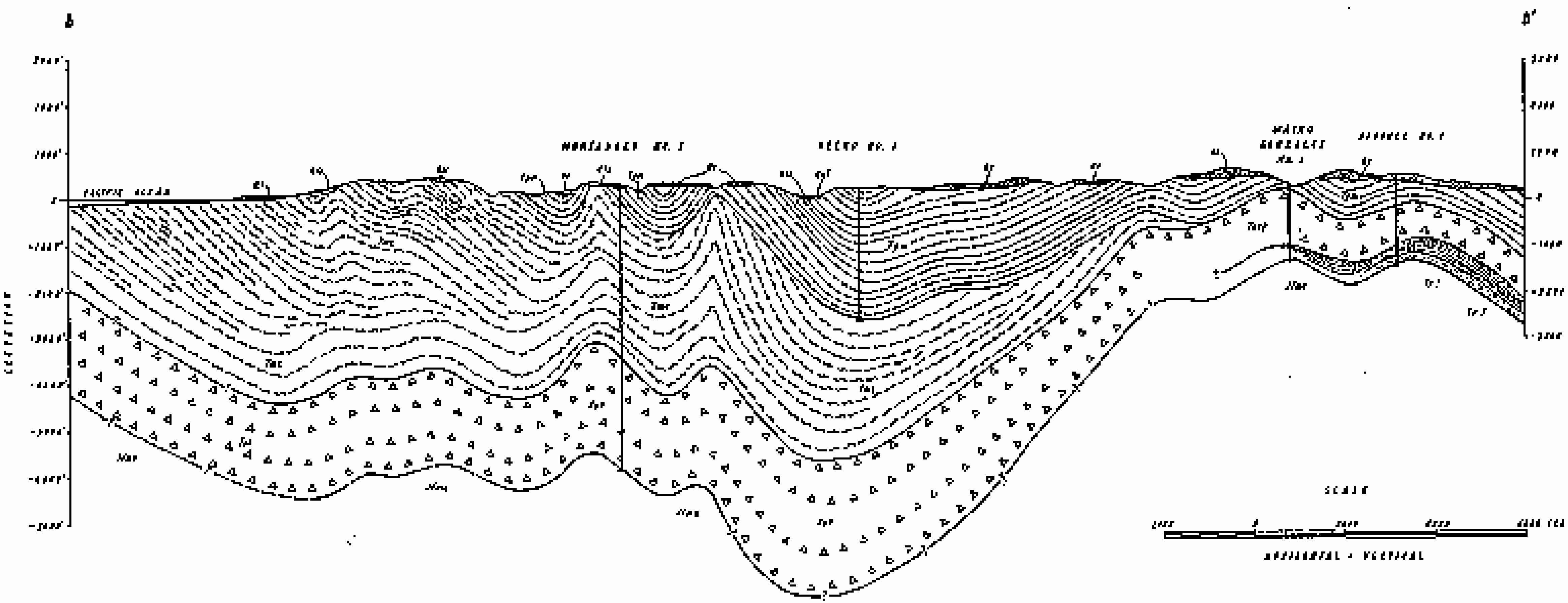
(ORIGINALLY: PLATE VIII APPENDIX 2.5D)

FSAR UPDATE

UNITS 1 AND 2 DIABLO CANYON SITE

FIGURE 2.5 – 6

GEOLOGIC MAP OF THE MORRO BAY SOUTH AND PORT SAN LUIS QUADRANGLES, SAN LUIS OBISPO COUNTY, CALIFORNIA, AND ADJACENT OFFSHORE AREA.



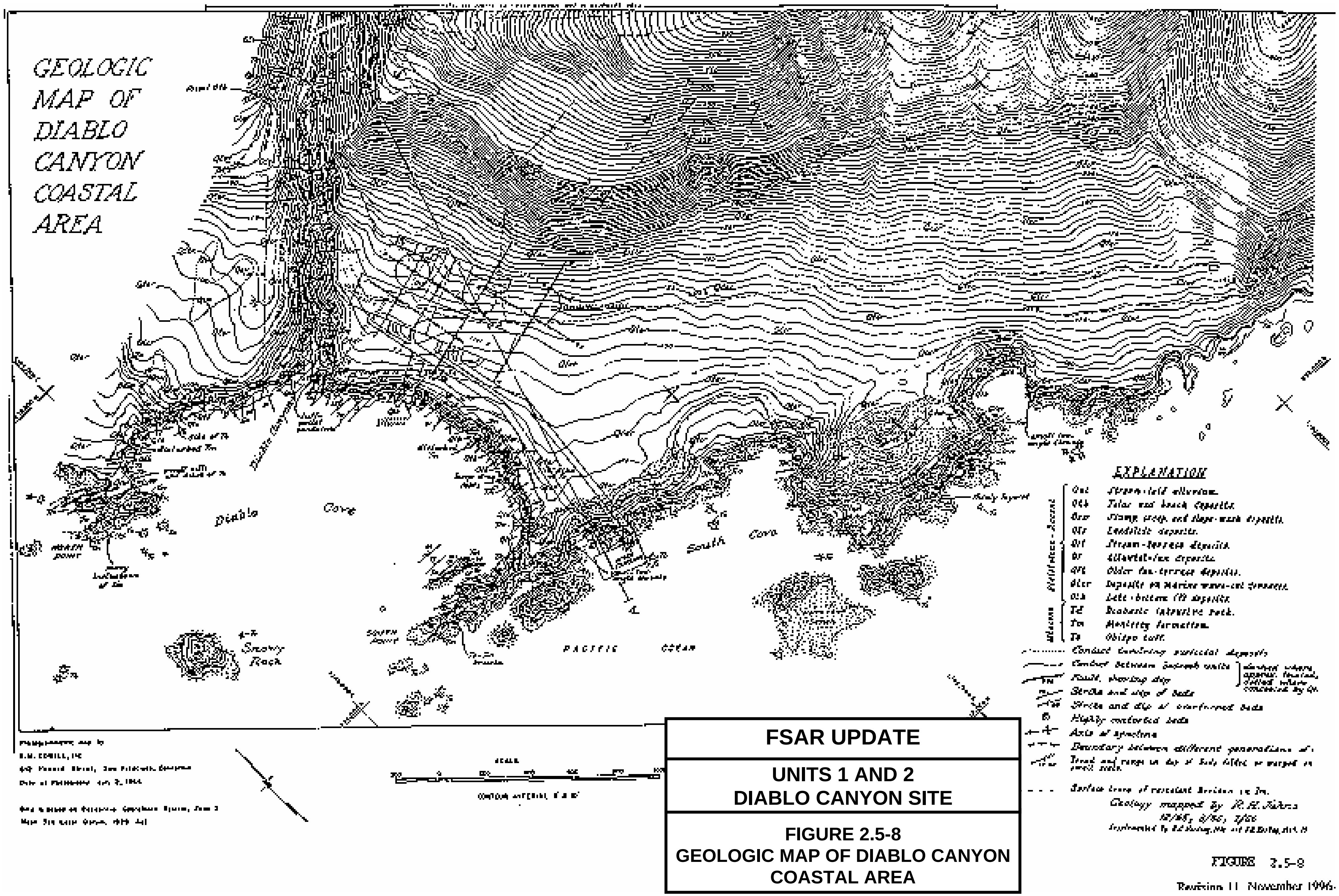
FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-7

GEOLOGIC SECTION THROUGH EXPLORATORY OIL WELLS IN THE SAN LUIS RANGE

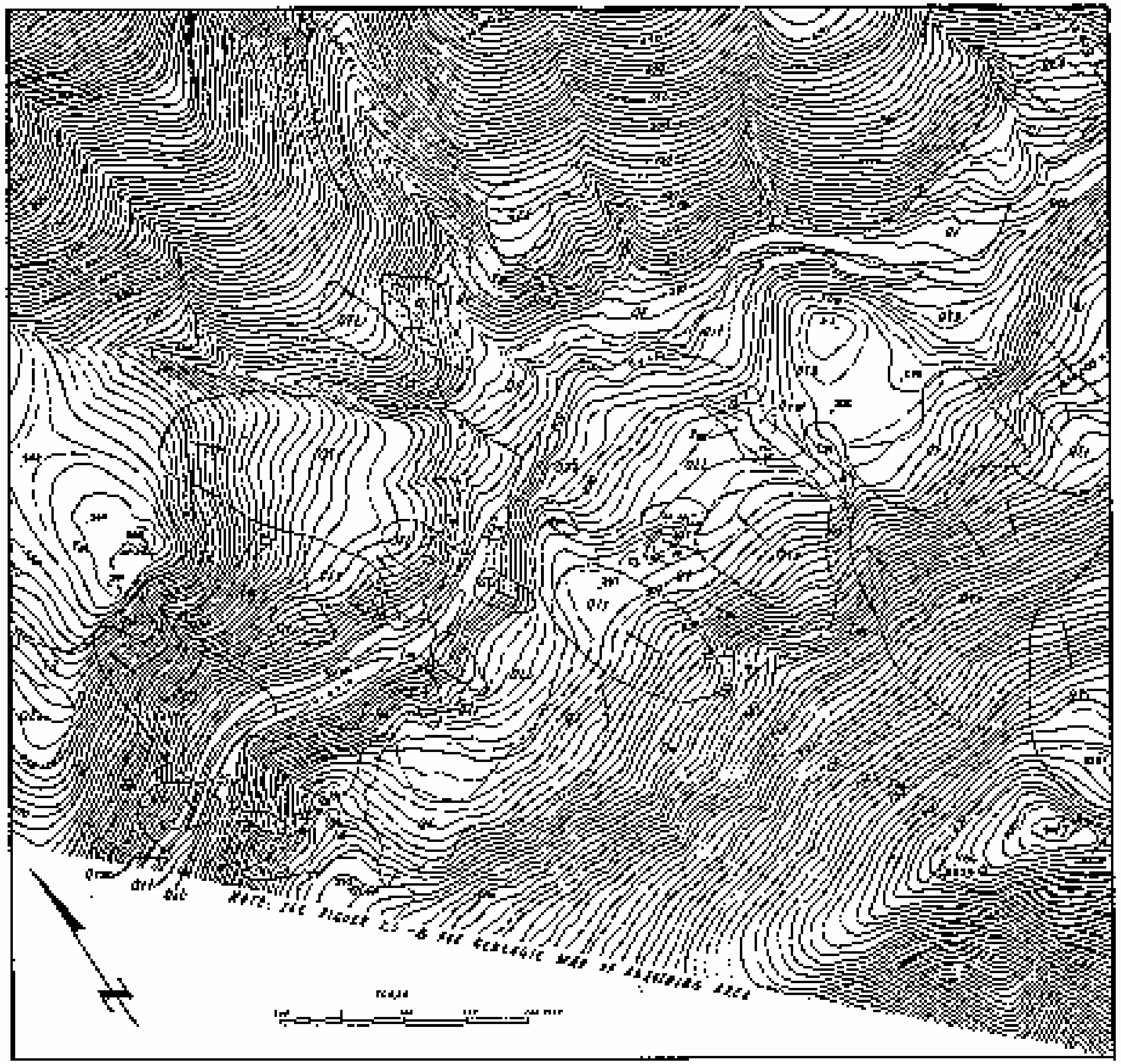
Revision 11 November 1996



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-8
GEOLOGIC MAP OF DIABLO CANYON
COASTAL AREA



EXPLANATION

PLEISTOCENE – RECENT

- Qal STREAM-LAID ALLUVIUM
- Qsw SLUMP, CREEP, AND SLOPE-WASH DEPOSITS
- Qls LANDSLIDE DEPOSITS
- Qst STREAM-TERRACE DEPOSITS
- Qf ALLUVIAL-FAN DEPOSITS
- Qft OLDER FAN-TERRACE DEPOSITS
- Qter DEPOSITS ON MARINE WAVE-CUT TERRACES.
- Qlb LAKE-BOTTOM (?) DEPOSITS

MIOCENE

- Td DIABASIC INTRUSIVE ROCK
- Tm MONTEREY FORMATION.

CONTACT INVOLVING SURFICIAL DEPOSITS.

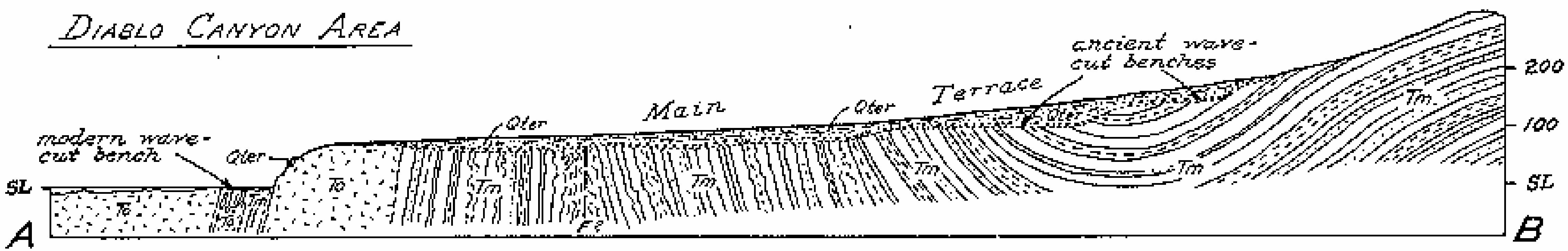
CONTACT BETWEEN BEDROCK UNITS. DASHED WHERE APPROX. LOCATED; DOTTED WHERE CONCEALED

70 STRIKE AND DIP OF BEDS

40-70 TREND AND RANGE IN DIP OF BEDS FOLDED OR WARPED ON A SMALL SCALE.

GEOLOGY MAPPED BY R.H. JAHNS AND A.M. JOHNSON, 1967, SUPPLEMENTED BY 1969.

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-9 GEOLOGIC MAP OF SWITCHYARD AREA

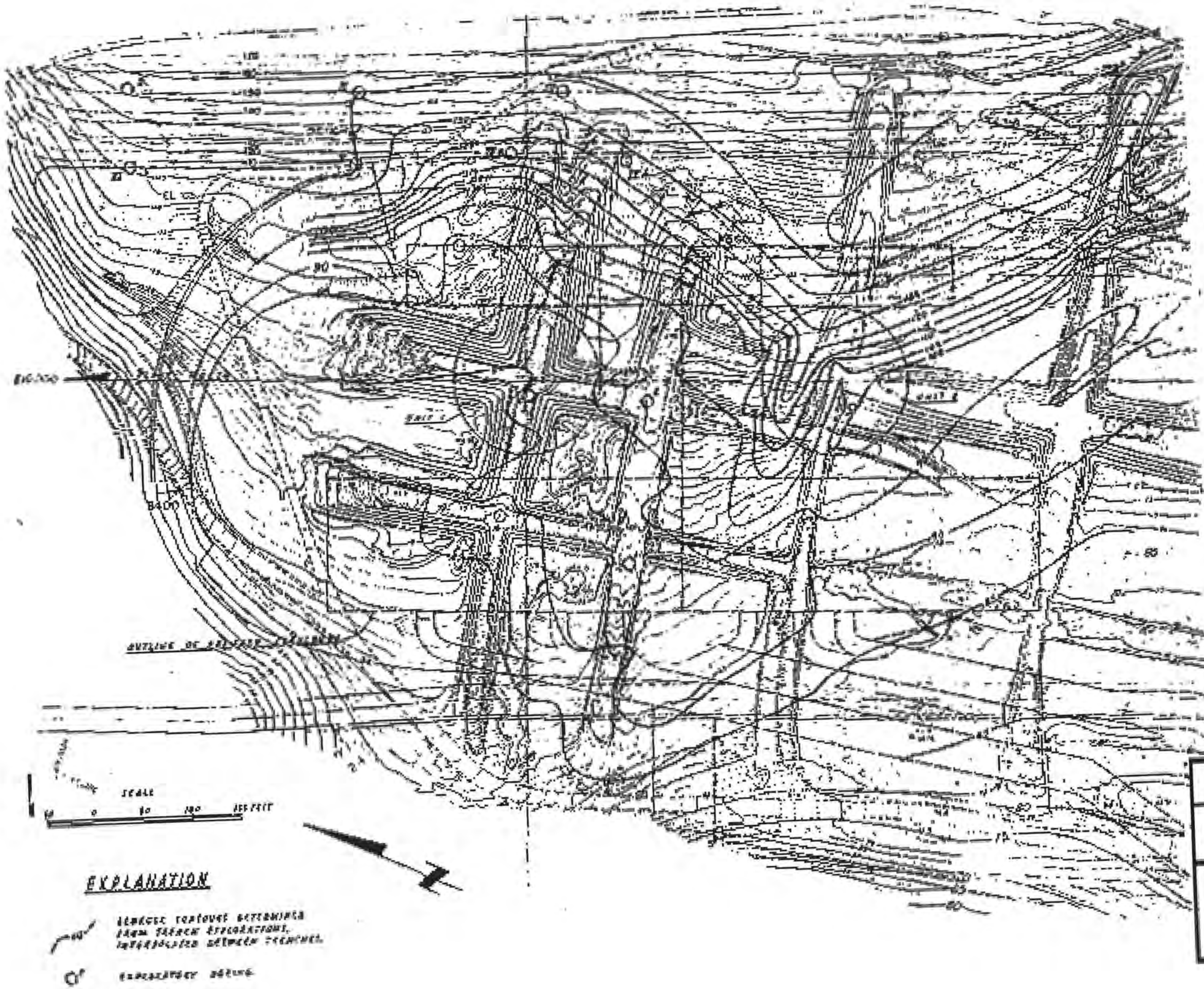


Section along Line A-B (Figure 2.5-8)

See geologic map for explanation of symbols

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-10 GEOLOGIC SECTION THROUGH THE PLANT SITE

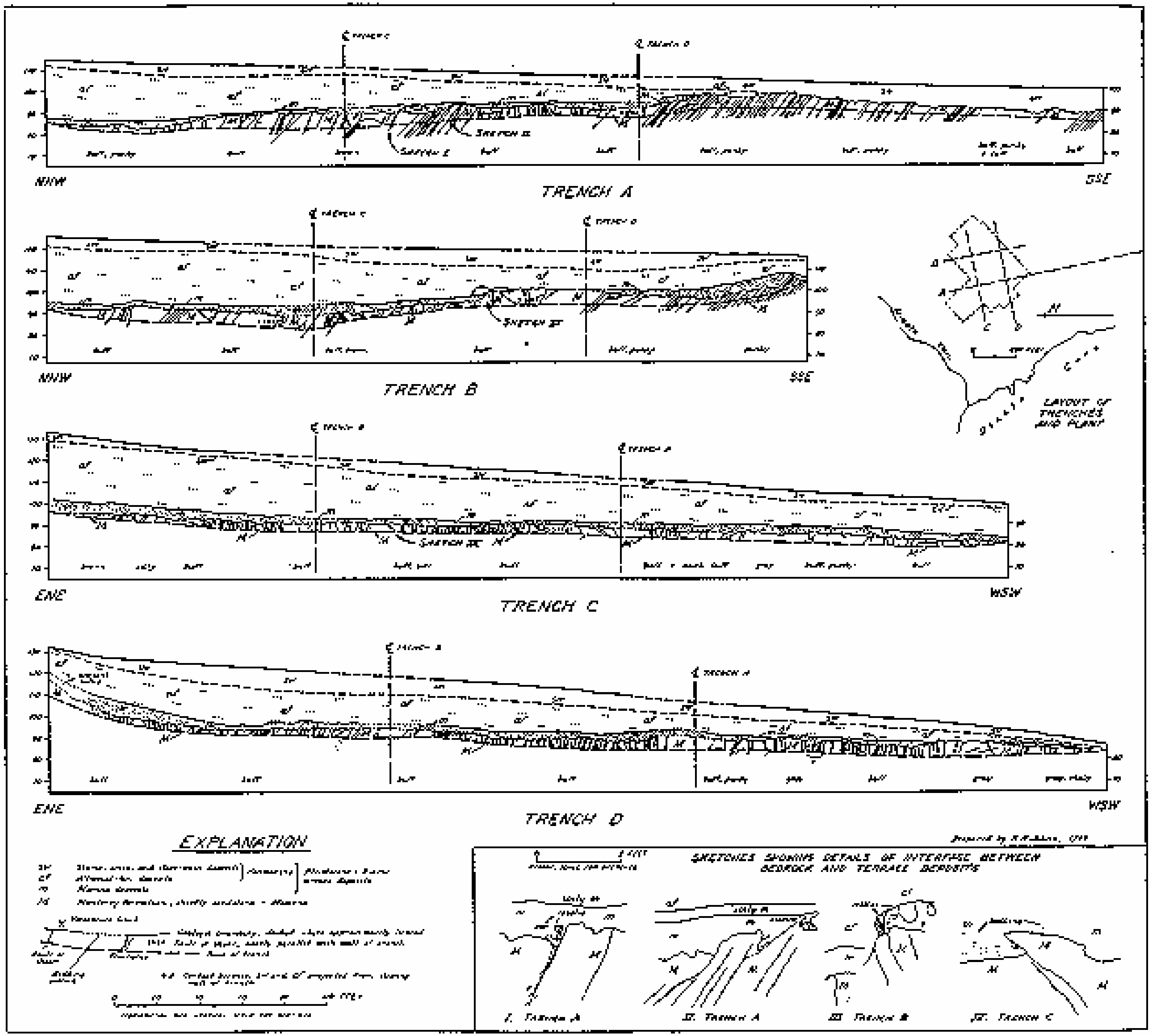
Revision 11 November 1996



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-11
SITE EXPLORATION FEATURES AND BEDROCK CONTOURS

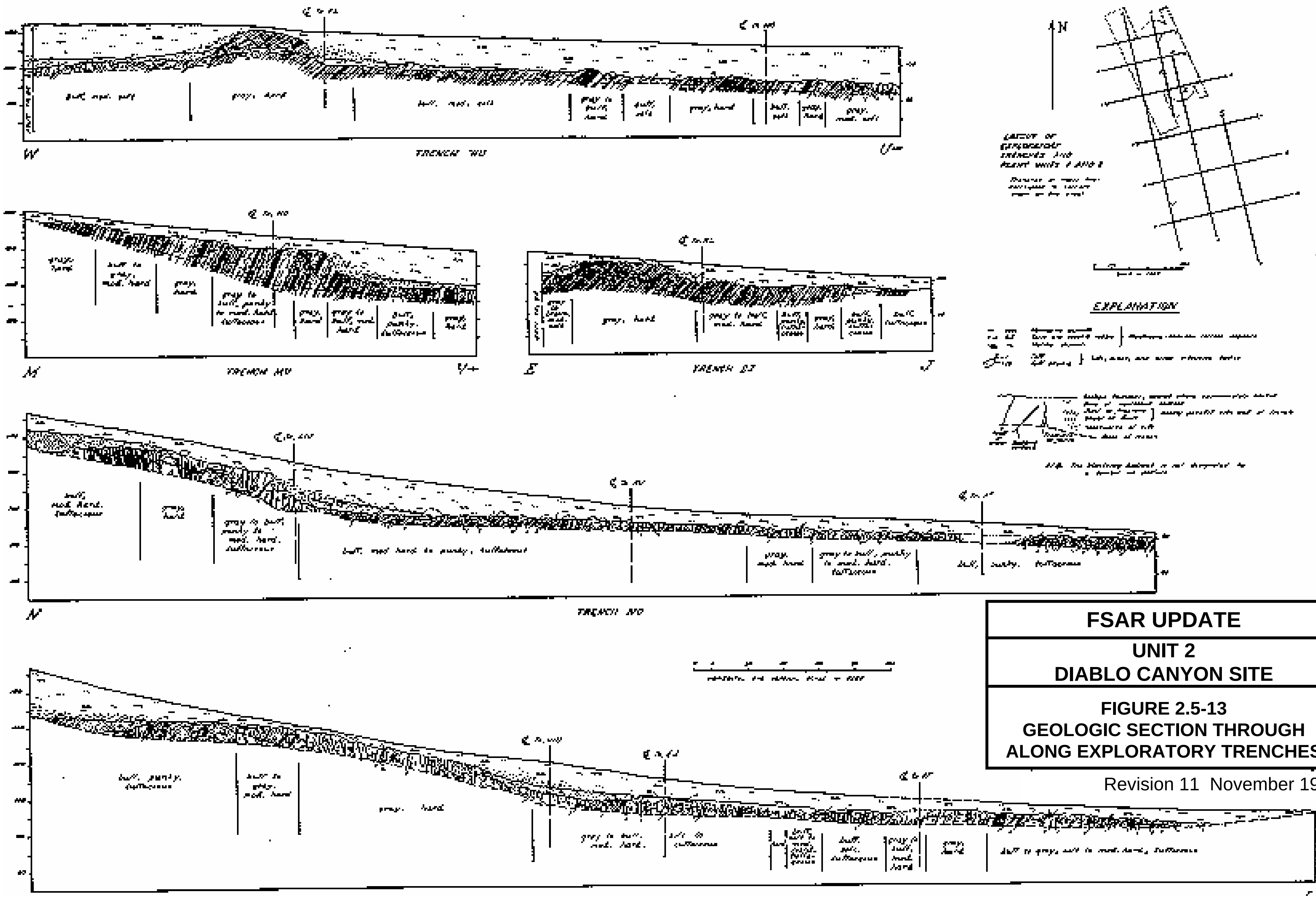


FSAR UPDATE

UNIT 1
DIABLO CANYON SITE

FIGURE 2.5-12
GEOLOGIC SECTIONS AND SKETCHES ALONG EXPLORATORY TRENCHES

Revision 11 November 1996

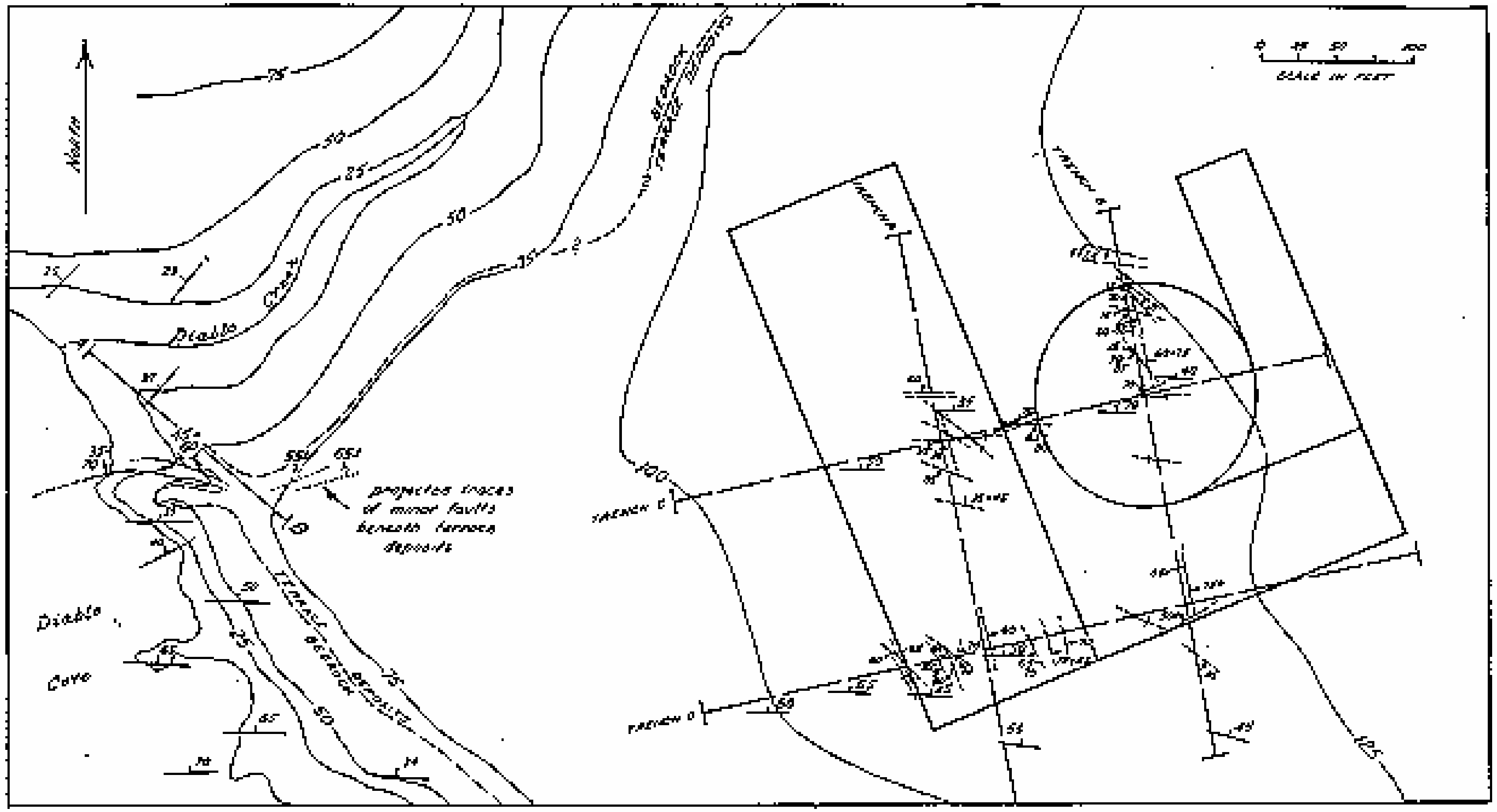


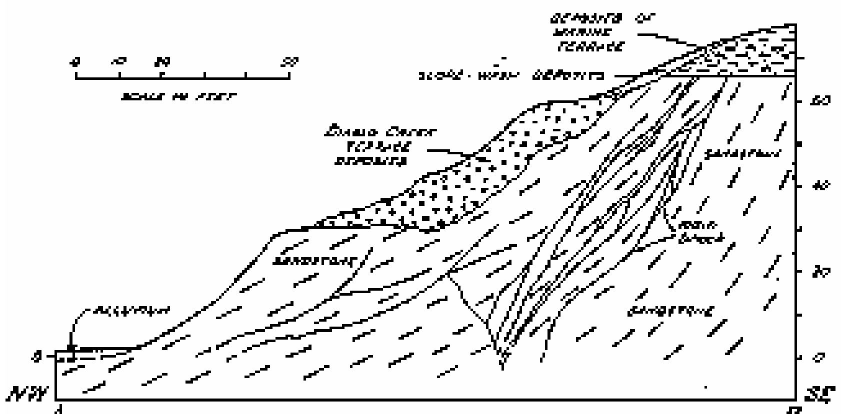
FSAR UPDATE

UNIT 2
DIABLO CANYON SITE

FIGURE 2.5-13
GEOLOGIC SECTION THROUGH
ALONG EXPLORATORY TRENCHES

Revision 11 November 1996





Section along line A-B

EXPLANATION

Strike and dip of beds

Generalized trace of bedding (in section only)

Fault or shear, showing dip and trend and plunge of slickensides

Vertical fault or shear

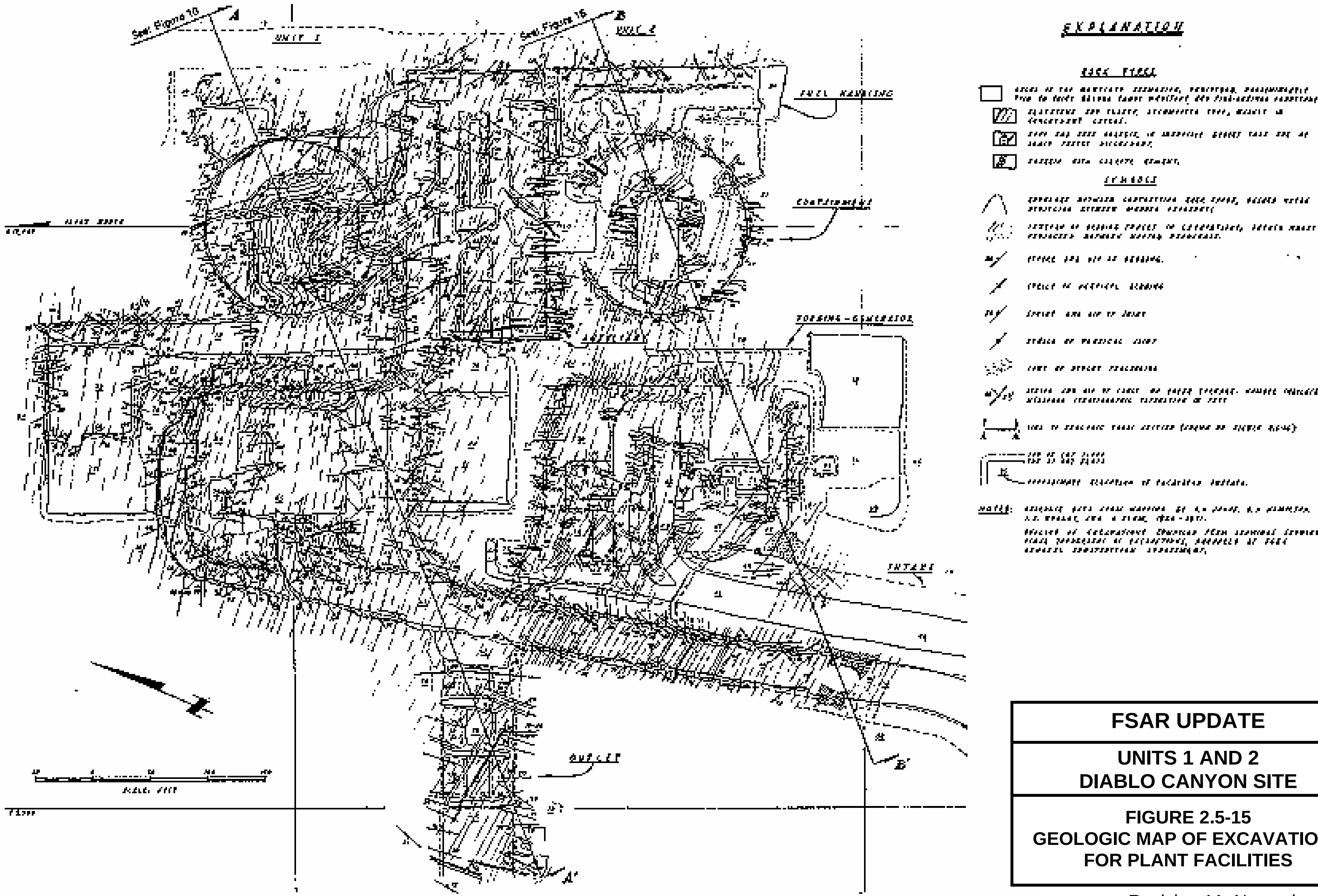
Center line of exploratory trench

Outline of power plant structure

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

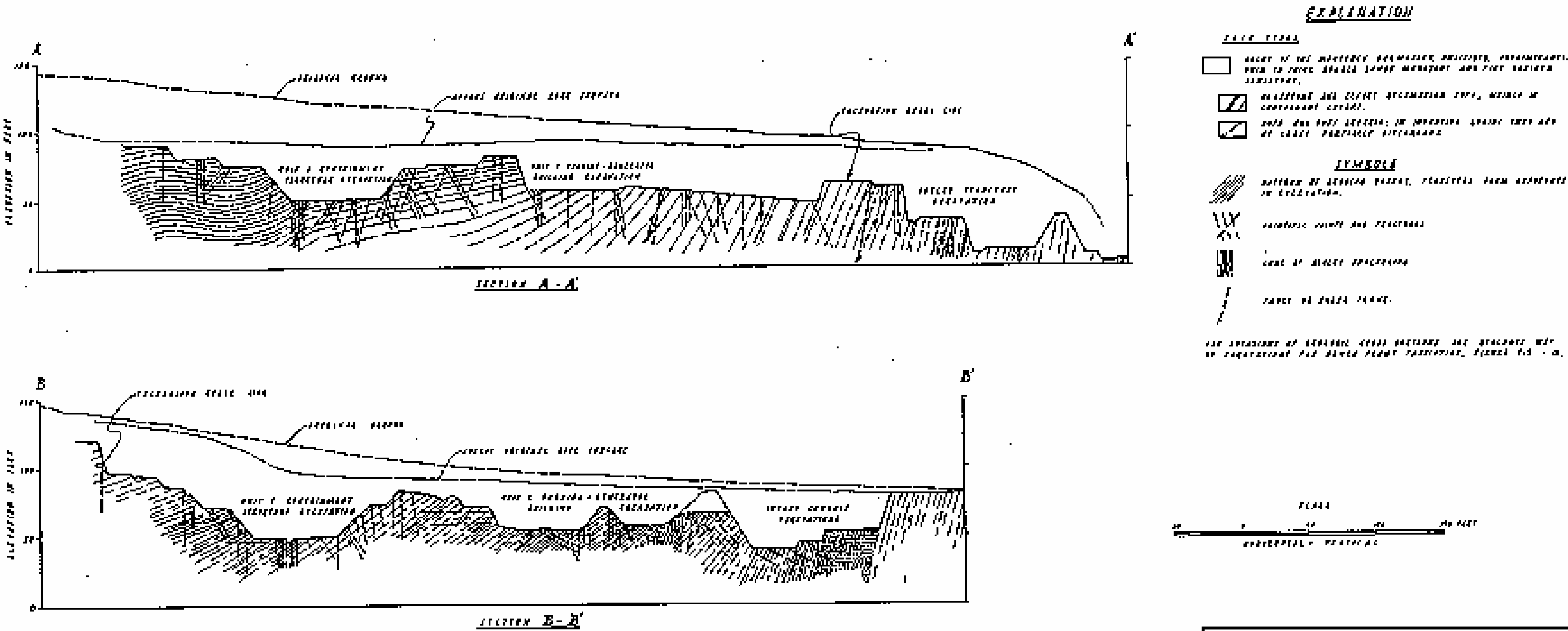
FIGURE 2.5-14
RELATIONSHIPS OF FAULTS AND SHEARS AT PLANT SITE



FSAR UPDATE

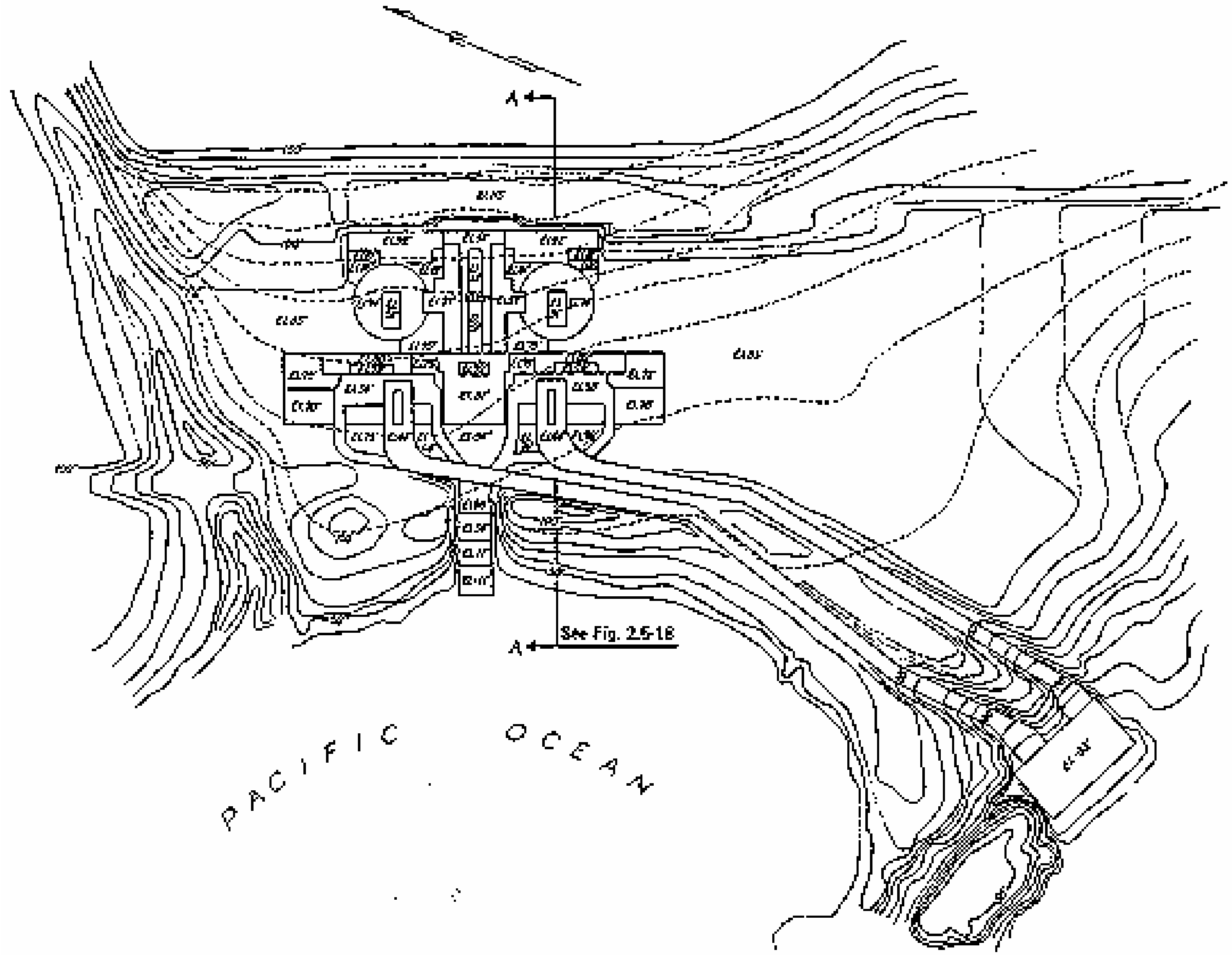
UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-15
GEOLOGIC MAP OF EXCAVATIONS
FOR PLANT FACILITIES



FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-16 GEOLOGIC SECTIONS THROUGH EXCAVATIONS FOR PLANT FACILITIES

Revision 11 November 1996

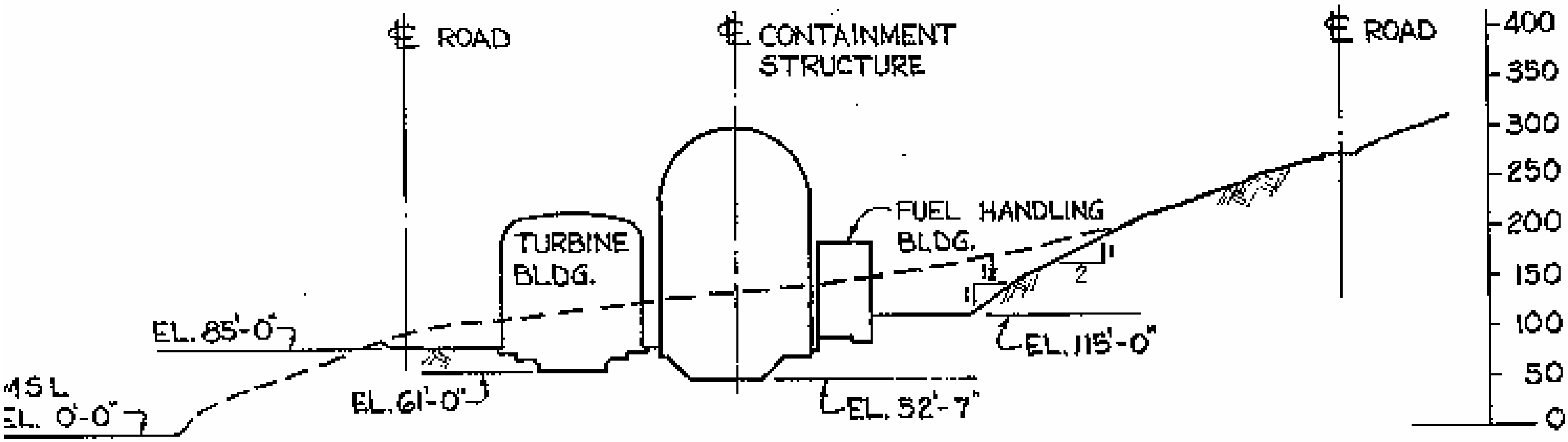


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-17

PLAN OF EXCAVATION AND BACKFILL

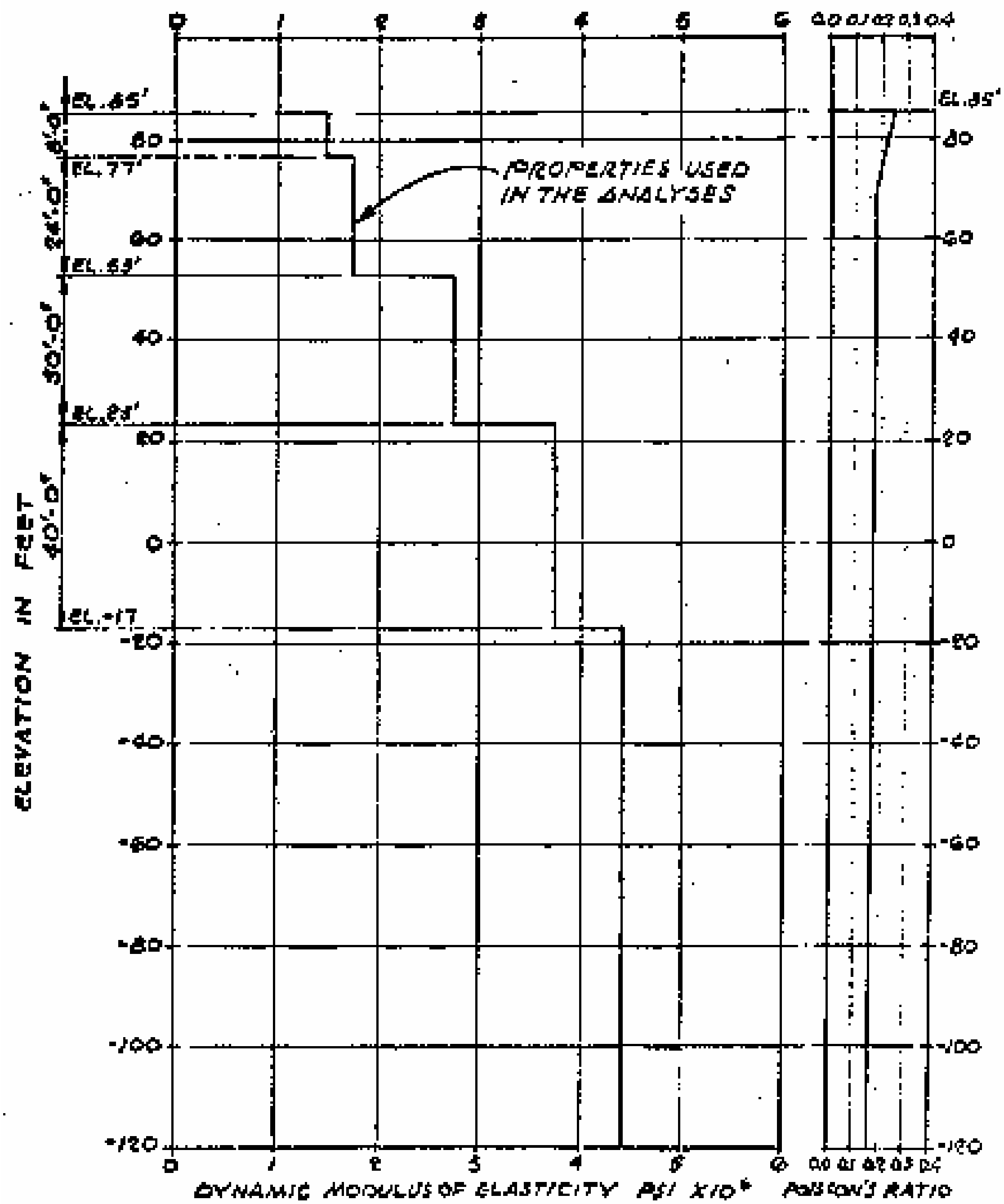


SECTION A-A

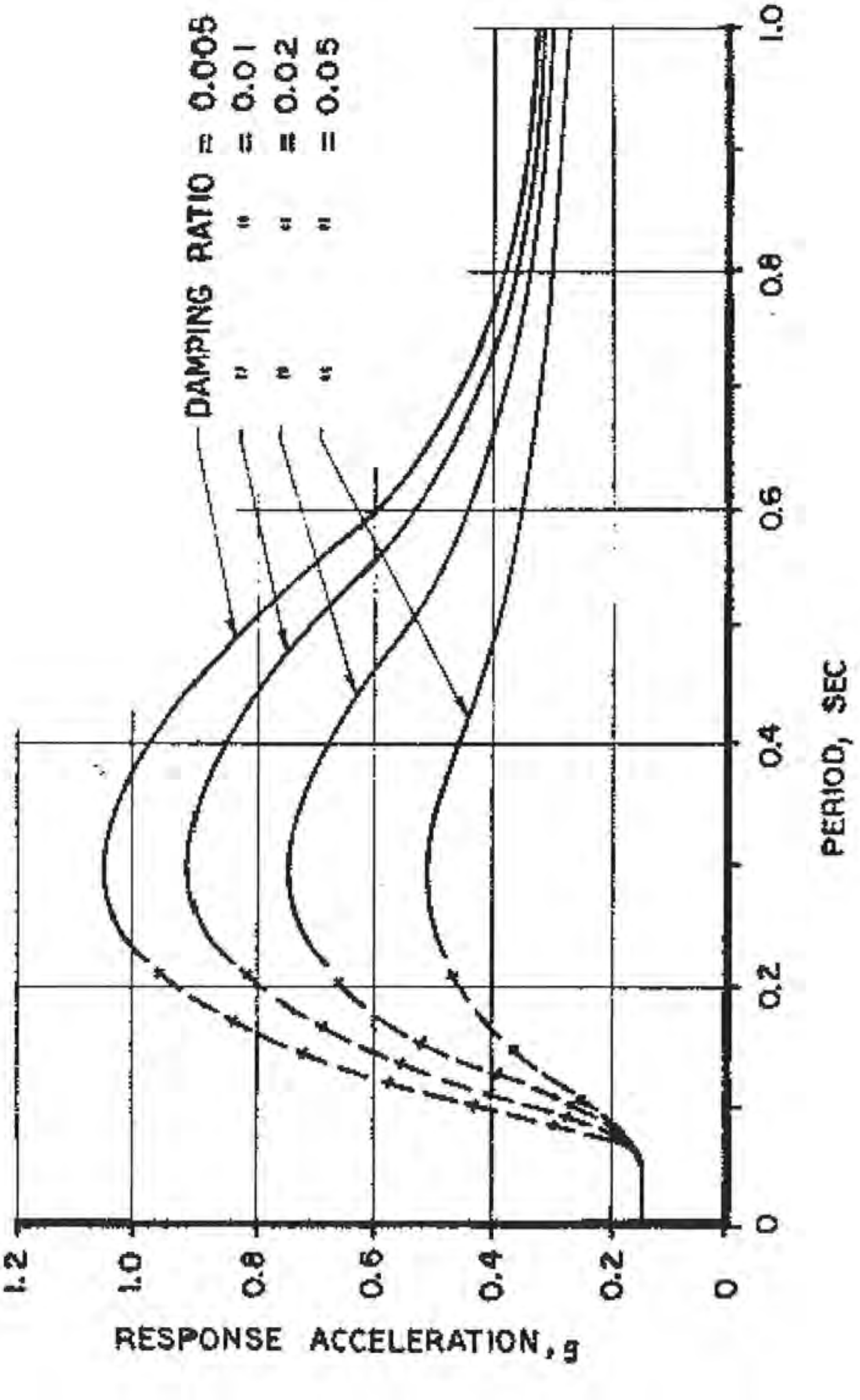
FROM FIGURE 2.5-17

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-18 SECTION A-A EXCAVATION AND BACKFILL

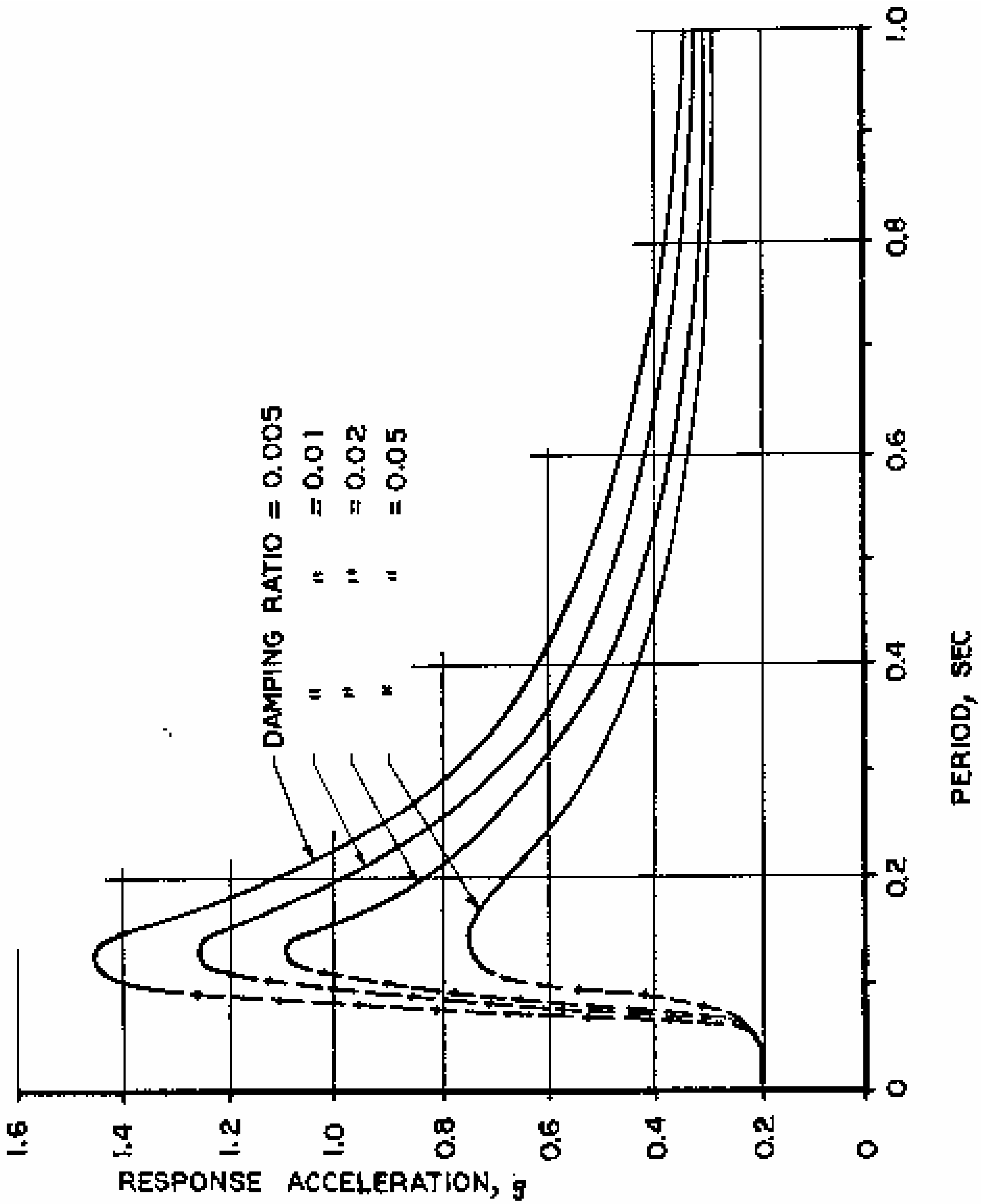
Revision 11 November 1996



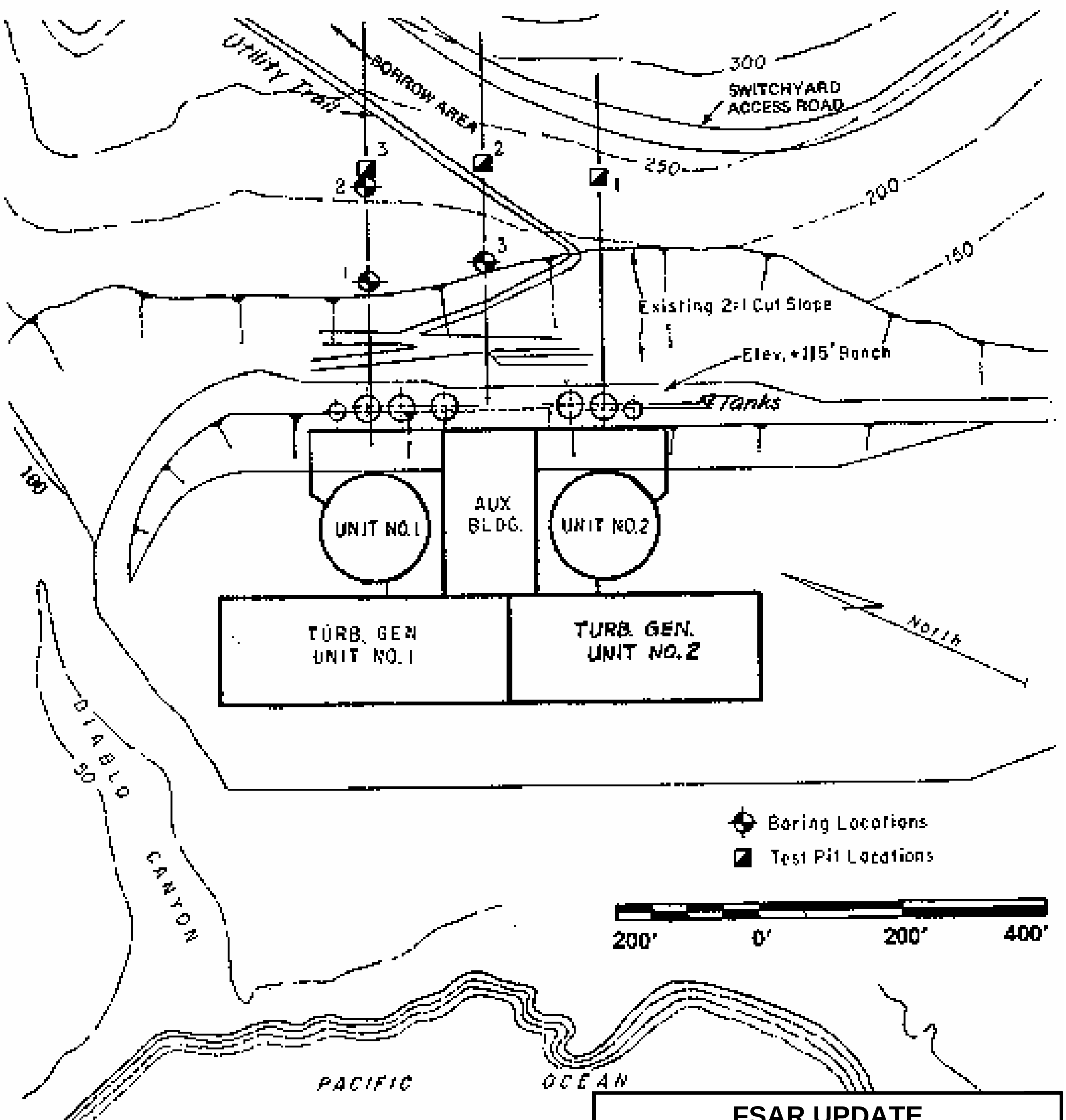
FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-19 SOIL MODULE OF ELASTICITY AND POISSON'S RATIO



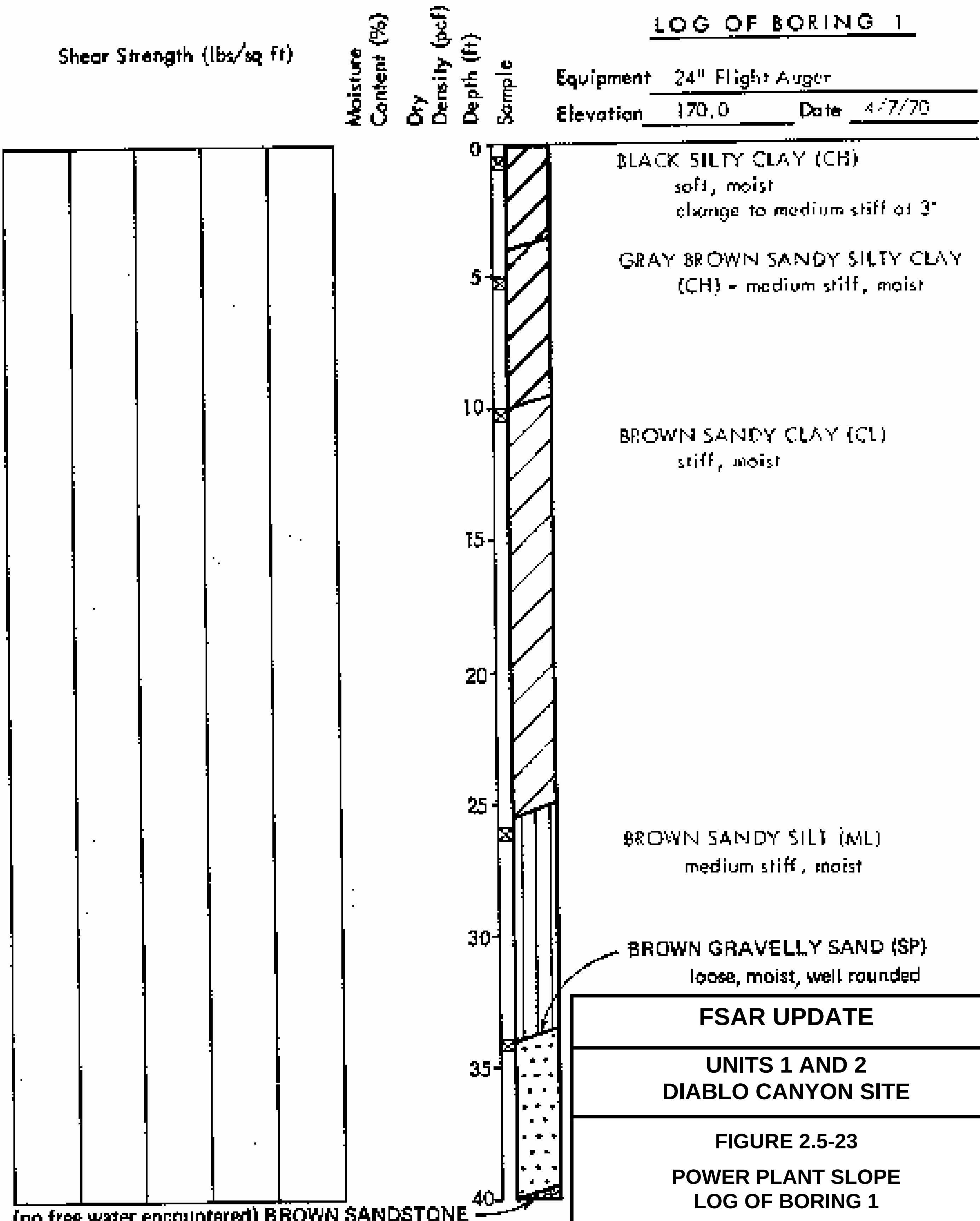
FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-20 SMOOTH RESPONSE ACCELERATION SPECTRA – EARTHQUAKE "B"



FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-21 SMOOTH RESPONSE ACCELERATION SPECTRA – EARTHQUAKE "D" MODIFIED



FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-22 POWER PLANT SLOPE PLAN

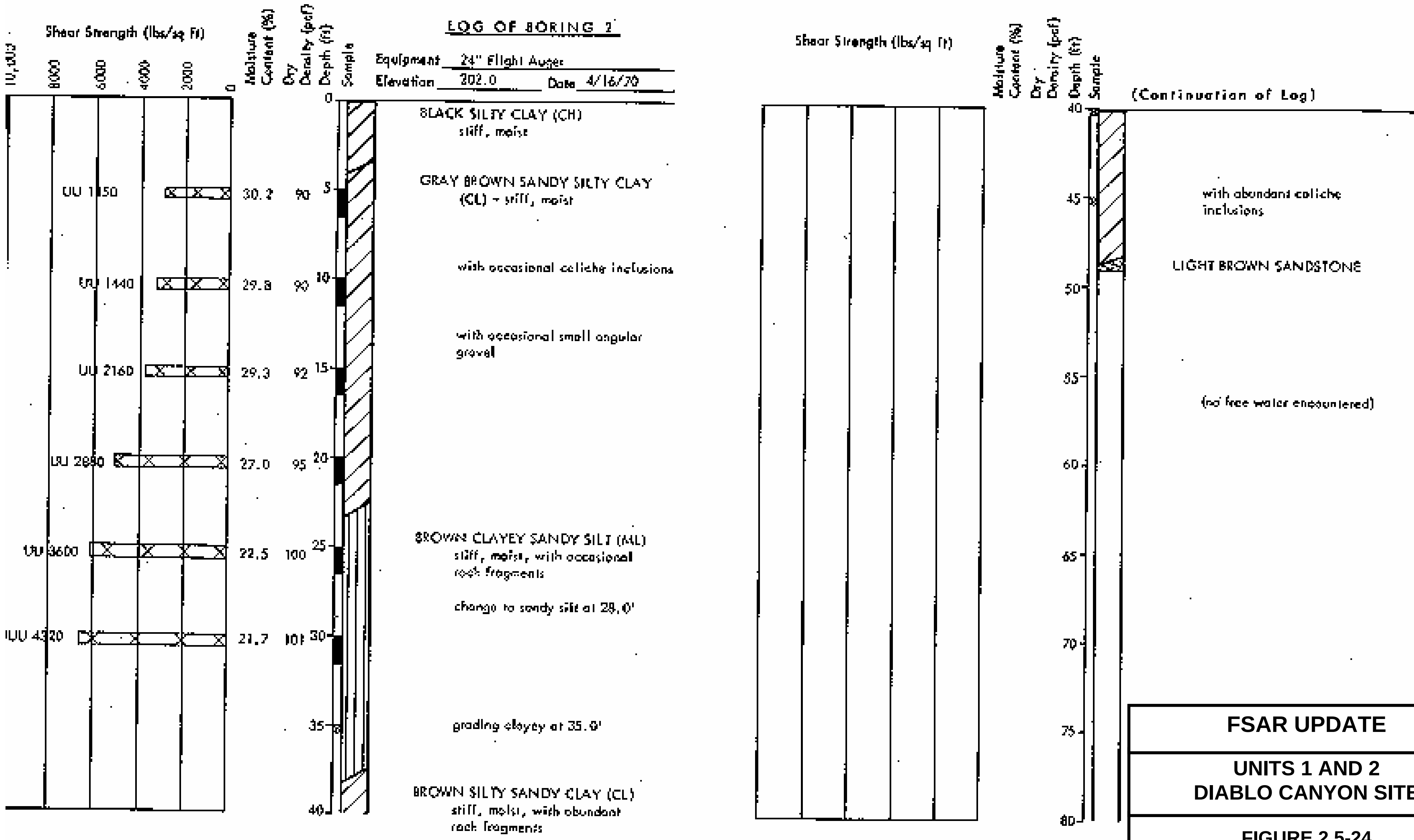


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-23

POWER PLANT SLOPE
LOG OF BORING 1



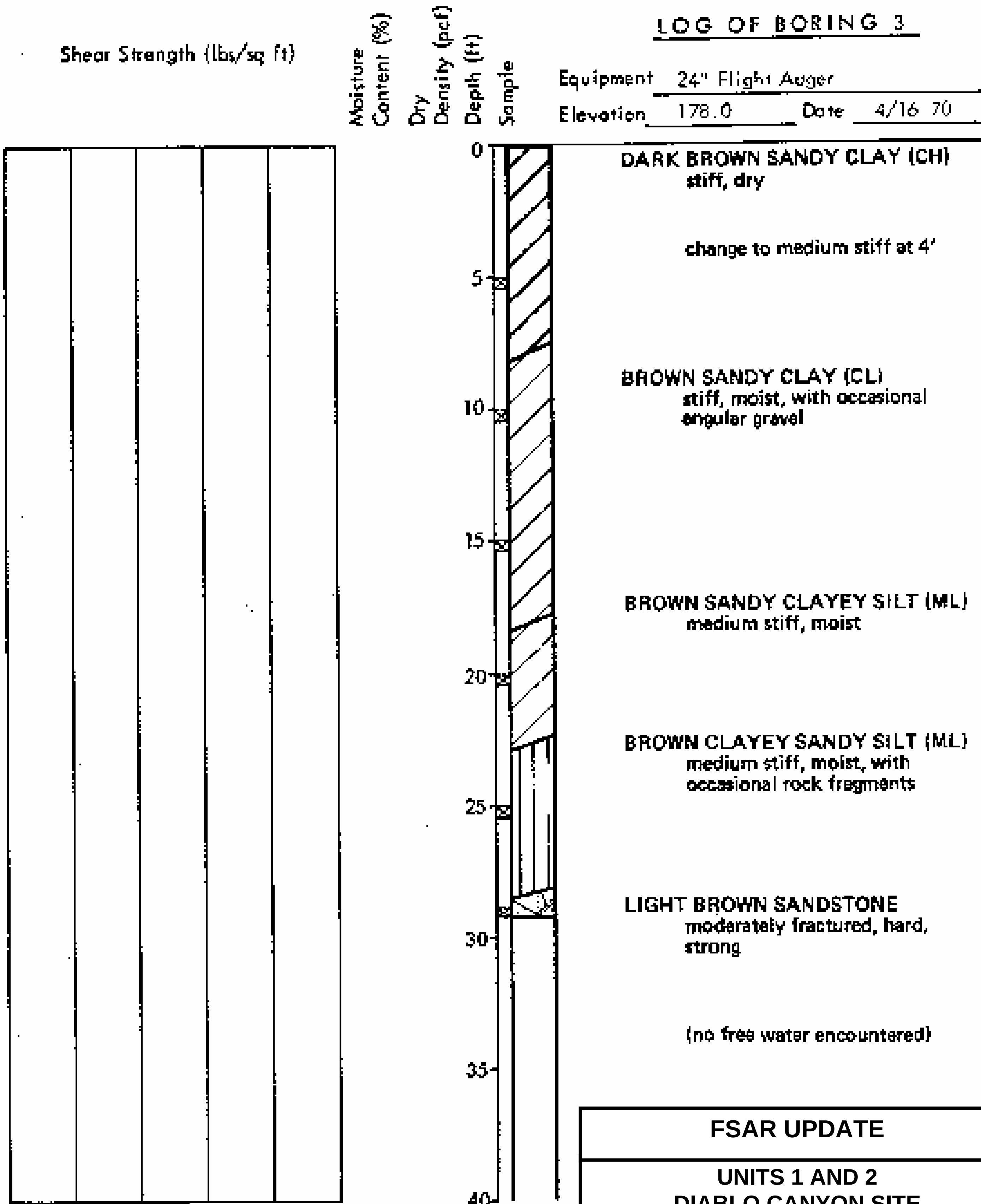
FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-24

POWER PLANT SLOPE
LOG OF BORING 2

Revision 11 November 1996

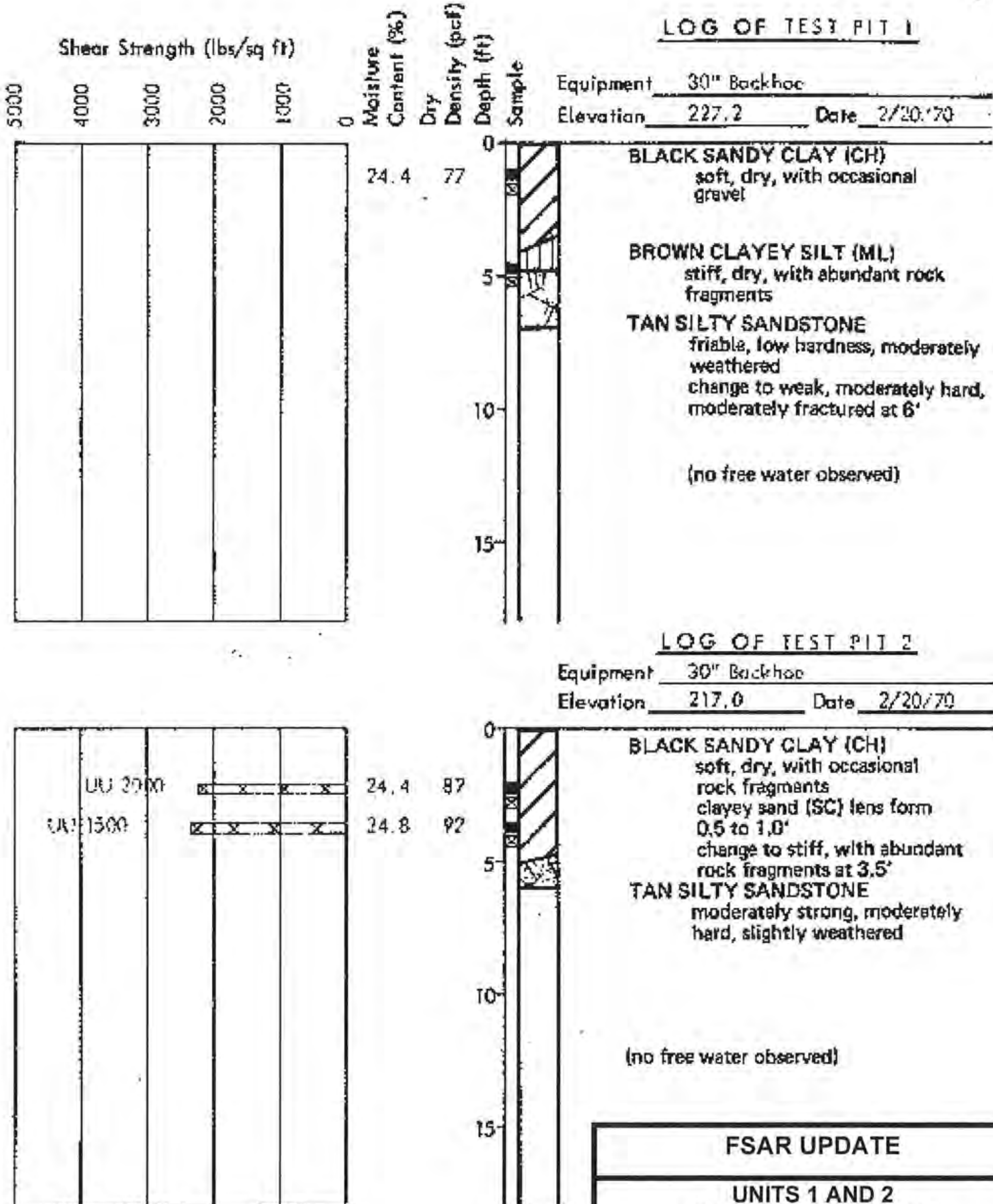


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-25

POWER PLANT SLOPE
LOG OF BORING 3

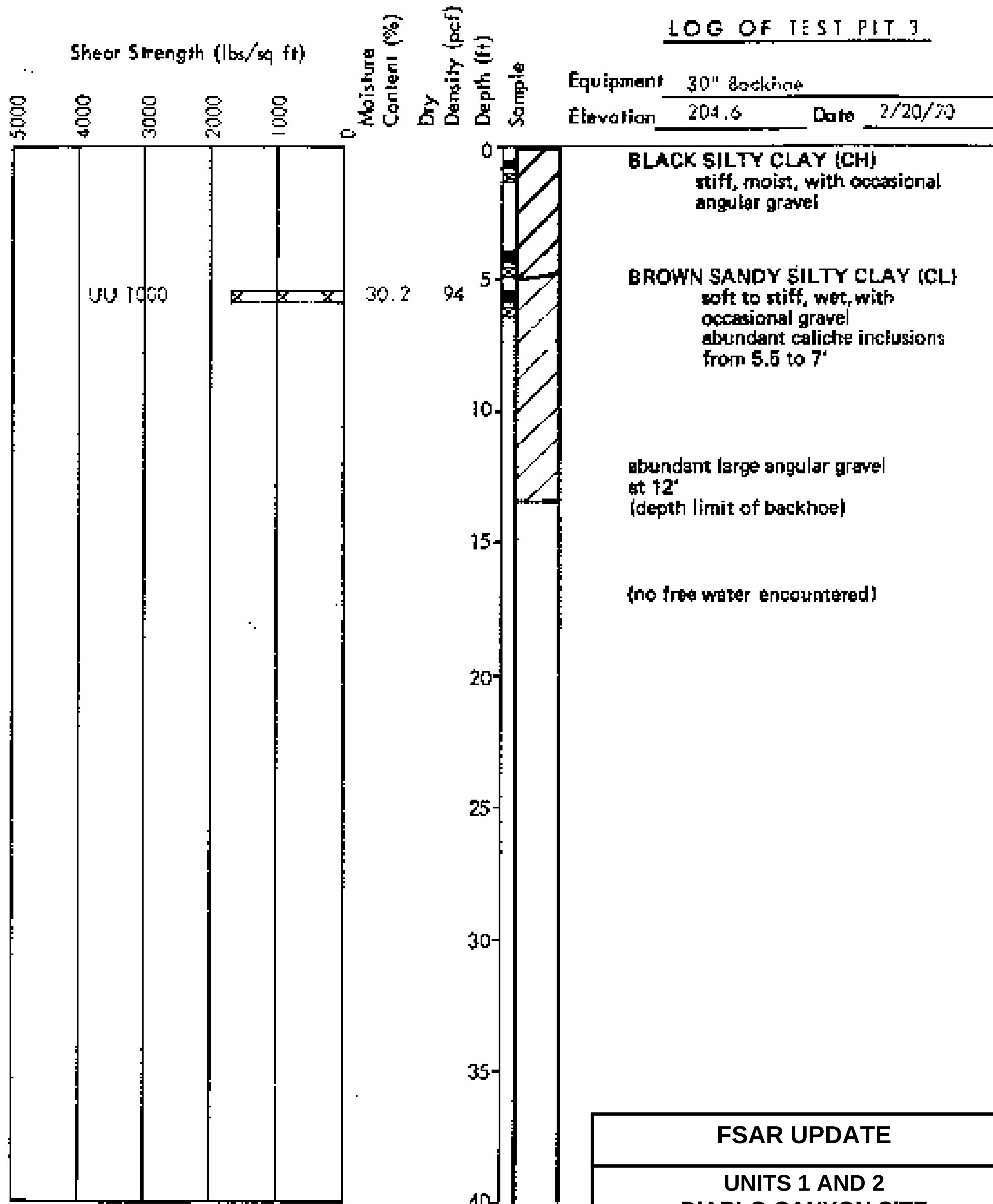


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-26

POWER PLANT SLOPE
LOG OF TEST PITS 1 & 2



FSAR UPDATE

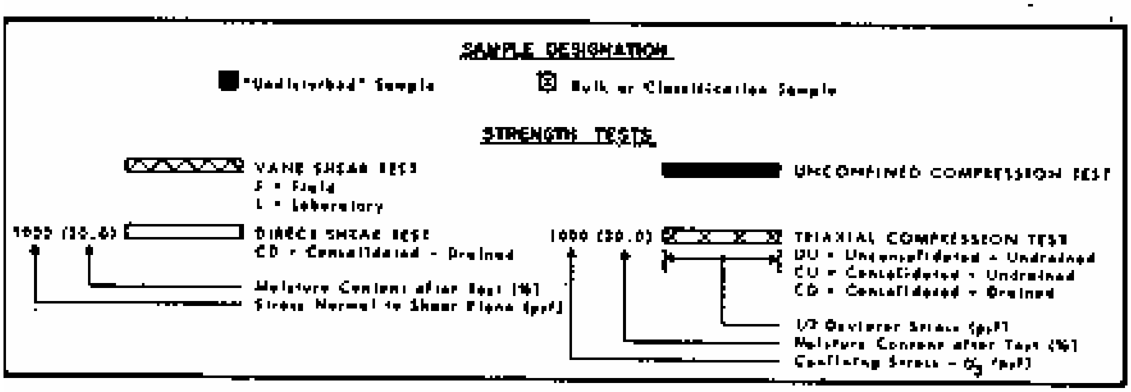
UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-27

POWER PLANT SLOPE
LOG OF TEST PIT 3

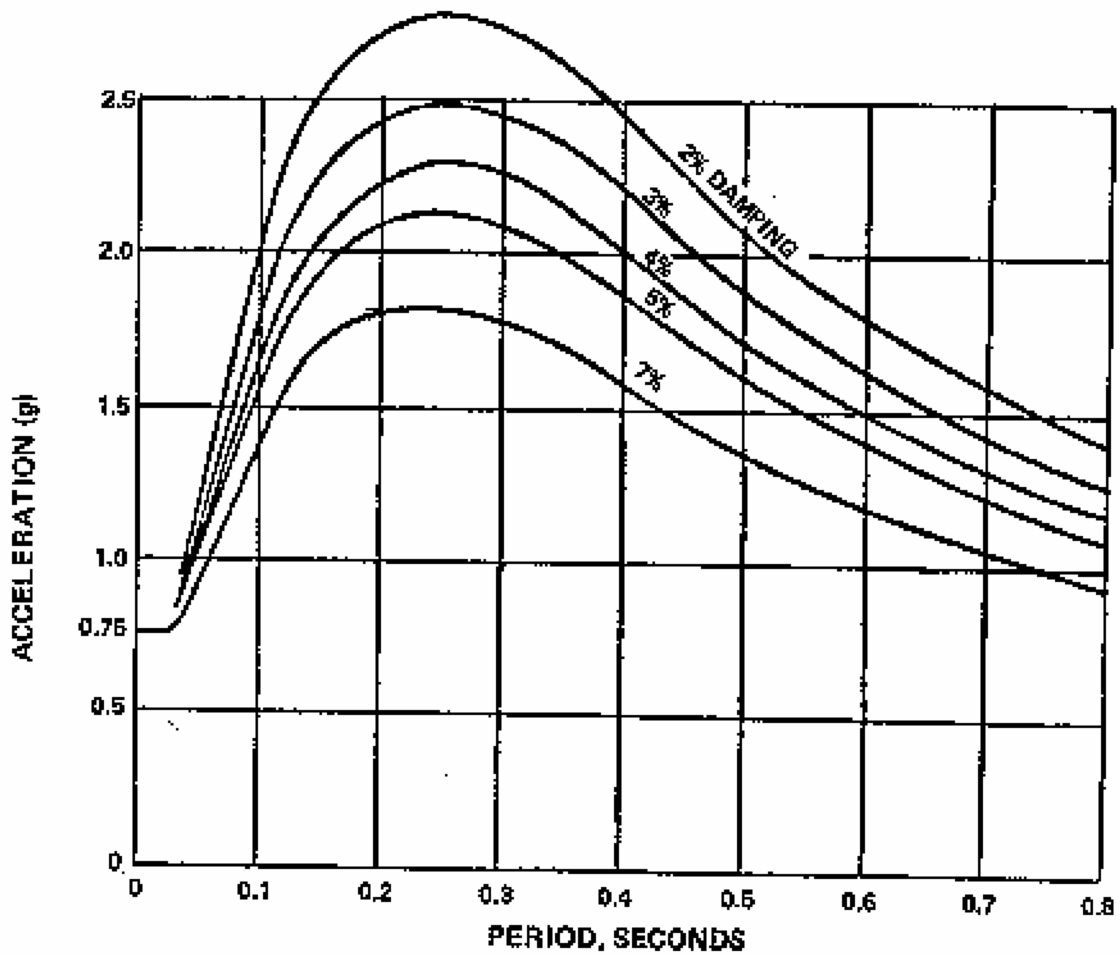
MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS (MORE THAN HALF IS LARGER THAN #200 SIEVE)	GRAVELS — MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL – SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL – SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL – SAND – SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL – SAND – CLAY MIXTURES
	SANDS — MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND – SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND – CLAY MIXTURES
FINE GRAINED SOILS (MORE THAN HALF IS SMALLER THAN #200 SIEVE)	SILTS AND CLAYS — LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS — LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM



KEY TO TEST DATA

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-28 POWER PLANT SLOPE SOIL CLASSIFICATION CHART AND KEY TO TEST AREA

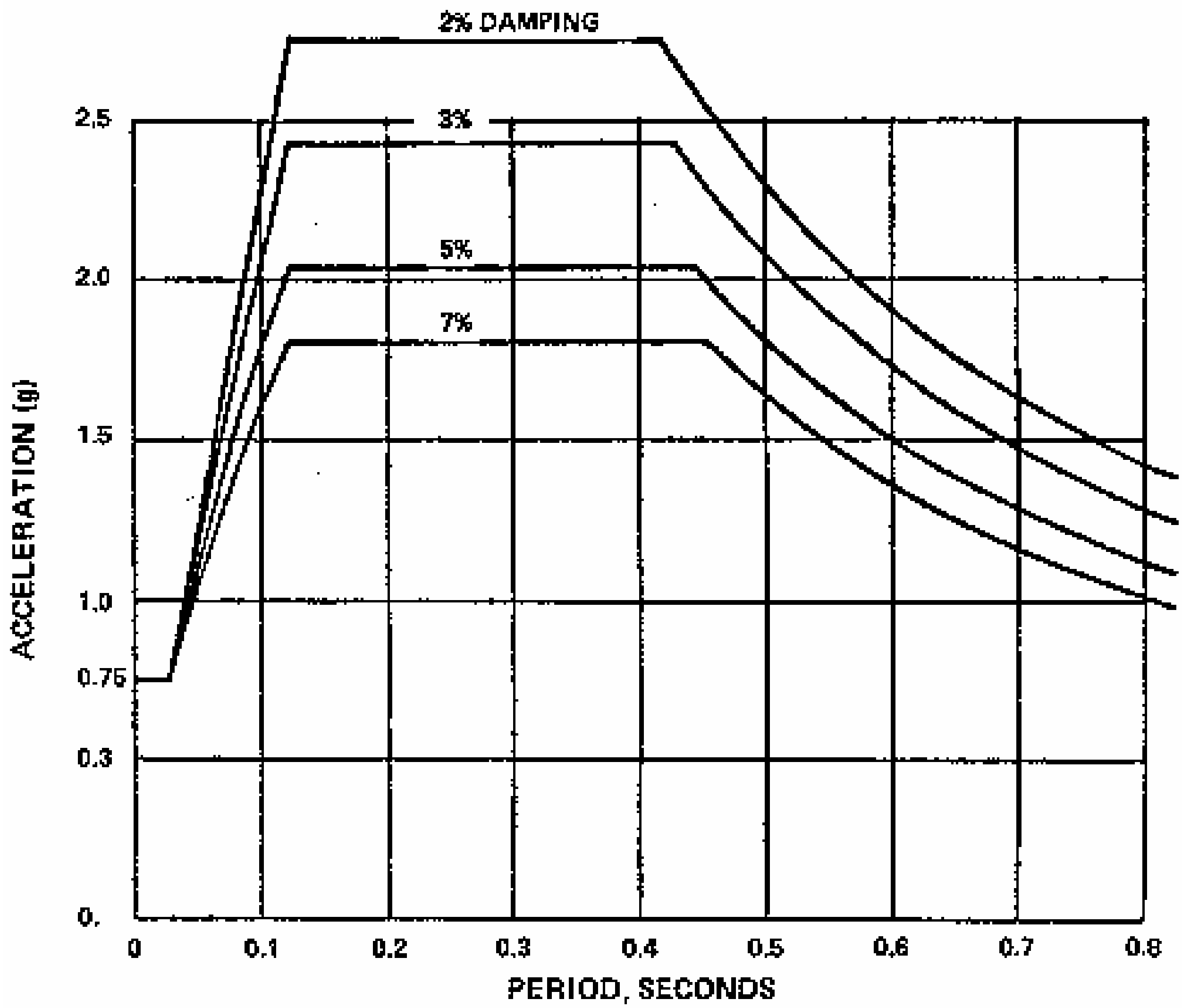


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

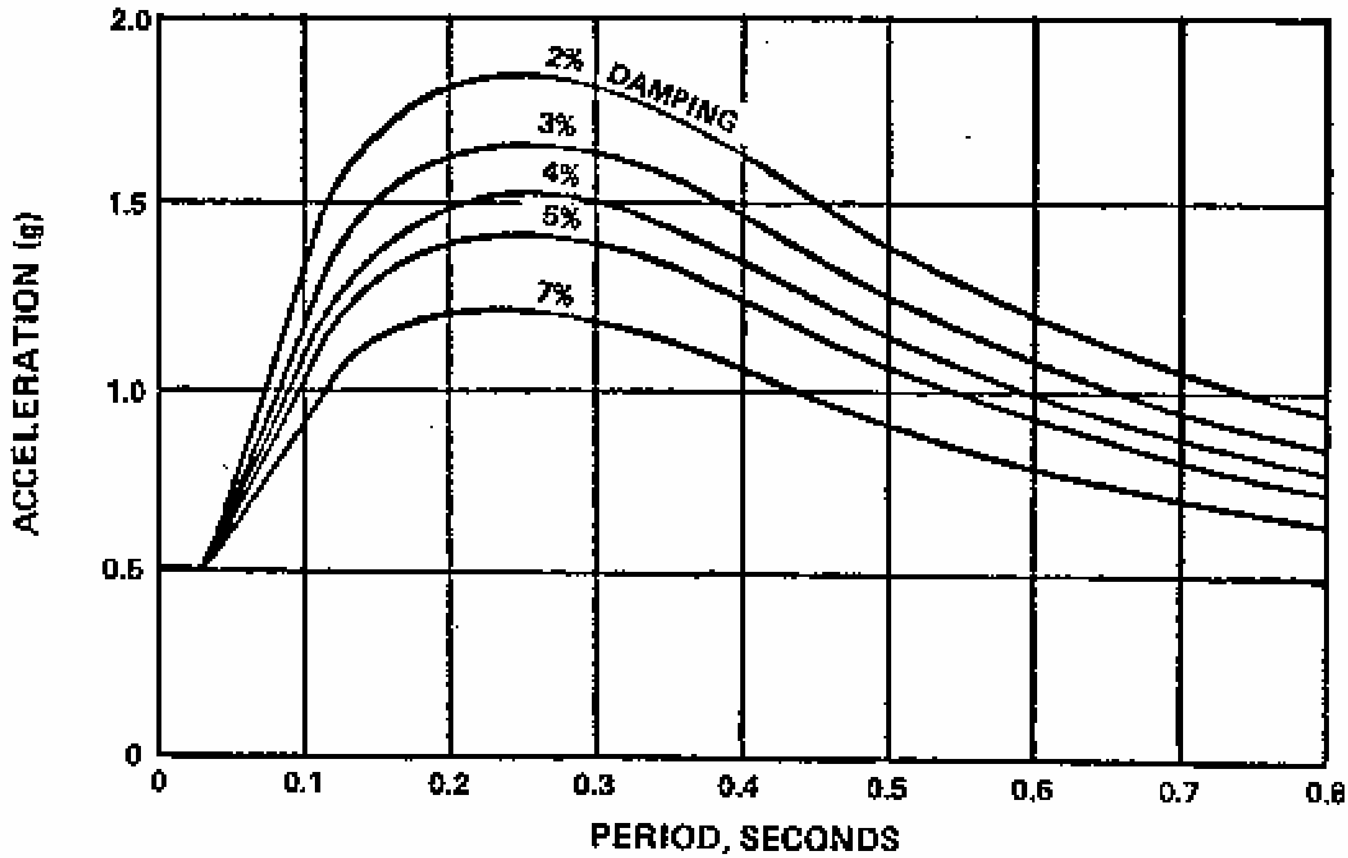
FIGURE 2.5-29
FREE FIELD SPECTRA
HORIZONTAL
HOSGRI 7.5M/BLUME

Revision 11 November 1996



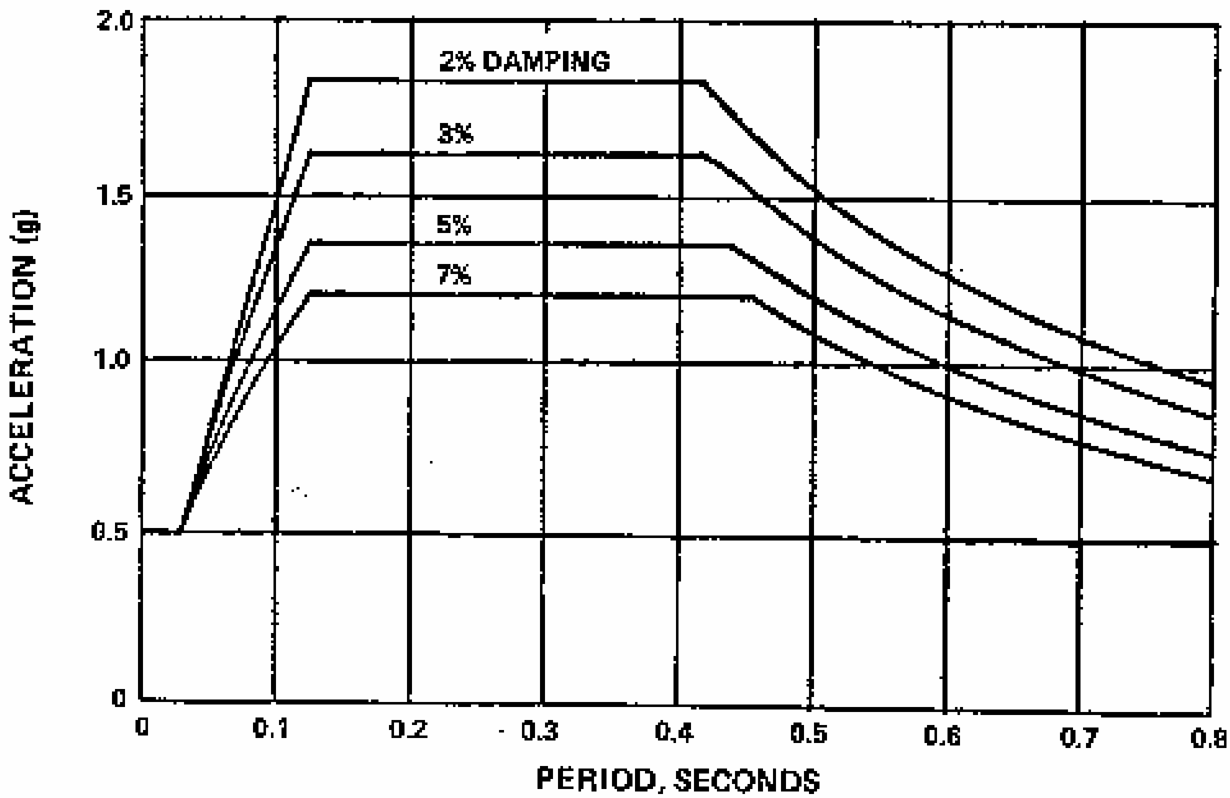
FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-30 FREE FIELD SPECTRA HORIZONTAL HOSGRI 7.5M/NEWMARK

Revision 11 November 1996



FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-31 FREE FIELD SPECTRA VERTICAL HOSGRI 7.5M/BLUME

Revision 11 November 1996

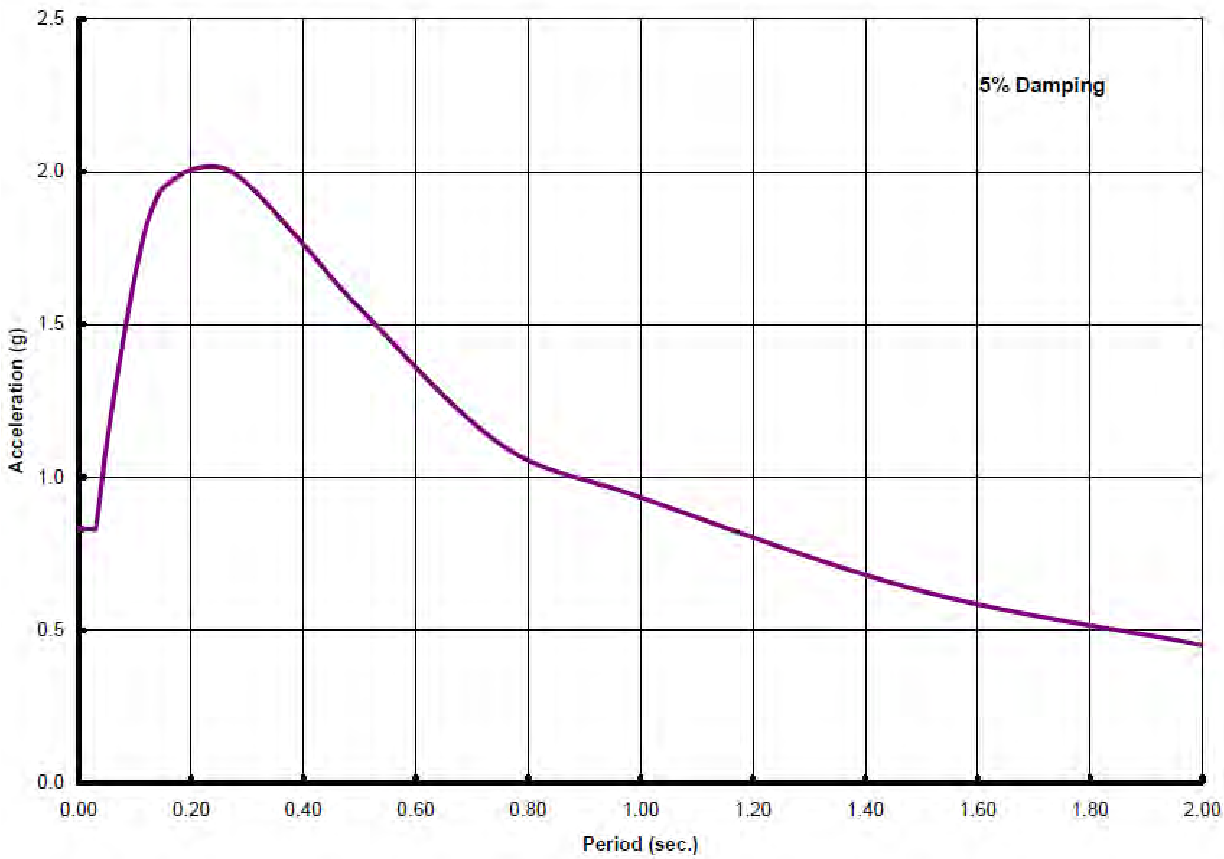


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-32
FREE FIELD SPECTRA
VERTICAL
HOSGRI 7.5M/NEWMARK

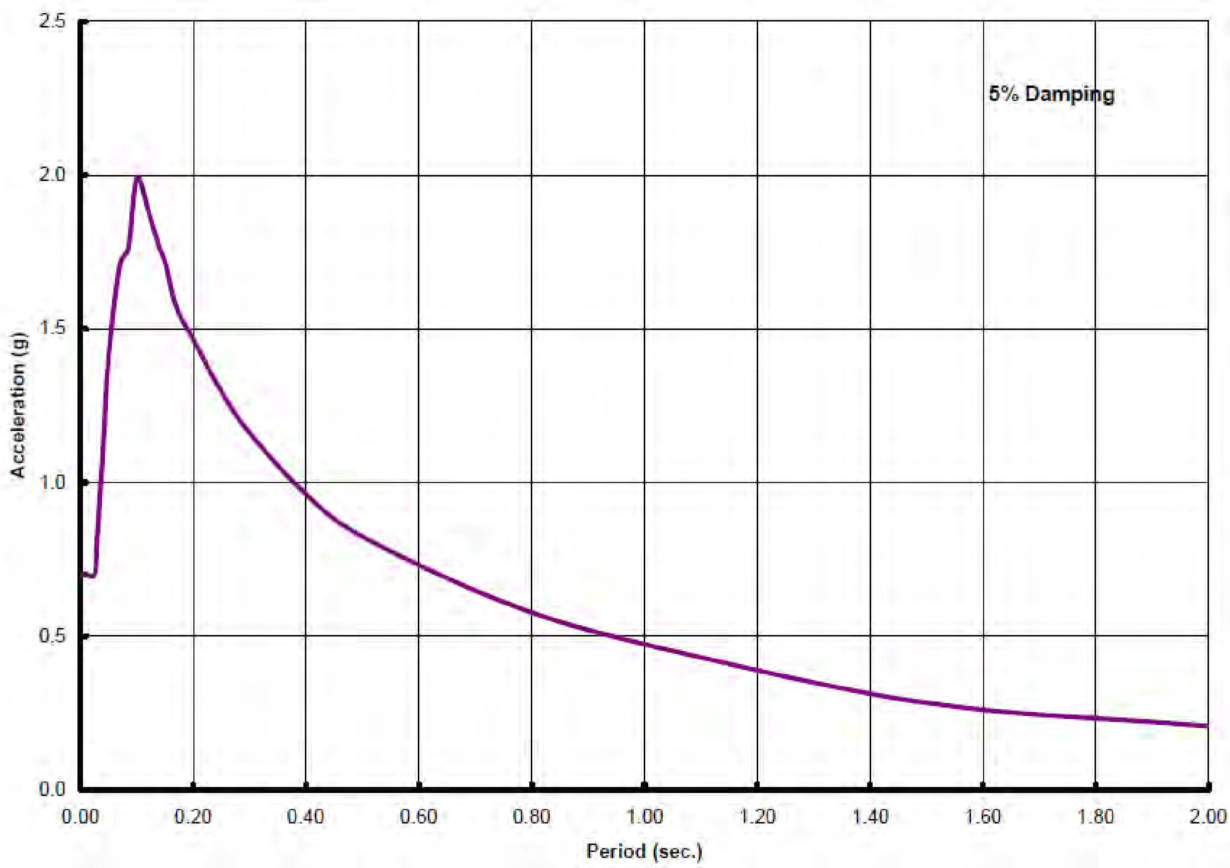
Revision 11 November 1996



NOTES:

1. This figure is based on Reference 42, Figure 2.4

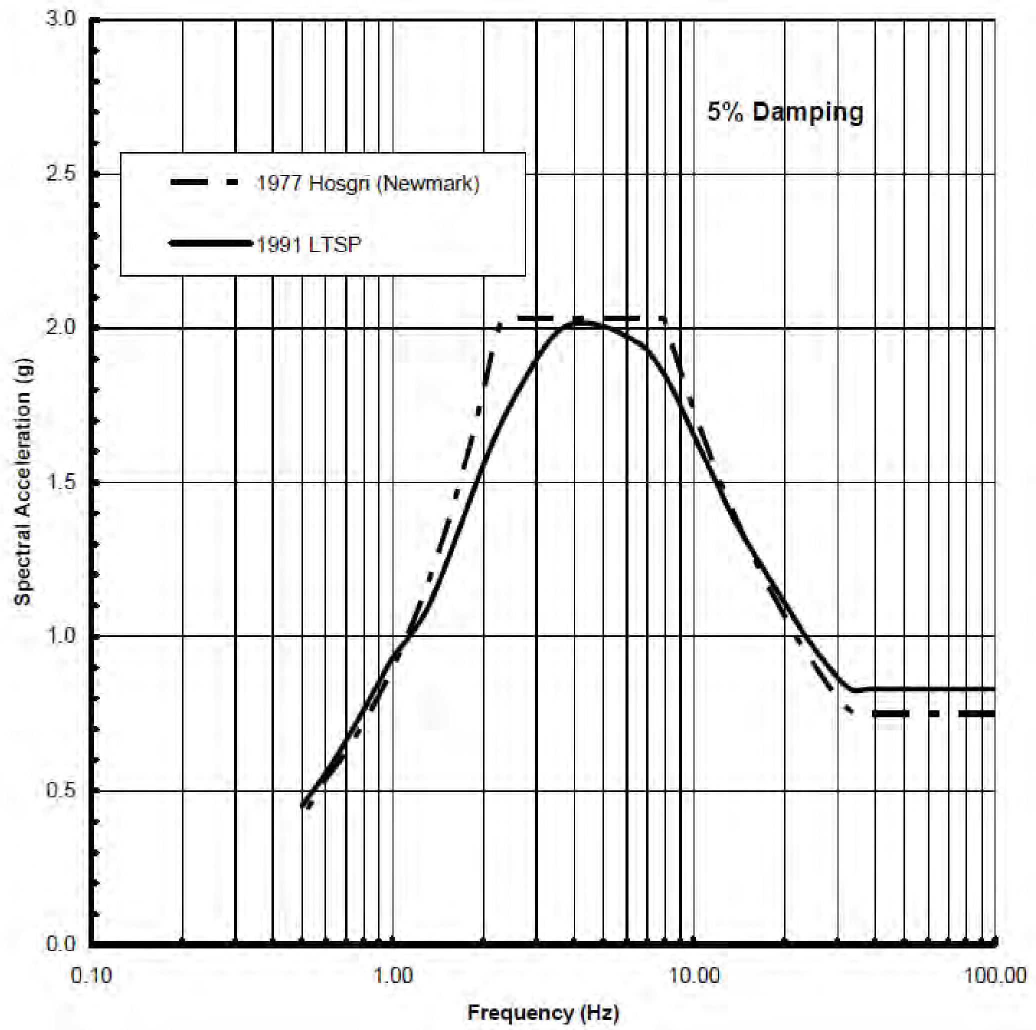
FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-33 FREE FIELD SPECTRUM HORIZONTAL 1991 LTSP (84TH PERCENTILE NON-EXCEEDANCE) AS MODIFIED PER SSER-34



NOTES:

1. This Figure is based on Reference 42, Figure 2.5.

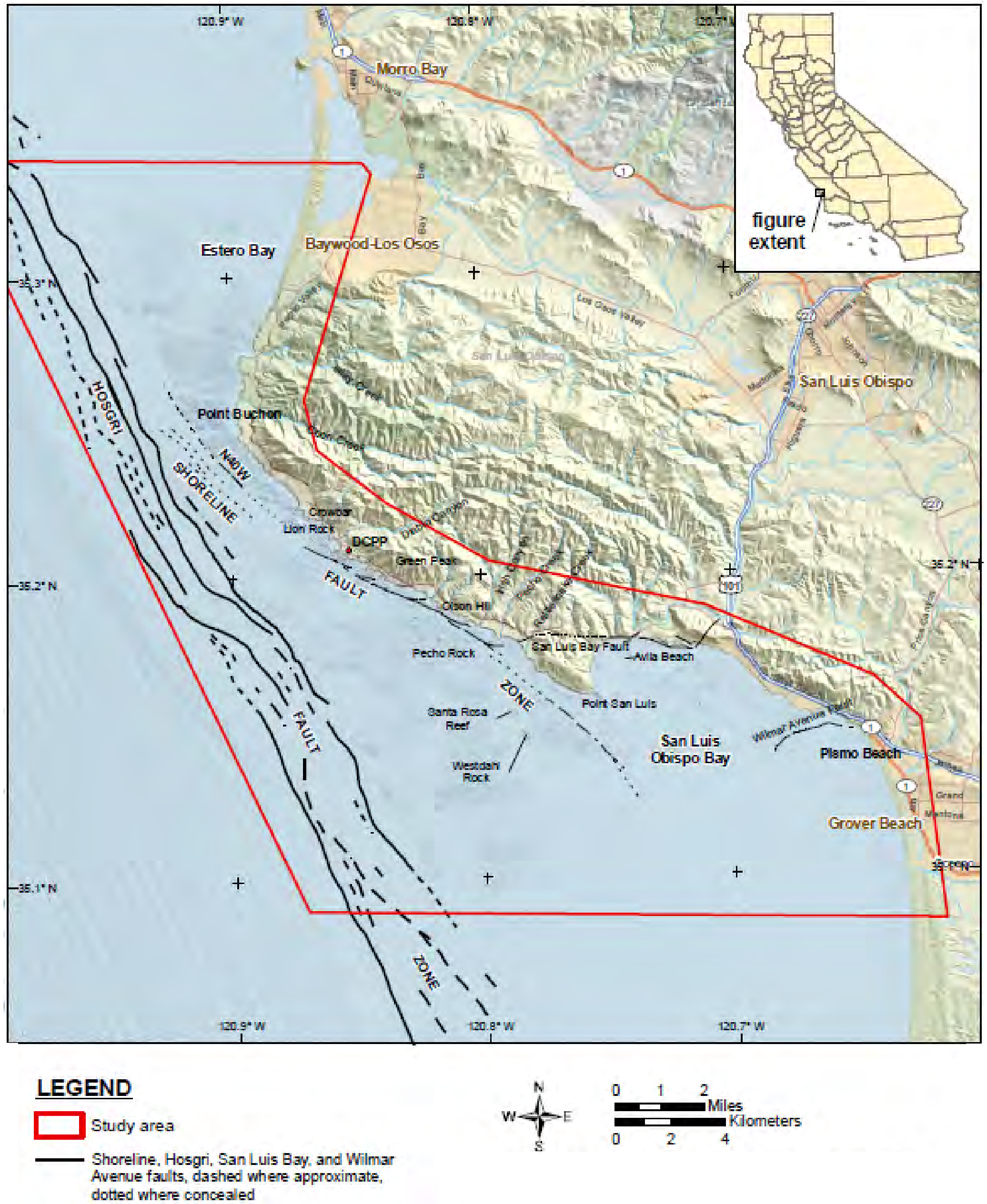
FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-34 FREE FIELD SPECTRUM VERTICAL 1991 LTSP (84TH PERCENTILE NON-EXCEEDANCE) AS MODIFIED PER SSER-34



NOTES:

1. This Figure is based on Reference 40, Figure 7-2; however, the LTSP response spectrum has been adjusted in accordance with Reference 42, Figure 2.5.
2. This Figure is for comparison purposes only. Do not use for design.
3. Legend: 1977 Hosgri (Newmark) corresponds to the spectrum shown in Figure 2.5-30
   1991 LTSP corresponds to the spectrum shown in Figure 2.5-33

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-35 FREE FIELD SPECTRA HORIZONTAL LTSP (PG&E 1998) GROUND MOTION VS. HOSGRI (NEWMARK 1977)



NOTE:

1. This figure is based on Reference 52, Figure 1-1.

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-36 MAP OF SHORELINE FAULT STUDY AREA

Chapter 3

DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS

CONTENTS

CONTENTS (Continued)

CONTENTS (Continued)

CONTENTS (Continued)

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TABLES (Continued)

TABLES (Continued)

TABLES (Continued)

Chapter 3

FIGURES

FIGURES (Continued)

FIGURES (Continued)

FIGURES (Continued)

FIGURES (Continued)

FIGURES (Continued)

FIGURES (Continued)

FIGURES (Continued)

FIGURES (Continued)

FIGURES (Continued)

FIGURES (Continued)

FIGURES (Continued)

FIGURES (Continued)

FIGURES (Continued)

NOTE:

[(a)] This figure corresponds to a controlled engineering drawing that is incorporated by reference into the FSAR Update. See Table 1.6-1 for the correlation between the FSAR Update figure number and the corresponding controlled engineering drawing number.

Chapter 3

APPENDICES

Chapter 3

DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS

3.1 CONFORMANCE WITH AEC GENERAL DESIGN CRITERIA

The Diablo Canyon Power Plant (DCPP) units are designed to comply with the Atomic Energy Commission (AEC) (now the Nuclear Regulatory Commission, or NRC) General Design Criteria (GDCs) for Nuclear Power Plant Construction Permits, published in July 1967. Sections 3.1.1 through 3.1.10, therefore, provide a listing of these criteria and a discussion of conformance. The DCPP design basis is the 1967 GDCs. Subsequent commitments to GDCs issued later (e.g., 1971 GDC, and the 1987 revision to GDC 4) are noted in the discussion of the related 1967 GDCs beginning in Section 3.1.2 and as detailed in other FSAR sections specific to systems and detailed analysis.

The GDCs are discussed in Sections. Sections 3.1.2 through 3.1.10 are grouped as presented in the proposed 10 CFR 50 Appendix A 1967 Groups. Note that where later editions of the GDC were implemented in the DCPP licensing basis, they have been incorporated in the Sections 3.1.2 through 3.1.10 related to the 1967 Group. Table 3.1-1, at the end of this section, provides a quick reference to the current license basis applicability of the GDCs. Although regulatory correspondence often refers to the 1971 criteria because NRC review is typically based on the current GDCs at the time, the DCPP licensing basis remains the 1967 GDCs except as follows:

GDC 3, 1971 - Fire Protection
GDC 4, 1987 Revision – Environmental and Missile Design Basis (LBB)
GDC 17, 1971 - Electric Power Systems
GDC 18, 1971 - Inspection and Testing of Electric Power Systems
GDC 19, 1971 - Control Room Habitability (Accident Dose)
GDC 54, 1971 - Piping Systems Penetrating Containment
GDC 55, 1971 - Reactor Coolant Pressure Boundary Penetrating Containment
GDC 56, 1971 - Primary Containment Isolation
GDC 57, 1971 - Closed System Isolation Valves

Table 3.1-2 provides a matrix listing of the 1971 criterion to related 1967 criterion.

Submittal of the FSAR using RG 1.70 Rev 1 format and content was expected by the NRC as part of the initial DCPP licensing process, even though the NRC acknowledged in NUREG-0675 (SER-00) that the DCPP design basis was the 1967 GDCs. DCPP included, as part of the original FSAR, Appendix 3.1A, “AEC General Design Criteria – 1971”, which was a summary of the extent to which the original DCPP principal design features (the 1967 GDCs plus additional design features) for plant structures, systems and components (SSCs) conformed to the intent of the AEC “General Design Criteria for Nuclear Power Plants" published in February 1971 as Appendix A to 10 CFR Part 50 (i.e., the 1971 GDCs).

Appendix 3.1A provides a summary discussion for each criterion of how the DCPP principal design features (the 1967 GDCs plus additional design features) conform to the intent of the 1971 general design criterion. Any exceptions to the 1971 GDCs that DCPP identified and the NRC approved in writing resulting from earlier DCPP design or construction commitments are identified in the discussion of the corresponding criterion in Appendix 3.1A.

The summary provided in the original FSAR in Appendix 3.1A was reviewed by the NRC to conclude that DCPP's design conformed to the intent of the 1971 GDCs. The degree to which the DCPP design (the 1967 GDCs plus additional design features) conforms to the intent of the 1971 GDCs, as summarized in Appendix 3.1A, establishes additional DCPP licensing basis which must be reviewed when evaluating facility changes.

3.1.1 SINGLE FAILURE CRITERIA

Each of the engineered safety features (ESF) is designed to tolerate a single failure during the period of recovery following an accident without loss of its protective function. This period of recovery consists of two segments: the short-term period and the long-term period. During the short-term period, the single failure is limited to a failure of an active component to complete its function as required. Should the single failure occur during the long-term period rather than the short-term, the related engineered safety system is designed to tolerate an active failure or a passive failure without loss of its protective function.

3.1.1.1 Definitions

The following definitions apply to the single failure criterion and to the GDC:

(1) Active Failure - The failure of a powered component, such as a piece of mechanical equipment, component of the electrical supply system, or instrumentation and control equipment, to act on command to perform its design function.

Examples include the failure of a motor-operated valve to move to its correct position, the failure of an electrical breaker or relay to respond, the failure of a pump, fan, or diesel generator to start, etc.

(2) Passive Failure - The structural failure of a static component that limits the component's effectiveness in carrying out its design function. When applied to a fluid system, this means a break in the pressure boundary resulting in abnormal leakage not exceeding 50 gpm for 30 minutes. Such leak rates are assumed for residual heat removal (RHR) pump seal failure.

(3) Accident - Any natural or accidental event of infrequent occurrence and its related consequences which affect the plant operation and require the use of ESF. Such events, analyzed independently and not assumed to occur simultaneously, include the loss-of-coolant accident (LOCA), steam line ruptures, steam generator tube ruptures, etc. A loss of normal (main generator) and all offsite ac power may be an isolated occurrence or may be concurrent with any event requiring ESF use if the event should cause turbine trip and grid failure.

(4) Short-term - The first 24 hours following the incident, during which time automatic actions are performed, system responses are checked, type of incident is identified, and preparations for long-term recovery operation are made.

(5) Long-term - The remainder of the recovery period following the short-term. In comparison with the short-term where the main concern is to remain within NRC specified site criteria, the long-term period of operation involves bringing the plant to cold shutdown conditions.

(6) Recovery Period - The time necessary to bring the plant to a cold shutdown. The recovery period is the sum of the short- and long-term periods defined above.

3.1.1.2 Applicability

The single failure criterion applies to the following safety-related fluid systems discussed in this FSAR Update:

System	Section
Containment Isolation Systems	6.2.4
Emergency Core Cooling System	6.3
Containment Spray System	6.2.2
Containment Fan Coolers	6.2
Auxiliary Feedwater System	6.5
Auxiliary Saltwater System	9.2.7
Component Cooling Water System	9.2.2
Chemical and Volume Control System (Boric Acid Injection Portion)	9.3.4
Diesel Fuel Oil System	9.5.4

3.1.2 OVERALL PLANT REQUIREMENTS

GDCs related to the overall plant are presented in this section. A summary discussion of conformance follows each 1967 criterion in the following subsections 3.1.2 through 3.1.10. The detailed conformance of the GDCs are further discussed in the other FSAR sections specific to systems and analyses.

3.1.2.1 Criterion 1, 1967 - Quality Standards (Category A)

Those systems and components of reactor facilities that are essential to the prevention of accidents which could affect the public health and safety, or mitigation of their consequences, shall be identified and then designed, fabricated, and erected to quality standards that reflect the importance of the safety function to be performed. Where generally recognized codes or standards on design, materials, fabrication, and inspection are used, they shall be identified. Where adherence to such codes or standards does not suffice to ensure a quality product in keeping with the safety functions, they shall be supplemented or modified as necessary. Quality assurance programs, test procedures, and inspection acceptance levels to be used shall be identified. A showing of sufficiency and applicability of codes, standards, quality assurance programs, test procedures, and inspection acceptance levels used is required.

Discussion

All systems and components of DCPP Units 1 and 2 are classified according to their importance in the prevention and mitigation of accidents. Those items vital to safe shutdown and isolation of the reactor, or whose failure might cause or increase the severity of a LOCA, or result in an uncontrolled release of excessive amounts of radioactivity, are designated Design Class I. Those items important to the reactor operation, but not essential to safe shutdown and isolation of the reactor or control of the release of substantial amounts of radioactivity, are designated Design Class II. Those items not related to reactor operation or safety are designated Design Class III.

Design Class I systems and components are essential to the protection of the health and safety of the public. Consequently, they are designed, fabricated, inspected, erected, and the materials selected to the applicable provisions of recognized codes, good nuclear practice, and to quality standards that reflect their importance. Discussions of applicable codes and standards as well as code classes are given in Section 3.2 for the major items and components. The quality assurance (QA) program conforms with the requirements of 10 CFR 50 Appendix B, Quality Assurance Criteria for Nuclear Power Plants. Details of the QA program are provided in Chapter 17.

3.1.2.2 Criterion 2, 1967 - Performance Standards (Category A)

Those systems and components of reactor facilities that are essential to the prevention of accidents which could affect the public health and safety, or to mitigation of their consequences, shall be designed, fabricated, and erected to performance standards that will enable the facility to withstand, without loss of the capability to protect the public, the additional forces that might be imposed by natural phenomena such as earthquakes, tornadoes, flooding conditions, winds, ice, and other local site effects. The design bases so established shall reflect (a) appropriate consideration of the most severe of these natural phenomena that have been recorded for the site and the surrounding area, and (b) an appropriate margin for withstanding forces greater than those recorded to reflect uncertainties about the historical data and their suitability as a basis for design.

Discussion

All systems and components designated Design Class I are designed so that there is no loss of function for ground acceleration associated with two times the design earthquake (DE) acting in the horizontal and vertical directions simultaneously. The ESF is included in the above. The working stresses for Class I items are kept within code allowable values for the DE. Similarly, measures are taken in the plant design to protect against possible effects of tsunamis, lightning storms, strong winds, and other natural phenomena.

The site characteristics are discussed in Chapter 2. Wind design criteria and flood design criteria are found in Sections 3.3 and 3.4, respectively.

3.1.2.3 Criterion 3, 1971 - Fire Protection

Structures, systems, and components important to safety shall be designed and located to minimize, consistent with other safety requirements, the probability and effect of fires and explosions. Noncombustible and heat resistant materials shall be used wherever practical throughout the unit, particularly in locations such as the containment and control room. Fire detection and fighting systems of appropriate capacity and capability shall be provided and designed to minimize the adverse effects of fires on structures, systems, and components important to safety. Firefighting systems shall be designed to assure that their rupture or inadvertent operation does not significantly impair the safety capability of these structures, systems, and components.

Discussion

GDC 3 (1971) is invoked by 10 CFR 50.48, Fire Protection. The fire protection program for DCPP satisfies the requirements of GDC 3 (1971) by complying with the guidelines of Appendix A to NRC Branch Technical Position (BTP) (APCSB) 9.5-1, and with the provisions of 10 CFR 50 Appendix R, Sections III.G, J, L, and O, as stipulated by Operating License Conditions 2.C(5) and 2.C(4) for Units 1 and 2, respectively. Approved deviations from Appendix A to BTP (APCSB) 9.5-1, and Appendix R sections are identified in Supplement Numbers 8, 9, 13, 23, 27, and 31 to the Safety Evaluation Report (NUREG-0675).

The probability of fires and explosions is minimized by extensive use of noncombustible and fire resistant materials, by physical isolation and protection of flammable fluids, by providing both automatic and manual fire extinguishing systems, and by use of fire detection systems.

Electrical insulation is made of fire retardant, self-extinguishing materials. All exposed electrical raceways are metal and have fire stops liberally applied. Electrical conductors have adequate ratings and overcurrent protection to prevent breakdown or excessive heating.

Electrical equipment for safety systems is physically arranged to minimize the effect of a potential fire. Vital interconnecting circuits are located to avoid potential fire hazards as much as possible, with mutually redundant circuits placed in separate raceways. The facility is equipped with a fire protection system (FPS) for controlling any fire that might originate in plant equipment. This system is described in Section 9.5.1.

The containment and auxiliary building ventilation systems are operated from the control room. Critical areas of the plant have detectors and alarms to alert the control room operator of the possibility of fire, so that prompt action can be taken to prevent significant damage.

3.1.2.4 Criterion 4, 1967 - Sharing of Systems (Category A)

Reactor facilities shall not share systems or components unless it is shown safety is not impaired by the sharing.

Discussion

Those systems or components that are shared, either between the two units or functionally within a single unit, are designed in such a manner that plant safety is not impaired by the sharing. A list of shared systems and components is provided in Section 1.2.2.10.

3.1.2.5 Criterion 5, 1967 - Records Requirements (Category A)

Records of the design, fabrication, and construction of essential components of the plant shall be maintained by the reactor operator or under its control throughout the life of the reactor.

Discussion

Records of the design, fabrication, construction, and testing of Design Class I components of the plant will be maintained by Pacific Gas and Electric Company (PG&E) or kept under its control throughout the life of the plant. Chapter 17 describes the procedures for keeping these records. Operating records to be maintained throughout the life of the plant are described in Chapter 13.

3.1.3 PROTECTION BY MULTIPLE FISSION PRODUCT BARRIERS

GDCs related to prevention of fission product release are presented in this section. A discussion of conformance follows each criterion.

3.1.3.1 Criterion 6, 1967 - Reactor Core Design (Category A)

The reactor core shall be designed to function throughout its design lifetime, without exceeding acceptable fuel damage limits which have been stipulated and justified. The core design, together with reliable process and decay heat removal systems, shall provide for this capability under all expected conditions of normal operation with appropriate margins for uncertainties and for transient situations which can be anticipated, including the effects of the loss of power to recirculation pumps, tripping out of a turbine generator set, isolation of the reactor from its primary heat sink, and loss of all offsite power.

Discussion

Each reactor core with its related control and protection systems is designed to function throughout its design lifetime without exceeding acceptable fuel damage limits. Core design, together with reliable process and decay heat removal systems, provides for this capability under all expected conditions of normal operation with appropriate margins for uncertainties and anticipated transient situations, including the effects of the loss of reactor coolant flow, trip of the turbine-generator, loss of normal feedwater, and loss of all offsite power.

The reactor control and protection instrumentation systems are designed to actuate a reactor trip for any anticipated combination of plant conditions when necessary to ensure a minimum departure from nucleate boiling ratio (DNBR) equal to or greater than the applicable limit value (refer to Sections 4.4.1.1 and 4.4.2.3) and fuel center temperatures below the melting point of UO_2.

Chapter 4 discusses the design bases and design evaluation of reactor components. The details of the control and protection instrumentation systems design and logic are discussed in Chapter 7. This information supports the accident analyses presented in Chapter 15.

3.1.3.2 Criterion 7, 1967 - Suppression of Power Oscillations (Category B)

The core design, together with reliable controls, shall ensure that power oscillations which could cause damage in excess of acceptable fuel damage limits are not possible or can be readily suppressed.

Discussion

Power oscillations of the fundamental mode are inherently eliminated by the negative Doppler and non-positive moderator temperature coefficients of reactivity.

Oscillations due to xenon spatial effects, in the radial, diametral, and azimuthal overtone modes, are heavily damped due to the inherent design and due to the negative Doppler and non-positive moderator temperature coefficients of reactivity.

Oscillations due to xenon spatial effects, in the axial first overtone mode, may occur. Assurance that fuel design limits are not exceeded by xenon axial oscillations is provided as a result of reactor trip functions using the measured axial power imbalance as an input.

Oscillations due to xenon spatial effects, in axial modes higher than the first overtone, are heavily damped due to the inherent design and due to the negative Doppler coefficient of reactivity.

The stability of the cores against xenon-induced power oscillations and the functional requirements of instrumentation for monitoring and measuring core power distributions are discussed in Section 4.3. Details of the instrumentation design and logic are discussed in Chapter 7.

3.1.3.3 Criterion 8, 1967 - Overall Power Coefficient (Category B)

The reactor shall be designed so that the overall power coefficient in the power operating range shall not be positive.

Discussion

Prompt compensatory reactivity feedback effects are ensured when each reactor is critical by the negative fuel temperature effect (Doppler effect) and by the operational limit on moderator temperature coefficient of reactivity. The negative Doppler coefficient of reactivity is ensured by the inherent design using low-enrichment fuel. The limits on moderator temperature coefficient of reactivity are ensured by administratively controlling the dissolved neutron absorber concentration and control rod position.

These reactivity coefficients are discussed in Section 4.3.

3.1.3.4 Criterion 9, 1967 - Reactor Coolant Pressure Boundary (Category A)

The reactor coolant pressure boundary shall be designed and constructed so as to have an exceedingly low probability of gross rupture or significant leakage throughout its design lifetime.

Discussion

The reactor coolant system (RCS) boundaries are designed to accommodate the system pressures and temperatures attained under all expected modes of plant operation, including all anticipated transients, and to maintain the stresses within applicable stress limits. The reactor coolant pressure boundary materials selection and fabrication techniques ensure a low probability of gross rupture or significant leakage. Additional details are presented in Section 5.2.

In addition to the loads imposed on the system under normal operating conditions, abnormal loading conditions, such as seismic loading and pipe rupture, are also considered, as discussed in Sections 3.6 and 3.7. The systems are protected from overpressure by means of pressure-relieving devices as required by applicable codes.

Means are provided to detect significant uncontrolled leakage from either reactor coolant pressure boundary with indication in the control room as discussed in Section 5.2. Each RCS boundary has provisions for inspection, testing, and surveillance of critical areas to assess the structural and leaktight integrity. The details of these provisions are given in Section 5.2. For each reactor vessel, a material surveillance program conforming to applicable codes is provided. Additional details are provided in Section 5.4.

The materials of construction of the pressure-retaining boundary of the RCS are protected by control of coolant chemistry from corrosion that might otherwise reduce the system structural integrity during its service lifetime. In addition, secondary side water chemistry is controlled to minimize steam generator tube degradation and protect the reactor coolant pressure boundary as discussed in Sections 10.4.6, 10.4.8 and 10.4.9.

3.1.3.5 Criterion 10, 1967 - Containment (Category A)

Containment shall be provided. The containment structure shall be designed to sustain the initial effects of gross equipment failures, such as a large coolant boundary break, without loss of required integrity and, together with other engineered safety features as may be necessary, to retain for as long as the situation requires the functional capability to protect the public.

Discussion

The reactor containment is a reinforced concrete structure with a steel liner that is capable of withstanding the pressure buildup resulting from a major LOCA.

The reactor containment, together with the containment spray system and the containment fan cooler system, are described in Chapter 6. The structural design criteria are presented in Chapter 3. The consequences of major LOCAs are analyzed in Chapter 15.

3.1.4 NUCLEAR AND RADIATION CONTROLS

GDCs related to nuclear and radiation controls are presented in this section. A discussion of conformance follows each criterion.

3.1.4.1 Criterion 11, 1967 - Control Room (Category B)

The facility shall be provided with a control room from which actions to maintain safe operational status of the plant can be controlled. Adequate radiation protection shall be provided to permit access, even under accident conditions, to equipment in the control room or other areas as necessary to shut down and maintain safe control of the facility without radiation exposures of personnel in excess of 10 CFR 20 limits. It shall be possible to shut the reactor down and maintain it in a safe condition if access to the control room is lost due to fire or other cause.

Discussion

The plant is provided with a centralized control room common to both units that contains the controls and instrumentation necessary for operation of both units under normal and accident conditions.

The control room is continuously occupied by the operating personnel under all operating and accident conditions. Sufficient shielding, distance, and containment are provided to ensure that the control room personnel are not subject to radiation exposures in excess of 10 CFR 20 limits. Adequate radiation protection is provided to permit access and occupancy of the control room under accident conditions without personnel receiving radiation exposures in excess of 5 rem whole body, or its equivalent to any part of the body, for the duration of the accident to meet the requirements of GDC 19, 1971. Control room shielding is described in Section 12.1, and postaccident control room exposures are described in Section 15.5.

The control room ventilation system is described in Section 9.4.1. It consists of a dual system providing a large percentage of recirculated air. In the event of fire in the control room, provisions are made for 100 percent outside air makeup operation. In the event of airborne toxic gas outside the control room, provisions are made for operation with 100 percent recirculated air. In the event of airborne radioactivity outside the control room, provisions are made to isolate and pressurize the control room.

The risk of fire is minimized by the use of noncombustible and fire retardant materials in the construction of the control room and its furnishings. Fire fighting equipment is located in the control room, and the use and storage of combustible supplies are minimized.

Provisions are made to enable plant operators to readily shut down and maintain the plant at safe shutdown (Mode 3) by means of controls located outside the control room.

3.1.4.1.1 Criterion 19, 1971 – Control Room

A control room shall be provided from which actions can be taken to operate the nuclear power unit safely under normal conditions and to maintain it in a safe condition under accident conditions, including loss-of-coolant accidents. Adequate radiation protection shall be provided to permit access and occupancy of the control room under accident conditions without personnel receiving radiation exposures in excess of 5 rem whole body, or its equivalent to any part of the body, for the duration of the accident.

Discussion

Adequate radiation protection is provided to permit access and occupancy of the control room under accident conditions without personnel receiving radiation exposures in excess of 5 rem whole body, or its equivalent to any part of the body, for the duration of the accident to meet the requirements of GDC 19, 1971. Refer to Section 6.4.

3.1.4.2 Criterion 12, 1967 - Instrumentation and Control Systems (Category B)

Instrumentation and controls shall be provided as required to monitor and maintain variables within prescribed operating ranges.

Discussion

Reactor, control rod, boron concentration, pressurizer pressure and level, feedwater, steam dump, and turbine instrumentation and controls are provided to monitor and maintain variables within prescribed operating ranges. Reactor protection systems that receive plant instrumentation signals and automatically actuate alarms, inhibit control rod withdrawal, initiate load cutback, and/or trip the reactors as prescribed limits are approached or reached are also provided. These systems are discussed in Chapter 7.

3.1.4.3 Criterion 13, 1967 - Fission Process Monitors and Controls (Category B)

Means shall be provided for monitoring and maintaining control over the fission process throughout core life and for all conditions that can reasonably be anticipated to cause variations in reactivity of the core, such as indication of position of control rods and concentration of soluble reactivity control poisons.

Discussion

Control over the fission process for each reactor will be maintained throughout the core life by the combination of control rods and chemical shim (boration). Adequate indication of the core reactivity status is provided by the nuclear instrumentation system (NIS). Periodic samples of boron concentration and continuous indication of RCS temperature and control rod position provide additional fission process information.

During operation, the shutdown rod banks are fully withdrawn. The control rod system automatically maintains a programmed average reactor temperature compensating for reactivity effects associated with scheduled and transient load changes. The shutdown rod banks along with the control banks are designed to shut down the reactor under conditions of normal operation and anticipated operational occurrences.

The boron system maintains the reactor in the cold shutdown state independent of the position of the control rods and can compensate for all xenon burnout transients.

The reactivity control and NIS are discussed in Chapters 4 and 7.

3.1.4.4 Criterion 14, 1967 - Core Protection Systems (Category B)

Core protection systems, together with associated equipment, shall be designed to act automatically to prevent or to suppress conditions that could result in exceeding acceptable fuel damage limits.

Discussion

Operational limits for the core protection systems are defined by analyses of all plant operating and fault conditions requiring rapid rod insertion to prevent or limit core damage. The protection system design bases for all anticipated transients or faults are:

(1) Minimum DNBR shall be the applicable limit value (refer to Sections 4.4.1.1 and 4.4.2.3)

(2) Cladding strain on the fuel element shall not exceed 1 percent

(3) Center melt shall not occur in the fuel elements

A region of permissible core operation is defined in terms of power, axial power distribution, and coolant flow and temperature. The protection systems monitor these process variables (as well as many other process and plant variables). If the region limits are approached during operation, the protection systems automatically actuate alarms, initiate load runback, prevent control rod withdrawal, and/or trip the reactor. Operation within the permissible region and complete core protection is ensured by the overtemperature ΔT and overpower ΔT reactor trips over the system pressure range defined by the pressurizer high-pressure and pressurizer low-pressure reactor trips, provided that the transient is slow with respect to piping delays from the core to the temperature sensors. High neutron flux and low coolant flow reactor trips provide core protection against transients that are faster than the ΔT response. Also, thermal transients are anticipated and avoided by reactor trips actuated by turbine trip and primary coolant pump circuit breaker position. The protection systems are discussed in Section 7.2.

3.1.4.5 Criterion 15, 1967 - Engineered Safety Features Protection Systems (Category B)

Protection systems shall be provided for sensing accident situations and initiating the operation of necessary engineered safety features.

Discussion

An important safety function of the reactor protection system is that of processing signals used for ESF actuation and generation of the actuation demand. ESFs are discussed in Chapter 6 and Section 7.3.

3.1.4.6 Criterion 16, 1967 - Monitoring Reactor Coolant Pressure Boundary (Category B)

Means shall be provided for monitoring the reactor coolant pressure boundary to detect leakage.

Discussion

All RCS components are designed, fabricated, inspected, and tested in conformance with the ASME Boiler and Pressure Vessel Code.

Leakage is detected by an increase in the amount of makeup water required to maintain a normal level in the pressurizer. The reactor vessel closure joint is provided with a temperature monitored leakoff between double gaskets.

Leakage into the reactor containment is drained to the reactor building sump where the level is monitored.

Leakage is also detected by measuring the airborne activity and quantity of the condensate drained from each reactor containment fan cooler unit.

These leakage detection methods are described in detail in Section 5.2.

3.1.4.7 Criterion 17, 1967 - Monitoring Radioactivity Releases (Category B)

Means shall be provided for monitoring the containment atmosphere, the facility effluent discharge paths, and the facility environs for radioactivity that could be released from normal operations, from anticipated transients and from accident conditions.

Discussion

The containment atmosphere, the plant vents, and the liquid and gaseous waste systems effluent discharge paths are monitored for radioactivity concentrations during all modes of operations. The monitoring systems are described in Section 11.4. The offsite radiological monitoring program is described in Section 11.6.

Waste handling systems are incorporated in each facility design for processing and/or retention of normal operation radioactive wastes with appropriate controls and monitors to ensure that releases do not exceed the limits of 10 CFR 20. The facilities are also designed with provisions to monitor radioactivity release during accidents and to prevent releases from causing exposures in excess of the guideline levels specified in 10 CFR 100.

3.1.4.8 Criterion 18, 1967 - Monitoring Fuel and Waste Storage (Category B)

Monitoring and alarm instrumentation shall be provided for fuel and waste storage and handling areas for conditions that might contribute to loss of continuity in decay heat removal and to radiation exposures.

Discussion

The fuel and waste storage and handling areas are provided with monitoring and alarm systems for radioactivity, and the plant vents are monitored for radioactivity during all operations. The monitoring systems are described in Section 11.4.

The spent fuel pool cooling system is equipped with adequate instrumentation for normal operation. Water temperatures in the pool and at the outlet of the heat exchanger are indicated locally, and high pool temperature is alarmed in the control room. The spent fuel pool cooling system is described in Section 9.1.

3.1.5 RELIABILITY AND TESTABILITY OF PROTECTION SYSTEMS

GDCs related to reliability and testing of protection systems are presented in this section. A discussion of conformance follows each criterion.

3.1.5.1 Criterion 19, 1967 - Protection Systems Reliability (Category B)

Protection systems shall be designed for high functional reliability and in-service testability commensurate with the safety functions to be performed.

Discussion

The protection systems are designed for high functional reliability and inservice testability. Each design employs redundant logic trains and measurement and equipment diversity. Sufficient redundancy is provided to enable individual end-to-end channel tests with each reactor at power without compromise of the protective function. Built-in semiautomatic testers provide means to test the majority of system components very rapidly. The protection systems are described in Section 7.2.

3.1.5.2 Criterion 20, 1967 - Protection Systems Redundancy and Independence (Category B)

Redundancy and independence designed into protection systems shall be sufficient to assure that no single failure or removal from service of any component or channel of a system will result in loss of the protection function. The redundancy provided shall include, as a minimum, two channels of protection for each protection function to be served.

Different principles shall be used where necessary to achieve true independence of redundant instrumentation components.

Discussion

Sufficient redundancy and independence is designed into the protection systems to ensure that no single failure nor removal from service of any component or channel of a system will result in loss of the protection function. The minimum redundancy is exceeded in each protection function that is active with the reactor at power.

Functional diversity and consequential location diversity are designed into the systems. DCPP uses the Westinghouse Eagle 21 Process Protection System, which is discussed in detail in Section 7.2.

3.1.5.3 Criterion 21, 1967 - Single Failure Definition (Category B)

Multiple failures resulting from a single event shall be treated as a single failure.

Discussion

When evaluating the protection systems, the ESF, and their support systems, multiple failures resulting from a single event are treated as a single failure. The ability of each system to perform its function with a single failure is discussed in the sections describing the individual systems. The single failure criterion is discussed further at the beginning of Section 3.1.1.

3.1.5.4 Criterion 22, 1967 - Separation of Protection and Control Instrumentation Systems (Category B)

Protection systems shall be separated from control instrumentation systems to the extent that failure or removal from service of any control instrumentation system component or channel, or of those common to control instrumentation and protection circuitry, leaves intact a system satisfying all requirements for the protection channels.

Discussion

The protection systems comply with the requirements of IEEE-279, 1971, Criteria for Protection Systems for Nuclear Power Generating Stations, although construction permits for the DCPP units were issued prior to issuance of the 1971 version of the standard. Each protection system is separate and distinct from the respective control systems. The control system is dependent on the protection system in that control signals are derived from protection system measurements, where applicable. These signals are transferred to the control system by isolation amplifiers that are classified as protection system components. The adequacy of system isolation has been verified by testing or analysis under conditions of all postulated credible faults. Isolation devices that serve to protect Instrument Class IA instrument loops have all been tested. For certain applications where the isolator is protecting an Instrument Class IB instrument loop, and the isolation device is a simple linear device with no complex failure modes, the analysis was used to verify the adequacy of the isolation device. The failure or removal of any single control instrumentation system component or channel, or of those common to the control instrumentation system component or channel and protection circuitry, leaves intact a system that satisfies the requirements of the protection system. The protection systems and control systems are discussed in Chapter 7.

3.1.5.5 Criterion 23, 1967 - Protection Against Multiple Disability of Protection Systems (Category B)

The effects of adverse conditions to which redundant channels or protection systems might be exposed in common, either under normal conditions or those of an accident, shall not result in loss of the protection function.

Discussion

Physical separation and electrical isolation of redundant channels and subsystems, functional diversity of subsystems, and safe failure modes are employed in design of the reactors as defenses against functional failure through exposure to common causative factors. The redundant logic trains, reactor trip breakers, and ESF actuation devices are physically separated and electrically isolated. Physically separate channel trays, conduits, and penetrations are maintained upstream from the logic elements of each train.

The protection system components have been qualified by testing under extremes of the normal environment. In addition, components are tested and qualified according to individual requirements for the adverse environment specific to their location that might result from postulated accident conditions. The protection systems are discussed in Section 7.2.

3.1.5.6 Criterion 24, 1967 - Emergency Power for Protection Systems (Category B)

In the event of loss of all offsite power, sufficient alternate sources of power shall be provided to permit the required functioning of the protection systems.

Discussion

The facility is supplied with normal and standby emergency power to provide for the required functioning of the protection systems.

In the event of loss of normal power, emergency ac power is supplied by six diesel generators, as described in Chapter 8. Only four diesels are required to supply the power requirements with one unit in an accident situation and to bring the other to the shutdown condition from full power.

The instrumentation and controls portions of the protection systems are supplied initially from the station batteries and subsequently from the emergency diesel generators. A single failure of any one component will not prevent the required functioning of protection systems.

3.1.5.7 Criterion 25, 1967 - Demonstration of Functional Operability of Protection Systems (Category B)

Means shall be included for testing protection systems while the reactor is in operation to demonstrate that no failure or loss of redundancy has occurred.

Discussion

All reactor protection channels employed in power operation are sufficiently redundant so that individual testing and calibration, without degradation of the protection function or violation of the single failure criterion, can be performed with the reactors at power. Such testing discloses failures or reduction in redundancy that may have occurred. Removal from service of any single channel or component does not result in loss of minimum required redundancy. For example, a two-out-of-three function becomes a one-out-of-two function when one channel is removed.

Semiautomatic testers are built into each of the two logic trains in the reactor protection system. These testers have the capability of testing the major part of the protection system very rapidly while the reactor is at power. Between tests, the testers continuously monitor a number of internal protection system points, including the associated power supplies and fuses. Outputs of the monitors are logically processed to provide alarms for failures in one train and automatic reactor trip for failures in both trains. A self-testing provision is designed into each tester. Additional details can be found in Section 7.2.

3.1.5.8 Criterion 26, 1967 - Protection Systems Fail-Safe Design (Category B)

The reactor protection systems shall be designed to fail into a safe state or into a state established as tolerable on a defined basis if conditions such as disconnection of the system, loss of energy (e.g., electric power, instrument air), or adverse environments (e.g., extreme heat or cold, fire, steam, or water) are experienced.

Discussion

The protection systems are designed with due consideration of the most probable failure modes of the components under various perturbations of the environment and energy sources. Each reactor trip channel is designed on the de-energize-to-trip principle, so loss of power, disconnection, open channel faults, and the majority of internal channel short circuit faults cause the channel to go into its tripped mode. Additional defenses against loss of function are discussed under Criterion 23. The protection system details can be found in Section 7.2.

3.1.6 REACTIVITY CONTROL

GDCs related to reactivity control are presented in this section. A discussion of conformance follows each criterion.

3.1.6.1 Criterion 27, 1967 - Redundancy of Reactivity Control (Category A)

At least two independent reactivity control systems, preferably of different principles, shall be provided.

Discussion

Two independent reactivity control systems are provided for each reactor. These are rod cluster control assemblies (RCCAs) and chemical shim (boration). The RCCAs are inserted into the core by the force of gravity.

The RCCAs can compensate for the reactivity effects of fuel/water temperature changes accompanying power level changes over the full range from full load to no-load at the design maximum load change rate. Automatic control by the assemblies is, however, limited to the range of 15 to 100 percent of power rating for reasons unrelated to reactivity or reactor safety. The assemblies can also compensate for xenon burnout reactivity transients over the allowed range of travel.

The boron system maintains the reactor in the cold shutdown state independent of the position of the control rods and can compensate for all xenon burnout transients. Details of the construction of the RCCA are included in Section 4.2, with the operation discussed in Chapter 7. The means of controlling the boric acid concentration is described in Section 9.3.4.

3.1.6.2 Criterion 28, 1967 - Reactivity Hot Shutdown Capability (Category A)

At least two of the reactivity control systems provided shall be capable of independently making and holding the core subcritical from any hot standby or hot operating condition, including those resulting from power changes, sufficiently fast to prevent exceeding acceptable fuel damage limits.

Discussion

The RCCA system is capable of making and holding the core subcritical from all operating and hot shutdown conditions sufficiently fast to prevent exceeding acceptable fuel damage limits. The chemical shim control is also capable of making and holding the core subcritical, but at a slower rate, and is not employed as a means of compensating for rapid reactivity transients. The RCCA system is, therefore, used in protecting each core from fast transients. Details of the operation and effectiveness of these systems are included in Chapters 4 and 9.

3.1.6.3 Criterion 29, 1967 - Reactivity Shutdown Capability (Category A)

At least one of the reactivity control systems provided shall be capable of making the core subcritical under any condition (including anticipated operational transients) sufficiently fast to prevent exceeding acceptable fuel damage limits. Shutdown margins greater than the maximum worth of the most effective control rod when fully withdrawn shall be provided.

Discussion

As discussed in Chapter 4, the reactors may be made subcritical by the RCCA systems sufficiently fast to prevent exceeding acceptable fuel damage limits, under all anticipated conditions, with the most reactive RCCA fully withdrawn.

3.1.6.4 Criterion 30, 1967 - Reactivity Holddown Capability (Category B)

At least one of the reactivity control systems provided shall be capable of making and holding the core subcritical under any conditions with appropriate margins for contingencies.

Discussion

The boron reactivity (chemical shim) control systems are capable of making and holding the core subcritical under any anticipated condition and with appropriate margin for contingencies. These means are discussed in detail in Chapters 4 and 9. Normal reactivity shutdown capability is provided by rapid control rod insertion. The chemical shim control system permits the necessary shutdown margin to be maintained during long-term xenon decay and plant cooldown.

3.1.6.5 Criterion 31, 1967 - Reactivity Control Systems Malfunction (Category B)

The reactivity control systems shall be capable of sustaining any single malfunction, such as, unplanned continuous withdrawal (not ejection) of a control rod, without causing a reactivity transient which could result in exceeding acceptable fuel damage limits.

Discussion

Reactor shutdown by RCCA insertion is completely independent of the normal control function, since the trip breakers interrupt power to the drive mechanisms regardless of existing control signals. The protection system is designed to limit reactivity transients so that DNBR will exceed the applicable limit value (refer to Sections 4.4.1.1 and 4.4.2.3) for any single malfunction in either reactor control system.

The analysis presented in Chapter 15 shows that for postulated dilution during refueling, startup, or manual or automatic operation at power, the operator has ample time to determine the cause of dilution, terminate the source of dilution, and initiate reboration before the shutdown margin is lost. The facility reactivity control systems are discussed further in Chapter 7, and analyses of the effects of the other possible malfunctions are discussed in Chapter 15. The analyses show that acceptable fuel damage limits are not exceeded in the event of a single malfunction of either system.

3.1.6.6 Criterion 32, 1967 - Maximum Reactivity Worth of Control Rods (Category A)

Limits, which include considerable margin, shall be placed on the maximum reactivity worth of control rods or elements and on rates at which reactivity can be increased to ensure that the potential effects of a sudden or large change of reactivity cannot: (a) rupture the reactor coolant pressure boundary, or (b) disrupt the core, its support structures, or other vessel internals sufficiently to impair the effectiveness of emergency core cooling.

Discussion

The maximum reactivity worth of control rods and the maximum rates of reactivity insertion employing control rods and boron removal are limited to values that could not cause rupture of the RCS boundary or disruptions of the core or vessel internals to a degree that could impair the effectiveness of emergency core cooling.

The appropriate reactivity insertion rate for the withdrawal of RCCA and the dilution of the boric acid in the RCS are determined by safety analyses for the facility. The analyses include appropriate graphs that show the permissible manual withdrawal limits and overlap of function of the several RCCA banks as a function of power. These data on reactivity insertion rates, dilution, and withdrawal limits are also discussed in Section 4.3. The capability of the chemical and volume control system to avoid an inadvertent excessive rate of boron dilution is discussed in Chapter 9. The relationship of the reactivity insertion rates to plant safety is discussed in Chapter 15.

Assurance of core cooling capability following accidents, such as rod ejection, steam line break, etc., is provided by keeping the reactor coolant pressure boundary stresses within faulted condition limits as specified by applicable ASME codes. Structural deformations are also checked and are limited to values that do not jeopardize the operation of needed safety features.

3.1.7 REACTOR COOLANT PRESSURE BOUNDARY

GDCs related to the reactor coolant pressure boundary are presented in this section. A discussion of conformance follows each criterion.

3.1.7.1 Criterion 33, 1967 - Reactor Coolant Pressure Boundary Capability (Category A)

The reactor coolant pressure boundary shall be capable of accommodating without rupture, and with only limited allowance for energy absorption through plastic deformation, the static and dynamic loads imposed on any boundary components as a result of any inadvertent and sudden release of energy to the coolant. As a design reference, this sudden release shall be taken as that which would result from a sudden reactivity insertion such as rod ejection (unless prevented by positive mechanical means), rod dropout, or cold water addition.

Discussion

Each reactor coolant boundary is shown in Chapter 15 to be capable of accommodating, without rupture, the static and dynamic loads imposed as a result of a sudden reactivity insertion such as a rod ejection.

The operation of each reactor is such that the severity of an ejection accident is inherently limited. Since control rod clusters are used to control load variations and core depletion is followed with boron dilution, only the RCCAs in the controlling groups are inserted in the core at power, and these rods are only partially inserted. Rod insertion limit monitors are provided as an administrative aid to the operator to ensure that this condition is met.

3.1.7.2 Criterion 34, 1967 - Reactor Coolant Pressure Boundary Rapid Propagation Failure Prevention (Category A)

The reactor coolant pressure boundary shall be designed to minimize the probability of rapidly propagating type failures. Consideration shall be given: (a) to the notch-toughness properties of materials extending to the upper shelf of the Charpy transition curve, (b) to the state of stress of materials under static and transient loadings, (c) to the quality control specified for materials and component fabrication to limit flaw sizes, and (d) to the provisions for control over service temperature and irradiation effects that may require operational restrictions.

Discussion

Close control is maintained over material selection and fabrication for the RCS. RCS materials exposed to the coolant are corrosion-resistant stainless steel or Inconel. The nil ductility transition temperature (NDTT) of the reactor vessel material samples are established by Charpy V-notch and drop weight tests. The materials testing is consistent with 10 CFR 50, Appendices G and H. These tests also ensure that only materials with adequate toughness properties are used.

As part of the reactor vessel specification, certain additional tests, not specified by the applicable ASME codes, are performed:

(1) Ultrasonic Testing

In addition to code requirements, the performance of a 100 percent volumetric ultrasonic test of reactor vessel plate for shear wave and a post-hydrotest ultrasonic map of all welds in the pressure vessel are required. Cladding bond ultrasonic inspection to more restrictive requirements than code is also required to preclude interpretation problems during inservice inspection.

(2) Radiation Surveillance Program

In the surveillance programs, the evaluation of the radiation damage is based on pre-irradiation and post-irradiation testing of Charpy V-notch and tensile specimens. These programs are directed toward evaluation of the effect of radiation on the fracture toughness of reactor vessel steels based on the transition temperature approach and the fracture mechanics approach, and are in accord with ASTM-E-185, Recommended Practice for Surveillance Tests for Nuclear Reactor Vessels.

The fabrication and quality control techniques used in the fabrication of each RCS, described in Section 5.2, are equivalent to those used for the reactor vessel. The inspections of reactor vessel, pressurizer, piping, pumps, and steam generator are governed by ASME code requirements.

The allowable heatup and cooldown rates as well as the static loading stresses during plant life are predicted, using conservative values for the change in ductility transition temperature due to irradiation.

3.1.7.3 Criterion 35, 1967 - Reactor Coolant Pressure Boundary Brittle Fracture Prevention (Category A)

Under conditions where reactor coolant pressure boundary system components constructed of ferritic materials may be subjected to potential loadings, such as a reactivity-induced loading, service temperatures shall be at least 120°F above the nil ductility transition temperature of the component material if the resulting energy release is expected to be absorbed by plastic deformation, or 60°F above the NDTT of the component material if the resulting energy release is expected to be absorbed within the elastic strain energy range.

Discussion

Sufficient testing and analysis of materials employed in RCS components have been performed to ensure that the required NDTT limits specified in the criterion are met. Removable test capsules installed in the reactor vessel are removed and tested at various times in the plant lifetime to determine the effects of operation on system materials. Details of the testing and analysis programs are included in Chapter 5.
Close control is maintained over material selection and fabrication for the RCS. Materials exposed to the coolant are corrosion-resistant stainless steel or Inconel. Materials testing consistent with 10 CFR 50 ensures that only materials with adequate toughness properties are used.

The fabrication and quality control techniques used in the fabrication of the RCS are equivalent to those used for the reactor vessel. The inspections of reactor vessel, steam generators, pressurizer, pumps, and piping are governed by ASME code requirements.

3.1.7.4 Criterion 36, 1967 - Reactor Coolant Pressure Boundary Surveillance (Category A)

Reactor coolant pressure boundary components shall have provisions for inspection, testing, and surveillance by appropriate means to assess the structural and leaktight integrity of the boundary components during their service lifetime. For the reactor vessel, a material surveillance program conforming with ASTM-E-185-66 shall be provided.

Discussion

The design of the reactor coolant pressure boundary provides for accessibility during service life to the entire internal surface of the reactor vessel and certain external zones of the vessel, including the nozzle to reactor coolant piping welds and the top and bottom heads, except where control rod drive or instrument penetrations prevent access. The reactor arrangement within each containment provides sufficient space for inspection of the external surfaces of the reactor coolant piping, except for the area of pipe within the primary shielding concrete. The inspection capability complements the leakage detection systems in assessing the pressure boundary components' integrity.

Monitoring of the NDT temperature properties of each core region plate, forging, weldment, and associated heat-treated zones are performed in accordance with ASTM-E-185, Recommended Practice for Surveillance Tests on Structural Materials in Nuclear Reactors. Samples of reactor vessel plate materials are retained and cataloged in case future engineering development shows the need for further testing.

The material properties surveillance program includes not only the conventional tensile and impact tests, but also fracture mechanics specimens. The observed shifts in NDTT of the core region materials with irradiation are used to confirm the calculated limits to startup and shutdown transients.

To define permissible operating conditions below NDTT, a pressure range is established that is bounded by a lower limit for pump operation and an upper limit that satisfies reactor vessel stress criteria. To allow for thermal stresses during heatup or cooldown of the reactor vessel, an equivalent pressure limit is defined to compensate for thermal stress as a function of rate of change of coolant temperature. Since the normal operating temperature of the reactor vessel is well above the maximum expected NDTT brittle fracture during normal operation, it is not considered to be a credible mode of failure. Additional details can be found in Section 5.2.

3.1.8 ENGINEERED SAFETY FEATURES

GDCs related to ESFs are presented in this section. A discussion of conformance follows each criterion.

3.1.8.1 Criterion 37, 1967 - Engineered Safety Features Basis for Design (Category A)

Engineered safety features shall be provided in the facility to back up the safety provided by the core design, the reactor coolant pressure boundary, and their protection systems. As a minimum, such engineered safety features shall be designed to cope with any size reactor coolant pressure boundary break up to and including the circumferential rupture of any pipe in that boundary assuming unobstructed discharge from both ends.

Discussion

Engineered safety features are provided to cope with any size reactor coolant pipe break up to and including the circumferential rupture of any pipe in that boundary assuming unobstructed discharge from both ends, and to cope with any steam or feedwater line break up to and including the main steam or feedwater headers.

Limiting the release of fission products from the reactor fuel is accomplished by the emergency core cooling system (ECCS) which, by cooling the core, keeps the fuel in place and substantially intact and limits the metal-water reaction to an acceptable amount. A reinforced concrete, steel-lined containment structure is provided and encloses the entire RCS. It is designed to sustain, without loss of required integrity, all effects of gross equipment failures up to and including the rupture of the largest pipe in the RCS. ESFs are described in Chapter 6.

3.1.8.2 Criterion 38, 1967 - Reliability and Testability of Engineered Safety Features (Category A)

All engineered safety features shall be designed to provide high functional reliability and ready testability. In determining the suitability of a facility for a proposed site, the degree of reliance upon and acceptance of the inherent and engineered safety afforded by the systems, including engineered safety features, will be influenced by the known and the demonstrated performance capability and reliability of the systems, and by the extent to which the operability of such systems can be tested and inspected where appropriate during the life of the plant.

Discussion

A comprehensive program of testing has been formulated for all equipment and instrumentation vital to the functioning of ESFs. The program consists of startup tests of system components and integrated tests of the system. Periodic tests of the activation circuitry and system components, throughout the station lifetime, with maintenance performed as necessary, ensure that the initially high reliability will be maintained and that the system will perform on demand. Details of the test program are provided in the Technical Specifications (Reference 1). The ESFs are described in Chapter 6.

3.1.8.3 Criterion 39, 1967 - Emergency Power for Engineered Safety Features (Category A)

Criterion 39, 1967 is no longer part of the DCPP license basis and has been replaced by GDC-17, 1971 and GDC-18, 1971.

3.1.8.3.1 Criterion 17, 1971 - Electric Power Systems

An onsite electric power system and an offsite electric power system shall be provided to permit functioning of structures, systems, and components important to safety. The safety function for each system (assuming the other system is not functioning) shall be to provide sufficient capacity and capability to assure that (1) specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded as a result of anticipated operational occurrences and (2) the core is cooled and containment integrity and other vital functions are maintained in the event of postulated accidents.

The onsite electric power supplies, including the batteries, and the onsite electric distribution system, shall have sufficient independence, redundancy, and testability to perform their safety functions assuming a single failure.

Electric power from the transmission network to the onsite electric distribution system shall be supplied by two physically independent circuits (not necessarily on separate rights of way) designed and located so as to minimize to the extent practical the likelihood of their simultaneous failure under operating and postulated accident and environmental conditions. A switchyard common to both circuits is acceptable. Each of these circuits shall be designed to be available in sufficient time following a loss of all onsite alternating current power supplies and the other offsite electric power circuit, to assure that specified acceptable fuel design limits and design conditions of the reactor coolant pressure boundary are not exceeded. One of these circuits shall be designed to be available within a few seconds following a loss-of-coolant accident to assure that core cooling, containment integrity, and other vital safety functions are maintained.

Provisions shall be included to minimize the probability of losing electric power from any of the remaining supplies as a result of, or coincident with, the loss of power generated by the nuclear power unit, the loss of power from the transmission network, or the loss of power from the onsite electric power supplies.

Discussion

The DCPP Offsite Power System is designed to supply offsite electrical power by two physically independent circuits. The 230-kV system provides startup and standby power, and is immediately available following a design basis accident to assure that core cooling, containment integrity, and other vital safety functions are maintained. The 500-kV system provides for transmission of the plant's electric power output. The 500-kV connection also provides a delayed access source of offsite power after the main generator is disconnected. A combination of the 230-kV and the 500-kV circuits provides independent sources of offsite power as required by GDC 17, 1971. The onsite emergency power source consists of three diesel generators for each unit.

Both offsite and onsite systems have sufficient independence, capacity, and testability to permit the operation of the ESFs assuming a failure of a single active component in each power system. The combination of two 230-kV lines plus the 500-kV system provides a high degree of assurance that offsite power will be available when required. The 230-kv and 500-kV systems meet the requirements of 1971 GDC 17. Further details are provided in Chapter 8.

3.1.8.3.2 Criterion 18, 1971 - Inspection and Testing of Electric Power Systems

Electric power systems important to safety shall be designed to permit appropriate periodic inspection and testing of important areas and features, such as wiring, insulation, connections, and switchboards, to assess the continuity of the systems and the condition of their components. The systems shall be designed with a capability to test periodically (1) the operability and functional performance of the components of the systems, such as onsite power sources, relays, switches, and buses, and (2) the operability of the systems as a whole and, under conditions as close to design as practical, the full operation sequence that brings the systems into operation, including operation of applicable portions of the protection system, and the transfer of power among the nuclear power unit, the offsite power system, and the onsite power system.

<u>Discussion</u>

The electric power system and its components have provisions for periodic inspection and testing. Electric power components have been provided with convenient and safe features for inspecting and testing to meet the requirements of GDC 18, 1971.

3.1.8.4 Criterion 40, 1967 - Missile Protection (Category A)

Protection for engineered safety features shall be provided against dynamic effects and missiles that might result from plant equipment failures.

<u>Discussion</u>

Use of conservative design methods, segregated routing of piping, provision of missile shield walls, and use of engineered hangers and pipe restraints are incorporated in the design to accommodate dynamic effects of postulated accidents. The various sources of missiles that might affect ESFs have been identified, and protective measures have been devised to minimize these effects (see Section 3.5).

Electrical raceways containing circuits for the ESFs have not been installed in zones where provision against dynamic effects must be made, with a few exceptions. When routing through such zones was necessary, metallic conduits only were used, and conduits containing redundant circuits were separated physically as far as practical.

3.1.8.4.1 Criterion 4, 1987 revision - Environmental and Dynamic Effects Design Bases

Structures, systems, and components important to safety shall be designed to accommodate the effects of and to be compatible with the environmental conditions associated with normal operation, maintenance, testing, and postulated accidents, including loss-of-coolant accidents. These structures, systems, and components shall be appropriately protected against dynamic effects, including the effects of missiles, pipe whipping, and discharging fluids, that may result from equipment failures and from events and conditions outside the nuclear power unit. However, dynamic effects associated with postulated pipe ruptures in nuclear power units may be excluded from the design basis when analyses reviewed and approved by the Commission demonstrate that the probability of fluid system piping rupture is extremely low under conditions consistent with the design basis for the piping.

<u>Discussion</u>

General Design Criterion (GDC) 4, 1987 revision, "Environmental and dynamic effects design bases," states in part, that

> ...dynamic effects associated with postulated pipe ruptures in nuclear power units may be excluded from the design basis when analyses reviewed and approved by the Commission demonstrate that the probability of fluid system piping rupture is extremely low under conditions consistent with the design basis for the piping.

The leak-before-break (LBB) methodology was applied to the primary loops of DCPP Units 1 and 2. The following postulated breaks were eliminated: the six terminal ends in the cold, hot, and crossover legs; a split in the steam generator inlet elbow; and the loop closure weld in the crossover leg. Reference PG&E letter dated March 16, 1992 (DCL-92-059).

The NRC allows the application of LBB technology on the primary piping systems under the broad-scope revision to 10 CFR Part 50, Appendix A, GDC 4 revision (52 FR 4128841295; October 27, 1987). Use of LBB at DCPP has been approved by the NRC in a safety evaluation dated March 2, 1993.

3.1.8.5 Criterion 41, 1967 - Engineered Safety Features Performance Capability (Category A)

Engineered safety features such as emergency core cooling and containment heat removal systems shall provide sufficient performance capability to accommodate partial loss of installed capacity and still fulfill the required safety function. As a minimum, each engineered safety feature shall provide this required safety function assuming a failure of a single active component.

Discussion

The overall capacity of the ESF meets the requirements of 10 CFR 100 for the occurrence of any rupture of a reactor coolant or steam system pipe, including the double-ended rupture of a reactor coolant pipe, known as the design basis accident (DBA). Additional details can be found in Chapters 6 and 15.

3.1.8.6 Criterion 42, 1967 - Engineered Safety Features Components Capability (Category A)

Engineered safety features shall be designed so that the capability of each component and system to perform its required function is not impaired by the effects of a loss of coolant accident.

Discussion

Instrumentation, motors, cables, and penetrations located inside the containment are selected to meet the most adverse accident conditions to which they may be subjected. These items are either protected from containment accident conditions or are designed to withstand, without failure, exposure to the worst combination of temperature, pressure, and humidity expected during the required operational period.

The ECCS pipes serving each loop are anchored at the missile barrier in each loop area to restrict potential accident damage to the portion of piping beyond this point. The anchorage is designed to withstand, without failure, the thrust force exerted by any branch line severed from the reactor coolant pipe and discharging fluid to the atmosphere, and to withstand a bending moment equivalent to that producing failure of the piping under the action of a free-end discharge to atmosphere or motion of the broken reactor coolant pipe to which the emergency core cooling pipes are connected. This prevents possible failure at any point upstream from the support point including the branch line connection into the piping header. Chapter 6 contains the details of the containment structure and ESFs.

3.1.8.7 Criterion 43, 1967 - Accident Aggravation Prevention (Category A)

Engineered safety features shall be designed so that any action of the engineered safety features that might accentuate the adverse aftereffects of a loss of normal cooling is avoided.

Discussion

The reactor is maintained subcritical following a pipe rupture accident. Introduction of borated cooling water into the core does not result in a net positive reactivity addition. The control rods insert and remain inserted. The supply of water by the ECCS to cool hot core cladding does not produce significant water-metal reactions. The delivery of cold emergency core cooling water to the reactor vessel following accidental expulsion of reactor coolant does not cause further loss of integrity of the RCS boundary. The ESFs are discussed in detail in Chapter 6.

3.1.8.8 Criterion 44, 1967 - Emergency Core Cooling Systems Capability (Category A)

At least two emergency core cooling systems, preferably of different design principles, each with a capability for accomplishing abundant emergency core cooling, shall be provided. Each emergency core cooling system and the core shall be designed to prevent fuel and clad damage that would interfere with the emergency core cooling function and to limit the clad metal-water reaction to negligible amounts for all sizes of breaks in the reactor coolant pressure boundary, including the double-ended rupture of the largest pipe. The performance of each emergency core cooling system shall be evaluated conservatively in each area of uncertainty. The systems shall not share active components and shall not share other features or components unless it can be demonstrated that (a) the capability of the shared feature or component to perform its required function can be readily ascertained during reactor operation, (b) failure of the shared feature or component does not initiate a loss of coolant accident, and (c) capability of the shared feature or component to perform its required function is not impaired by the effects of a loss of coolant accident and is not lost during the entire period this function is required following the accident.

Discussion

By combining the use of passive accumulators with two centrifugal charging pumps (CCP1 and CCP2), two safety injection pumps, and two RHR pumps, emergency core cooling is provided even if there should be a failure of any single component in any system. The ECCS employs a passive system of accumulators that do not require any external signals or source of power for their operation to cope with the short-term cooling requirements of large reactor coolant pipe breaks. Two independent and redundant high-pressure flow and pumping systems, each capable of the required emergency cooling, are provided for small break protection and to keep the core submerged after the accumulators have discharged following a large break. These systems are arranged so that the single failure of any active component does not interfere with meeting the short-term cooling requirements.

Borated water is injected into the RCS by accumulators, safety injection pumps, RHR pumps, and charging pumps. Pump design includes consideration of fluid temperature and containment pressure in accordance with AEC Safety Guide (SG) 1. The failure of any single active component or the development of excessive leakage during the long-term cooling period does not interfere with the ability to meet necessary long-term cooling objectives with one of the systems.

The primary function of the ECCS is to deliver borated cooling water to the reactor core in the event of a LOCA. This limits the fuel cladding temperature and thereby ensures that the core will remain intact and in place, with its essential heat transfer geometry preserved. This protection is afforded for:

(1) All pipe break sizes up to and including the hypothetical circumferential rupture of a reactor coolant loop

(2) A loss of coolant associated with a rod ejection accident

The basic criteria for LOCA evaluations are (a) no cladding melting will occur, (b) zirconium-water reactions will be limited to an insignificant amount, and (c) the core geometry will remain essentially in place and intact so that effective cooling of the core will not be impaired. The zirconium-water reactions will be limited to an insignificant amount so that the accident:

(1) Does not interfere with the emergency core cooling function to limit cladding temperatures

(2) Does not produce hydrogen in an amount that, when burned, would cause the containment pressure to exceed the design value

For any rupture of a steam pipe and the associated uncontrolled heat removal from the core, the ECCS adds shutdown reactivity so that with a stuck rod, no offsite power, and minimum ESF, there is no consequential damage to the primary system and the core remains in place and intact. With no stuck rod, offsite power, and all equipment operating at design capacity, there is insignificant cladding rupture. The ECCS is described in Section 6.3. Chapter 15 provides the analysis for the LOCA.

3.1.8.9 Criterion 45, 1967 - Inspection of Emergency Core Cooling Systems (Category A)

Design provisions shall be made to facilitate physical inspection of all critical parts of the emergency core cooling system, including reactor vessel internals and water injection nozzles.

Discussion

Design provisions facilitate access to the critical parts of the reactor vessel internals, injection nozzles, pipes, and valves for visual or nondestructive inspection.

The components outside containment are accessible for leaktightness inspection during operation of the reactor.

Details of the inspection program for the reactor vessel internals are included in Section 5.4. Information on inspection for the ECCS is provided in Section 6.3.

3.1.8.10 Criterion 46, 1967 - Testing of Emergency Core Cooling System Components (Category A)

Design provisions shall be made so that active components of the emergency core cooling system, such as pumps and valves, can be tested periodically for operability and required functional performance.

Discussion

The design provides for periodic testing of both active and passive components of the ECCS for operability and functional performance.

Preoperational performance tests of the components were performed in the manufacturer's shop. Initial system flow tests demonstrate proper functioning of the system. Thereafter, periodic tests demonstrate that components are functioning properly.

Each active component of the ECCS may be individually actuated on the normal power source at any time during plant operation to demonstrate operability. The centrifugal charging pumps are part of the charging system, and this system is in continuous operation during plant operation. The test of the safety injection pumps employs the minimum flow recirculation test line that connects back to the refueling water storage tank (RWST). Remotely operated valves are exercised and actuation circuits tested. The automatic actuation circuitry, valves, and pump breakers also may be checked during integrated system tests performed during a planned cooldown of the RCS.

Details of the ECCS are found in Section 6.3. Performance under accident conditions is evaluated in Chapter 15. Periodic testing procedures are listed in Section 13.5.

3.1.8.11 Criterion 47, 1967 - Testing of Emergency Core Cooling Systems (Category A)

A capability shall be provided to test periodically the delivery capability of the emergency core cooling system at a location as close to the core as is practical.

Discussion

Design provisions include special instrumentation, testing, and sampling lines to perform tests during plant shutdown to demonstrate proper operation of the ECCS. A test signal is applied to initiate automatic action. The test demonstrates the operation of the valves, pump circuit breakers, and automatic circuitry. In addition, the periodic recirculation to the RWST can verify that the safety injection pumps attain required discharge heads. During a refueling outage the full flow capability of each injection pump can be verified.

Details of the ECCS are found in Section 6.3. Performance under accident conditions is evaluated in Chapter 15.

3.1.8.12 Criterion 48, 1967 - Testing of Operational Sequence of Emergency Core Cooling Systems (Category A)

A capability shall be provided to test under conditions as close to design as practical the full operational sequence that would bring the emergency core cooling system into action, including the transfer to alternate power sources.

Discussion

The design provides for capability to test initially, to the extent practical, the full operational sequence up to design conditions, including transfer to alternative power sources for the ECCS, to demonstrate the state of readiness and capability of the system. This functional test is performed with the RCS initially cold and at low pressure. The ECCS valving is set to initially simulate the system alignment for plant power operation.

Details of the ECCS are found in Section 6.3.

3.1.8.13 Criterion 49, 1967 - Containment Design Basis (Category A)

The containment structure, including access openings and penetrations, and any necessary containment heat removal systems shall be designed so containment structure can accommodate, without exceeding the design leakage rate, pressures and temperatures resulting from the largest credible energy release following a loss of coolant accident, including a considerable margin for effects from metal-water or other chemical reactions that could occur as a consequence of failure of emergency core cooling systems.

Discussion

The containment, including access openings and penetrations, has a design pressure of 47 psig. The greatest transient peak pressure associated with a postulated rupture of the piping in the RCS and the calculated effects of metal-water reaction do not exceed this value. The containment is strength tested at 54 psig.

The reactor containment structure and penetrations, with the aid of containment heat removal systems, are designed to limit radiation doses resulting from leakage of radioactive fission products from the containment to below 10 CFR 100 values, assuming the largest credible energy release following a LOCA, including a margin to cover the effects of metal-water or other undefined energy sources. The containment design is described in detail in Section 3.8.1.

3.1.8.14 Criterion 50, 1967 - NDT Requirement for Containment Material (Category A)

Principal load carrying components of ferritic materials exposed to the external environment shall be selected so that their temperatures under normal operating and testing conditions are not less than 30°F above nil ductility transition (NDT) temperature.

Discussion

The selection and use of containment structure materials comply with the applicable codes and standards.

The nil ductility transition temperature (NDTT) requirement of Criterion 50 reflects the requirements of the ASME B&PV Code, Section III, 1968 Edition for Class B (containment) vessels. Section N-1210 requires an NDTT at least 30°F below the lowest service temperature. NDTT may be determined either by Charpy V-notch test or dropweight test. Using the Charpy V-notch test, acceptance criteria specified by the Code depend on the material specification and range from 15 to 20 ft-lbs minimum average.

During construction, Charpy V-notch test temperature for material exposed to external environments was established by subtracting 30°F from the lowest service temperature. The lowest service temperature was taken to be the design 24-hour mean-low ambient temperature for the site of 30°F. Thus the Charpy V-notch test temperature was 0°F.

The containment liner is enclosed within the containment structure and thus not directly exposed to the temperature of the external environment and not subject to Criterion 50, 1967. Nevertheless, the design specification required Charpy V-notch tests at 20°F for the containment liner. This corresponds to a lowest service temperature of 50°F during operation.

During construction, compliance with Criterion 50, 1967 and the ASME Code was ensured by specifying appropriate test temperatures and minimum acceptance values for Charpy V-notch tests as part of the material specification. Mill test reports showing the Charpy V-notch test results were provided as documentation.

ASME Section III requirements have evolved over the years to reflect the industry's better understanding of the behavior of ferritic steel materials. The current ASME Code requirements for notch toughness testing for metallic containment vessels are contained in Subsection NE-2300. ASME has determined that an adequate level of toughness is ensured by performing the Charpy V-notch tests at or below the lowest service temperature rather than 30°F below the lowest service temperature as previously required. Acceptance criteria are based on material thickness rather than material specification and range from 20 to 25 ft.-lbs. minimum average or 20-25 mils lateral expansion minimum average. Notch toughness testing is not required for material 5/8 inch or less in thickness. For repair, replacement, or alteration or ferritic containment material subject to Criterion 50, 1967, the notch toughness testing requirements of NE-2300 will be used in lieu of the original requirements. Charpy V-notch testing will be performed at or below the lowest service temperature and materials 5/8 inch or less in thickness will be exempt from Charpy V-notch testing.

The concrete containment structure is not susceptible to a low temperature brittle fracture.

Further information on containment structure materials appears in Section 3.8.1.

3.1.8.15 Criterion 51, 1967 - Reactor Coolant Pressure Boundary Outside Containment (Category A)

If part of the reactor coolant pressure boundary is outside the containment, appropriate features as necessary shall be provided to protect the health and safety of the public in case of an accidental rupture in that part. Determination of the appropriateness of features such as isolation valves and additional containment shall include consideration of the environmental and population conditions surrounding the site.

Discussion

The reactor coolant pressure boundary is defined as those piping systems and components that contain reactor coolant at design pressure and temperature. With the exception of the reactor coolant sampling lines, the entire reactor coolant pressure boundary, as defined above, is located entirely within the containment structure. All sampling lines are provided with remotely operated valves for isolation in the event of a failure. These valves also close automatically on a containment isolation signal. Sampling lines are only used during infrequent sampling and can readily be isolated.

All other piping and components that may contain reactor coolant are low-pressure, low temperature systems which would yield minimal environmental doses in the event of failure.

The sampling system and low-pressure systems are described in Chapter 9. An analysis of malfunctions in these systems is included in Chapter 15.

3.1.8.16 Criterion 52, 1967 - Containment Heat Removal Systems (Category A)

Where active heat removal systems are needed under accident conditions to prevent exceeding containment design pressure, at least two systems, preferably of different principles, each with full capacity, shall be provided.

Discussion

Two separate heat removal systems, the containment spray system (CSS) and the containment fan coolers, are provided to remove heat from the containment following an accident. The design cooling rates of the two systems at the containment design pressure and temperature conditions are the same. The heat removal capability of either system is sufficient to rapidly reduce the containment pressure following an accident.

A second purpose served by the CSS is to remove radioactive iodine isotopes from the containment atmosphere should these fission products be released in the event of an accident. The system is designed to deliver enough sodium hydroxide mixed with the borated spray water from the RWST to provide pH control for iodine removal when mixed with the other sources of water in the containment recirculation sump. The containment heat removal systems are described in Section 6.2.

3.1.8.17 Criterion 53, 1967 - Containment Isolation Valves (Category A)

Penetrations that require closure for the containment function shall be protected by redundant valving and associated apparatus.

Discussion

Penetrations that require closure for the containment function are provided with at least two barriers. Additional details can be found in Section 6.2.

The Containment Isolation System is designed to meet the 1971 General Design Criteria (GDC) requirements with exceptions. The component cooling water penetration to the excess letdown heat exchanger is an exception that does not meet the 1971 GDC because of commitments to design and construction made prior to the issuance of the 1971 GDC; this penetration does comply with the 1967 GDC. Refer to Section 6.2.4.

3.1.8.17.1 Criterion 54, 1971 - Piping Systems Penetrating Containment

Piping systems penetrating primary reactor containment shall be provided with leak detection, isolation, and containment capabilities having redundancy, reliability, and performance capabilities which reflect the importance to safety of isolating these piping systems. Such piping systems shall be designed with a capability to test periodically the operability of the isolation valves and associated apparatus and to determine if valve leakage is within acceptable limits.

Discussion

The containment isolation design provides for a double barrier at the containment penetration in those fluid systems that are not required to function following a design basis event. Piping systems penetrating the containment are provided with test vents and test connections or have other provisions to allow periodic leakage testing.

Those automatic isolation valves that do not restrict normal plant operation are periodically tested to ensure operability. Refer to Section 6.2.4 for further discussion and details of the capability for periodic leakage rate testing.

3.1.8.17.2 Criterion 55, 1971 - Reactor Coolant Pressure Boundary Penetrating Containment

Each line that is part of the reactor coolant pressure boundary and that penetrates primary reactor containment shall be provided with containment isolation valves as follows, unless it can be demonstrated that the containment isolation provisions for a specific class of lines, such as instrument lines, are acceptable on some other defined basis:

(1) One locked closed isolation valve inside and one locked closed isolation valve outside containment; or

(2) One automatic isolation valve inside and one locked closed isolation valve outside containment; or

(3) One locked closed isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment; or

(4) One automatic isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment.

Isolation valves outside containment shall be located as close to containment as practical and upon loss of actuating power, automatic isolation valves shall be designed to take the position that provides greater safety.

Other appropriate requirements to minimize the probability or consequences of an accidental rupture of these lines or of lines connected to them shall be provided as necessary to assure adequate safety. Determination of the appropriateness of these requirements, such as higher quality in design, fabrication, and testing, additional provisions for inservice inspection, protection against more severe natural phenomena, and additional isolation valves and containment, shall include consideration of the population density, use characteristics, and physical characteristics of the site environs.

Discussion

The reactor coolant pressure boundary is defined as those piping systems and components that contain reactor coolant at design pressure and temperature. With the exception of the reactor coolant sampling lines, the entire reactor coolant pressure boundary, as defined above, is located entirely within the containment structure. All sampling lines are provided with remotely operated valves for isolation in the event of a failure. These valves also close automatically on a containment isolation signal. Sampling lines are only used during infrequent sampling and can readily be isolated. Further details are provided in Section 6.2.4.

3.1.8.17.3 Criterion 56, 1971 - Primary Containment Isolation

Each line that connects directly to the containment atmosphere and penetrates primary reactor containment shall be provided with containment isolation valves as follows, unless it can be demonstrated that the containment isolation provisions for a specific class of lines, such as instrument lines, are acceptable on some other defined basis:

(1) One locked closed isolation valve inside and one locked closed isolation valve outside containment; or

(2) One automatic isolation valve inside and one locked closed isolation valve outside containment; or

(3) One locked closed isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment; or

(4) One automatic isolation valve inside and one automatic isolation valve outside containment. A simple check valve may not be used as the automatic isolation valve outside containment.

Isolation valves outside containment shall be located as close to containment as practical and upon loss of actuating power, automatic isolation valves shall be designed to take the position that provides greater safety.

Discussion

Each Line that connects directly to the containment atmosphere and penetrates the primary reactor containment is provided with containment isolation valves or other acceptable barriers, as required. Refer to Section 6.2.4 for further discussion.

3.1.8.17.4 Criterion 57, 1971 - Closed System Isolation Valves

Each line that penetrates primary reactor containment and is neither part of the reactor coolant pressure boundary nor connected directly to the containment atmosphere shall have at least one containment isolation valve which shall be either automatic, or locked closed, or capable of remote manual operation. This valve shall be outside containment and located as close to the containment as practical. A simple check valve may not be used as the automatic isolation valve.

Discussion

Each line that penetrates the reactor containment in each unit, and is neither part of the reactor coolant pressure boundary nor connected directly to the containment atmosphere, has at least one containment isolation valve located outside the containment as close to the containment as practicable. Refer to Section 6.2.4 for further discussion.

3.1.8.18 Criterion 54, 1967 - Containment Leakage Rate Testing (Category A)

Containment shall be designed so that an integrated leakage rate test can be conducted at design pressure after completion and installation of all penetrations and the leakage rate measured over a sufficient period of time to verify its conformance with required performance.

Discussion

The containment structure design permits preoperational leakage rate tests, including an integrated leakage rate test of the containment structure and sensitive leakage rate tests of the penetrations and weld channels.

The integrated leakage rate test, which is at design pressure and could extend to a period of at least 24 hours, verifies that the structure leakage rate is less than the allowable value. Subsequent leakage tests are conducted at a pressure greater than or equal to 25 psig. A sensitive leakage rate test can be performed by pressurizing the double penetrations, the spaces between resilient seals on penetrations, and the weld channels at slightly above design pressure. This test would be conducted with the containment structure at atmospheric pressure.

The leakage rate tests and the sensitive leakage rate test demonstrate the integrity of the double leakage barriers provided by the penetrations and the overall integrity of the containment structure. The criterion for acceptance is that the measured leakage rate be less than 0.10 percent of the containment free volume per day.

Further details of the integrated leakage rate test and the sensitive leakage rate test provisions appear in Section 3.8.

3.1.8.19 Criterion 55, 1967 - Containment Periodic Leakage Rate Testing (Category A)

The containment shall be designed so that integrated leakage rate testing can be done periodically at design pressure during plant lifetime.

Discussion

The containment structure is provided with testable weld channels and penetrations so that periodic sensitive leakage rate tests at design pressure can be made of those areas where leakage may occur. The containment structure is designed to permit periodic full integrated leakage rate tests at reduced pressure.

3.1.8.20 Criterion 56, 1967 - Provisions for Testing of Penetrations (Category A)

Provisions shall be made for testing penetrations which have resilient seals or expansion bellows to permit leaktightness to be demonstrated at design pressure at any time.

Discussion

All penetrations are provided with a volume that can be pressurized to test for leaktightness. There are three configurations used: (a) weld channels over the penetration welds, (b) an annular space between the penetration insert and the sleeve, which is sealed at both ends, and (c) double resilient seals with a gap between the seals.

Further details appear in Section 3.8.

3.1.8.21 Criterion 57, 1967 - Provisions for Testing of Isolation Valves (Category A)

Capability shall be provided for testing functional operability of valves and associated apparatus essential to the containment function for establishing that no failure has occurred and for determining that valve leakage does not exceed acceptable limits.

Discussion

Capability is provided to the extent practical for testing the functional operability of valves and associated apparatus and the leakage during periods of reactor shutdown.

Initiation of containment isolation employs coincidence circuits that allow checking of the operability and calibration of one channel at a time. Removal or bypass of one signal channel places that channel in the tripped mode. Section 6.2 describes in detail the testing of isolation valves.

3.1.8.22 Criterion 58, 1967 - Inspection of Containment Pressure-Reducing Systems (Category A)

Design provisions shall be made to facilitate the periodic physical inspection of all important components of the containment pressure-reducing systems, such as, pumps, valves, spray nozzles, torus, and sumps.

Discussion

Where practicable, all active components and passive components of the containment cooling system are inspected periodically to demonstrate system readiness. The pressure-containing systems are inspected for leaks from pump seals, valve packing, flanged joints, and safety valves. During operational testing of the containment spray pumps, the portions of the system subjected to pump pressure are inspected for leaks. The containment fan coolers are normally in use, which provides an additional check on the readiness of the system. Five fan coolers are provided. Each is sized for one-quarter the capacity needed to maintain the containment temperature below 120°F during normal plant operation. Following a LOCA, two of the five fan coolers provide sufficient capacity to maintain containment pressure below design value when used in conjunction with one containment spray pump during the injection phase. During the recirculation phase, containment spray operation is not required.

Additional details are found in Section 6.2.

3.1.8.23 Criterion 59, 1967 - Testing of Containment Pressure-Reducing Systems Components (Category A)

The containment pressure-reducing systems shall be designed so that active components, such as pumps and valves, can be tested periodically for operability and required functional performance.

Discussion

To the extent practicable, active components of the containment fan coolers are given preoperational performance tests after installation. Since these coolers are in use during normal operation, they are continually subjected to operational tests. The same is true of the component cooling water system that supplies the cooling water for the fan coolers. Each unit can be isolated during plant operation and subjected to a leak test to determine that the leaktight integrity of the unit has not been lost.

Similarly, active components in the CSS are given preoperational performance tests after installation. Periodic tests demonstrate that components are functioning properly. Tests are performed after any component maintenance affecting operability. The containment systems are described in detail in Section 6.2.

3.1.8.24 Criterion 60, 1967 - Testing of Containment Spray Systems (Category A)

A capability shall be provided to test periodically the delivery capability of the containment spray system at a position as close to the spray nozzles as is practical.

Discussion

Permanent test lines for all the containment spray loops are located so that all components up to the isolation valves at the containment can be tested.

The air test lines for checking that spray nozzles are not obstructed are connected upstream of the spray ring isolation valves. Airflow through the nozzles is monitored by positive means. Details of the CSS are found in Section 6.2.

3.1.8.25 Criterion 61, 1967 - Testing of Operational Sequence of Containment Pressure-Reducing Systems (Category A)

A capability shall be provided to test under conditions as close to the design as practical the full operational sequence that would bring the containment pressure-reducing systems into action, including the transfer to alternate power sources.

Discussion

The design provides for capability to test, to the extent practicable, the full operational sequence for the CSS and the containment fan coolers to demonstrate the state of readiness for those sections of the systems not normally functioning during plant operation. Containment systems are described in detail in Section 6.2.

3.1.8.26 Criterion 62, 1967 - Inspection of Air Cleanup Systems (Category A)

Design provisions shall be made to facilitate physical inspection of all critical parts of containment air cleanup systems, such as, ducts, filters, fans, and dampers.

Discussion

The CSS, utilizing sodium hydroxide, serves as the air cleanup system. Where practical, all components of the CSS are inspected periodically to demonstrate system readiness. Special attention is given to critical parts such as pipes, pumps, nozzles, and storage facilities for sodium hydroxide. Additional details are found in Section 6.2.

3.1.8.27 Criterion 63 , 1967- Testing of Air Cleanup Systems Components (Category A)

Design provisions shall be made so that active components of the air cleanup systems, such as fans and dampers, can be tested periodically for operability and required functional performance.

Discussion

The CSS, utilizing sodium hydroxide, serves as the air cleanup system. The active components in the CSS are given preoperational performance tests after installation. Permanent test lines for all containment spray loops are located so that all components up to the isolation valves at the containment may be tested. The nozzles are tested by airflow that is monitored by positive means. The fan coolers are normally in use, which provides a check on the operability of the system. Additional details are found in Section 6.2.

3.1.8.28 Criterion 64, 1967 - Testing Air Cleanup Systems (Category A)

A capability shall be provided for in situ periodic testing and surveillance of the air cleanup systems to ensure: (a) filter bypass paths have not developed and (b) filter and trapping materials have not deteriorated beyond acceptable limits.

Discussion

The CSS water flow can be tested through the permanent test lines, permitting the test of the system up to the isolation valves at the containment. Permanent air testing lines, connected upstream of the containment isolation valves, permit operability testing of piping and nozzles beyond the isolation valves. Periodically, a sample of the sodium hydroxide solution is tested to ensure proper concentration. The fan coolers are normally in use and, therefore, the readiness of the system is verified. Additional details are found in Section 6.2.

3.1.8.29 Criterion 65, 1967 - Testing of Operational Sequence of Air Cleanup Systems (Category A)

A capability shall be provided to test, under conditions as close to design as practical, the full operational sequence that would bring the air cleanup systems into action, including the transfer to alternate power sources and the design air flow delivery capability.

Discussion

The CSS design provides for the capability to test initially, to the extent practicable, the full operational sequence from the CSS to demonstrate the state of readiness of the system. The fan cooler system is normally in use, which verifies its readiness. Details are contained in Section 6.2.

3.1.9 FUEL AND WASTE STORAGE SYSTEMS

GDC for fuel and waste storage systems are presented in this section. A discussion of conformance follows each criterion.

3.1.9.1 Criterion 66, 1967 - Prevention of Fuel Storage Criticality (Category B)

Criticality in new and spent fuel storage shall be prevented by physical systems or processes. Such means as geometrically safe configurations shall be emphasized over procedural controls.

Discussion

During reactor vessel head removal and while loading and unloading fuel from the reactor, the boron concentration in the refueling water and the spent fuel pool is maintained at not less than that required to shut down the core to a k_{eff} = 0.95.

Borated water is used to fill the spent fuel storage pools at a concentration comparable to that used in the reactor cavity and refueling canal during refueling operations. The fuel is stored vertically in an array with sufficient center-to-center distance between assemblies to ensure that, including uncertainties, a k_{eff} of less than or equal to 0.95 if the fuel racks are flooded with borated water, and a k_{eff} < 1.0, even if unborated water is used to fill the pool. The fuel storage and handling details are found in Section 9.1.

3.1.9.2 Criterion 67, 1967 - Fuel and Waste Storage Decay Heat (Category B)

Reliable decay heat removal systems shall be designed to prevent damage to the fuel in storage facilities that could result in radioactivity release to plant operating areas or the public environs.

Discussion

Refueling water provides a reliable and adequate cooling medium for spent fuel transfer, and heat removal is provided by an auxiliary cooling system. Natural radiation and convection is adequate for cooling the holdup tanks. The auxiliary systems are discussed in detail in Chapter 9.

3.1.9.3 Criterion 68, 1967 - Fuel and Waste Storage Radiation Shielding (Category B)

Shielding for radiation protection shall be provided in the design of spent fuel and waste storage facilities as required to meet the requirements of 10 CFR 20.

Discussion

The spent fuel pool is designed to provide a sufficient depth of water over the top of the active portion of a spent fuel assembly during handling operations so that, in all cases, operator dose levels will be within the requirements of 10 CFR 20.

Fuel handling and storage is described in Chapter 9. Shielding design is described in Chapter 12.

3.1.9.4 Criterion 69, 1967 - Protection Against Radioactivity Release from Spent Fuel and Waste Storage (Category B)

Containment of fuel and waste storage shall be provided if accidents could lead to release of undue amounts of radioactivity to the public environs.

Discussion

The spent fuel area is enclosed and maintained under negative pressure. All ventilation air is passed through HEPA filters prior to being released to the plant vent. In the event of an accident, high activity would be detected by the radiation monitor (see Section 11.4), and the exhaust air would be diverted through charcoal filters. For radioactive waste storage, refer to the detailed discussion in Chapter 11. Failure of a gas decay tank has been postulated and analyzed in Chapter 15.

3.1.10 PLANT EFFLUENTS

The general design criterion related to plant effluents is presented in this section. A discussion of conformance follows the criterion.

3.1.10.1 Criterion 70, 1967 - Control of Releases of Radioactivity to the Environment (Category B)

The facility design shall include those means necessary to maintain control over the plant radioactive effluents, whether gaseous, liquid, or solid. Appropriate holdup capacity shall be provided for retention of gaseous, liquid, or solid effluents, particularly where unfavorable environmental conditions can be expected to require operational limitations upon the release of radioactive effluents to the environment. In all cases, the design for radioactivity control shall be justified (a) on the basis of 10 CFR 20 requirements for normal operations and for any transient situation that might reasonably be anticipated to occur and (b) on the basis of 10 CFR 100 dosage level guidelines for potential reactor accidents of exceedingly low probability of occurrence except that reduction of the recommended dosage levels may be required where high population densities or very large cities can be affected by the radioactive effluents.

Discussion

Waste handling systems are incorporated in the facility design for processing and/or retention of radioactive wastes from normal operation, with appropriate controls and monitors to ensure that releases do not exceed the limits of 10 CFR 20. The radioactive waste processing system, the design criteria, and amounts of estimated releases of radioactive effluents to the environment are described in Chapter 11. Details of the monitoring system are found in Section 11.4.

The facility is designed to prevent radioactivity release during accidents from exceeding the limits of 10 CFR 100. The containment system, which forms a barrier to the escape of fission products should a loss of coolant occur, is described in Section 6.2. Postulated accidents that could release radioactivity to the environment are analyzed in Chapter 15.

3.1.11 REFERENCES

1. Technical Specifications, Diablo Canyon Power Plant Units 1 and 2, Appendix A to License Nos. DPR-80 and DPR-82, as amended.
2. Deleted in Revision 3.
3. Deleted in Revision 3.
4. Deleted in Revision 20
5. Diablo Canyon Plant Units 1 and 2 Final Safety Analysis Report (FSAR), July 1973.
6. Diablo Canyon Plant Units 1 and 2 FSAR Amendment 85, 1980.

3.2 CLASSIFICATION OF STRUCTURES, SYSTEMS, AND COMPONENTS

This section describes the DCPP classification system for structures, systems, and components (SSCs) that is used to implement quality standards in conformance with GDC 1, 1967.

It is recognized that during the design and construction of DCPP Units 1 and 2, significant industry and regulatory changes were made in establishing common methods of classification, e.g., American National Standards Institute (ANSI) standard N18.2, Draft August 1970 (Reference 1), Safety Guide 26, March 1972 (Reference 2), Safety Guide 29, June 1972 (Reference 3), and Regulatory Guide 1.143, October 1979, Revision 1 (Reference 6). However these methods all differ slightly in detail from those used for DCPP. Although these regulatory and industry documents are similar, these are not the DCPP Licensing Basis.

DCPP utilizes three primary classification categories (PG&E Design Class, PG&E Quality Assurance Classification and PG&E Quality/Code Class) as defined in Section 3.2.2. Although this section makes comparisons to the above documents, it is clear that the DCPP Licensing Basis is the PG&E classification categories.

GDC 1, 1967, requires that structures, systems and components important to safety be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed.

3.2.1 DESIGN BASES

3.2.1.1 General Design Criterion 1, 1967 - Quality Standards

DCPP SSCs that are essential to the prevention of accidents which could affect the public health and safety, or mitigation of their consequences are identified and designed, fabricated, and erected to quality standards that reflect the importance of the safety function to be performed.

3.2.1.2 General Design Criterion 2, 1967 - Performance Standards

DCPP SSCs that are essential to prevention of accidents which could affect the public health and safety, or to mitigation of their consequences are designed, fabricated, and erected to withstand the effects of natural phenomena such as earthquakes, tornadoes, flooding conditions, winds, and other local site effects.

3.2.1.3 10 CFR 50.55a - Codes and Standards

Title 10 of the U.S. Code of Federal Regulations, Section 50.55a (10 CFR 50.55a), requires that certain components of the reactor coolant pressure boundary be designed, fabricated, erected, and tested in accordance with the requirements for Class A components of Section III of the American Society of Mechanical Engineers (ASME)

Boiler and Pressure Vessel Code, or the most recently available industry codes and standards at the time of construction authorization application. PG&E Quality/Code Class I is applied to those components of the reactor coolant pressure boundary and implements the quality standards that satisfy the requirements of 10 CFR 50.55a. PG&E Quality/Code Class I components of the reactor coolant pressure boundary are listed in the DCPP Q-List (Reference 8), along with the industry codes and standards used for their design, fabrication, erection, and test.

3.2.1.4 10 CFR Part 50 Appendix B - Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants

Classification of SSCs includes the basis of Appendix B to 10 CFR Part 50, Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants, which requires that SSCs important to safety be designed and constructed in accordance with the quality assurance requirements described in Appendix B. Therefore, the requirements of the DCPP Quality Assurance Program apply to all SSCs classified as PG&E Design Class I. This ensures that plant features important to safety have met the requirements of Appendix B. Specific quality assurance requirements may also be applied to selected PG&E Design Class II features.

3.2.2 CLASSIFICATION CATEGORIES

DCPP SSCs are classified using three primary categories: PG&E Design Class, PG&E Quality Assurance Classification, and PG&E Quality/Code Class. The requirements and interrelationships of these categories are described in the sections below and Table 3.2-1.

The general applicability and requirements of the various PG&E classification categories are provided in Tables 3.2-1, 3.2-2 and 3.2-4.

The classifications of specific SSCs are provided in the DCPP Q-List (Reference 8). The DCPP Q-List is controlled by a PG&E procedure. The procedure requires that all changes to the contents of the DCPP Q-List be reviewed pursuant to the requirements of 10 CFR 50.59. Piping schematic drawings are illustrated in controlled engineering drawings in Figures 3.2-1 through 3.2-27 in Table 1.6-1.

3.2.2.1 PG&E Design Class

This section describes the PG&E design classes that are used to classify individual SSC's.

The design class for individual SSCs, including specific requirements and exceptions, are identified in the Q-list (Reference 8).

Note that there is no direct correlation between the design class of an SSC and the seismic design requirements. Refer to Section 3.2.2.4 for the discussion of seismic qualification requirements.

3.2.2.1.1 PG&E Design Class I

PG&E Design Class I is applicable to SSCs that are important to safety, including SSCs required to assure the following:

(a) The integrity of the reactor coolant pressure boundary.

(b) The capability to shut down the reactor and maintain it in a safe shutdown condition.

(c) The capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of 10 CFR Part 100.

(d) All plant features designated as PG&E Design Class I are designed to remain functional when subjected to the additional forces associated with the design basis earthquakes that they are required to withstand: the Design earthquake (DE), the Double Design earthquake (DDE) and/or the Hosgri earthquake (HE). Refer to Section 3.2.2.4 for additional information regarding the earthquake requirements for the design of SSCs.

The following SSCs, including their foundations and supports, are classified as PG&E Design Class I:

(1) The reactor coolant pressure boundary

(2) The reactor core and reactor vessel internals

(3) Systems (Refer to Notes (i) and (iii) below) or portions of systems that are required for emergency core cooling, post-accident containment heat removal, or post-accident containment atmosphere cleanup (Refer to Note (iv) below)

(4) Systems or portions of systems that are required for reactor shutdown and residual heat removal

(5) Those portions of the main steam, feedwater, and steam generator blowdown systems extending from and including the secondary side of the steam generators up to and including the outermost containment isolation valves, and connected piping up to and including the first valve (including a safety or relief valve) that is either normally closed or capable of

automatic closure during all modes of normal reactor operation (Refer to Note (iv) below)

(6) Auxiliary saltwater, component cooling water, and auxiliary feedwater systems or portions of these systems that are required for emergency core cooling, post-accident containment heat removal, post-accident containment atmosphere cleanup, and residual heat removal

(7) Component cooling water system and seal water systems, or portions of these systems that are required to function to support other systems or components important to safety

(8) Those portions of systems (other than the radioactive waste management systems) that contain or may contain radioactive material and whose postulated failure could result in conservatively calculated potential offsite exposures in excess of 0.5 rem whole body (or its equivalent to parts of the body) at the site boundary or beyond

(9) Systems or portions of systems that are required to supply fuel for emergency equipment

(10) Systems or portions of systems that are required for (a) post-accident monitoring of Regulatory Guide 1.97, Revision. 3, May 1983 (Reference 11), Category 1 variables and (b) actuation of systems important to safety (Refer to Table 7.5-1)

(11) The protection system (Refer to Note (ii) below)

(12) The spent fuel storage pool structure, including the spent fuel racks

(13) The reactivity control systems that are required to achieve safe shutdown of the plant; i.e., control rods, control rod drivers, and boron injection system

(14) The control room, including its associated vital equipment and life support systems, and any structures or equipment inside or outside of the control room whose failure could result in an incapacitating injury to the operators

(15) Reactor containment structure, including penetrations (Refer to Note (iv) below)

(16) Systems or portions of systems that are required to provide heating, ventilating, and/or air conditioning for safety-related equipment/areas

(17) Portions of the onsite electric power system, including the onsite electric power sources, that provide the emergency electric power needed for functioning of SSCs included in Items (1) through (16) above

(18) Portions of the spent fuel pool cooling system used to remove spent fuel decay heat from the spent fuel pool, and portions of the refueling water purification system used to recirculate and cleanup the contents of the refueling water storage tank

Notes:

(i) A system boundary includes those portions of the system required to accomplish the specified safety function and connected piping up to and including the first valve (including a safety or relief valve) that is either normally closed or capable of automatic closure when the safety function is required.

(ii) For purposes of these criteria, the protection system encompasses all electrical and mechanical devices and circuitry (from sensors to actuation device input terminals) involved in generating those signals associated with the protective function. These signals include those that actuate reactor trip and, in the event of a serious reactor accident, that actuate engineered safety features (ESFs) such as containment isolation, safety injection, pressure reduction, and air cleaning.

(iii) SSCs that form interfaces between PG&E Design Class I and PG&E Design Class II or III SSCs are designed to PG&E Design Class I requirements.

(iv) Certain valves in these systems that are used for accident mitigation only, and do not support safe shutdown following an HE, are qualified for active function for an HE to provide increased conservatism in accordance with Reference 7. Refer to the active valve list, Table 3.9-9.

While some portions of the fire protection system components are designated PG&E Design Class I, the system is not required to ensure the integrity of the reactor coolant pressure boundary or to shut down the reactor and maintain it in a safe shutdown condition. Fire protection SSCs meet the requirements defined in Branch Technical Position (BTP) APCSB 9.5-1 (Reference 5) after 1979 (Refer to Section 9.5) and, where designated in the Q-list as PG&E Design Class I, are designed to withstand the effects of an HE.

3.2.2.1.2 PG&E Design Class II

PG&E Design Class II is applicable to SSCs that are important to reactor operation but not essential to safe shutdown and isolation of the reactor, and failure of which would not result in the release of substantial amounts of radioactivity.

Seismic qualification of certain PG&E Design Class II SSCs for the DE, the DDE and/or the HE is required to satisfy licensing basis commitments. These SSCs, along with their seismic qualification requirements, are identified in the Q-list (Reference 8). Refer to Section 3.2.2.4 for additional information.

(a) Those fluid systems and fluid system components that contain or may contain radioactive material, but whose failure would not result in calculated potential exposures in excess of 0.5 rem whole body (or its equivalent to parts of the body) at the site boundary. These fluid systems and fluid system components are designed to the accepted industry codes and standards in effect during the design and construction of DCPP.

(b) Power and auxiliary service piping systems (as defined in ANSI standard B31.1 , - Paragraph 100.1), which might otherwise be considered as PG&E Design Class III (Refer to Section 3.2.2.1.3), are classified as PG&E Design Class II (i.e., PG&E Design Class III is not used for power and auxiliary service piping systems).

3.2.2.1.3 PG&E Design Class III

PG&E Design Class III is applicable to SSCs that are not related to reactor operation or safety

Seismic qualification of certain PG&E Design Class III SSCs for the DE, the DDE and/or the HE is required to satisfy licensing basis commitments. These SSCs, along with their seismic qualification requirements, are identified in the Q-list (Reference 8). Refer to Section 3.2.2.4 for additional information.

3.2.2.2 PG&E Quality Assurance Classification

Appendix B to 10 CFR Part 50, Quality Assurance Criteria for Nuclear Power Plants requires that SSCs important to safety be designed and constructed in accordance with the quality assurance requirements and establishes such requirements for the design, manufacture, construction, and operation of these SSCs. 10 CFR Part 50, Appendix B, Criterion III, Design Control, states: “Measures shall be established to assure that applicable regulatory requirements and the design basis, as defined in 10 CFR 50.2 and as specified in the license application, for those SSCs to which this appendix applies are correctly translated into specifications, drawings, procedures, and instructions." DCPP Quality Assurance Program (QAP), as described in Chapter 17, satisfies the requirements of Appendix B to 10 CFR Part 50.

PG&E Design Class I SSCs are subject to the requirements of the QAP, in order to ensure that SSCs important to safety meet the requirements of 10 CFR Part 50, Appendix B. The QAP also includes several graded quality programs that apply to specific PG&E Design Class II and III SSCs.

The following are PG&E Quality Assurance (QA) Classes:

QA Class	Definition
Q	Equipment and structures to which the QA provisions of Appendix B to 10 CFR Part 50 apply for design, procurement, and construction.
"Blank"	PG&E Design Class II or III equipment that is not subject to nuclear quality assurance requirements.
R	Those radioactive waste management items that require application of graded quality assurance requirements.
G	Those portions of the fire protection systems and emergency lighting and communication equipment that require application of a quality program as described in Appendix A to NRC BTP APCSB 9.5-1 (Reference 15).
S	Equipment within the scope of the seismic configuration control program as defined in Inter-Department Administrative Procedure (IDAP) CF3.ID11. This PG&E Design Class II or III equipment requires seismic qualification to satisfy licensing or FSAR Update commitments, or to assure the functionality of PG&E Design Class I components. Refer to section 3.2.2.4 for additional information.
T	Regulatory Guide 1.97, Revision 3, May 1983 Category 2 and 3 instrumentation that requires application of the graded QAP as defined in IDAP CF3.ID12. (Note: Other Regulatory Guide 1.97 Category 2 and 3 instrumentation which is within the Environmental Qualification (EQ) Program, is part of the pressure boundary of a PG&E Design Class I System, or is a Class 1E electrical device, is PG&E QA Class Q.)

The DCPP Q-List (Refer to Reference 8), indicates the QA Class associated with each SSC, along with the industry codes and standards used for their design, fabrication, erection, and testing. In addition, specific requirements dictated by the quality standards applicable to the respective commercial code classes (i.e., ASME, ANSI, or ASA) are identified.

In addition to the graded QA programs described above, there are other QA programs that do not directly relate to the design, fabrication, procurement, construction, modifications or installation of DCPP SSCs.(e.g., Emergency Preparedness, Fitness for Duty, and Security (Refer to Program Directive OM5, Attachment 4)). Reference to these other programs is not included in the Q-List.

3.2.2.3 PG&E Quality/Code Class for Fluid Systems and Fluid Components

This section describes the PG&E Quality/Code classes that are used to implement quality standards that satisfy GDC 1, 1967, for DCPP fluid systems and fluid system components. The interrelationship between the PG&E Quality/Code class, the PG&E Design Class (Section 3.2.2.1), the PG&E Piping Symbol (Section 3.2.2.3.4), and applicable codes and standards, is illustrated in Table 3.2-2.

A comparison of the PG&E Quality/code classes to the recommendations of ANSI N18.2, Draft 1970, Safety Guide 26, March 1972, and Safety Guide 29, June 1972 is provided in Table 3.2-4. PG&E is not committed to the classification categories described in these documents.

3.2.2.3.1 PG&E Quality/Code Class I Fluid Systems and Fluid System Components

10 CFR 50.55a requires that certain components of the reactor coolant pressure boundary be designed, fabricated, erected, and tested (Inservice Inspection and Inservice Testing per 10 CFR 50.55a) in accordance with the requirements for Class A[a] components of Section III of the ASME Boiler and Pressure Vessel Code, or the most recently available industry codes and standards. PG&E Quality/Code Class I is applied to those components of the reactor coolant pressure boundary and implements the quality standards that satisfy the requirements of 10 CFR 50.55a. PG&E Quality/Code Class I components of the reactor coolant pressure boundary are listed in the DCPP Q-List, along with the industry codes and standards used for their design, fabrication, erection, and testing (Refer to Table 3.2-2).

The following provides a discussion of the PG&E Licensing Basis as compared to the other relevant regulatory and industry documents for comparison information only:

The PG&E Quality/Code Class I generally includes the fluid systems and components identified as Safety Class I in ANSI standard N18.2, Draft 1970, and Quality Group A in AEC Safety Guide 26, March 1972. However, the classification and quality standards

[a] The 1971 edition of the ASME Boiler and Pressure Vessel Code, Section III, Nuclear Power Plant Components, uses the term Class I in lieu of Class A.

for DCPP fluid systems and components were established prior to the existence of these documents and, therefore, do not always fall within their strict definitions. All PG&E Quality/Code Class I fluid systems and components are in accordance with the accepted industry codes and standards that were in effect during the design and construction of DCPP. If fluid systems and components were designed and constructed to codes and standards outside of the requirements of the above-mentioned documents, additional quality standards have normally been applied.

3.2.2.3.2 PG&E Quality/Code Class II Fluid Systems and Fluid System Components

Generally, PG&E Quality/Code Class II is applied to fluid systems and fluid system components that are:

(1) Part of the reactor coolant boundary, but excluded from PG&E Quality/Code Class I requirements by 10 CFR 50.55a

(2) Not part of the reactor coolant pressure boundary, but part of:

(a) Systems or portions of systems[(b)] that are required for emergency core cooling, postaccident containment heat removal, or postaccident containment atmosphere cleanup

(b) Systems or portions of systems that are required for reactor shutdown and residual heat removal

(c) Those portions of the main steam, feedwater, and steam generator blowdown systems extending from and including the secondary side of steam generators up to and including the outermost containment isolation valves, and connected piping up to and including the first valve (including a safety or relief valve) that are either normally closed or capable of automatic closure during all modes of normal reactor operation

(d) Systems or portions of systems that are connected to the reactor coolant pressure boundary and are not capable of being isolated from the boundary during all modes of normal reactor operation by two valves, each of which is either normally closed or capable of automatic closure

[(b)] The system boundary includes those portions of the system required to accomplish the specified safety function and connected piping up to and including the first valve (including a safety or relief valve) that is either normally closed or capable of automatic closure when the safety function is required.

PG&E Quality/Code Class II fluid systems and fluid system components are listed in the DCPP Q-List (Reference 8), along with the industry codes and standards used for their design, fabrication, erection, and testing (Refer to Table 3.2-2).

The following provides a discussion of the PG&E Licensing Basis as compared to the other relevant regulatory and industry documents for comparison information only:

The PG&E Quality/Code Class II generally includes the fluid systems and components identified as Safety Class 2a in ANSI standard N18.2, Draft 1970, and Quality Group B in AEC Safety Guide 26, March 1972. However, the classification and quality standards for DCPP fluid systems and components were established prior to the existence of these documents and, therefore, do not always fall within their strict definitions. All PG&E Quality/Code Class II fluid systems and components are in accordance with the accepted industry codes and standards that were in effect during the design and construction of DCPP. If fluid systems and components were designed and constructed to codes and standards outside of the requirements of the above-mentioned documents, additional quality standards have normally been applied.

3.2.2.3.3 PG&E Quality/Code Class III Fluid Systems and Fluid System Components

Generally, PG&E Quality/Code Class III is applied to fluid systems and fluid system components that are not part of the reactor coolant pressure boundary, nor included in PG&E Quality/Code Class II, but that are part of:

(1) Auxiliary saltwater, component cooling water, and auxiliary feedwater systems, or portions of these systems that are required for (a) emergency core cooling, (b) postaccident containment heat removal, (c) postaccident containment atmosphere cleanup, or (d) residual heat removal from the reactor

(2) Systems or portions of systems that are connected to the reactor coolant pressure boundary and are capable of being isolated from the boundary during all modes of normal reactor operation by two valves, each of which is either normally closed or capable of automatic closure

(3) Those portions of systems other than radioactive waste management systems that contain or may contain radioactive material, and whose postulated failure could result in conservatively calculated potential offsite exposures in excess of 0.5 rem whole body (or its equivalent to parts of the body) at the site boundary or beyond

(4) Component cooling water system and seal water systems, or portions of these systems, that are required to function to support other systems or components important to safety

(5) Portions of the spent fuel pool cooling system required for spent fuel cooling, and the refueling water purification system whose postulated failure could result in a loss of refueling water storage tank inventory

PG&E Quality/Code Class III fluid systems and fluid system components are listed in the DCPP Q-List (Reference 8), along with the industry codes and standards used for their design, fabrication, erection, and testing (Refer to Table 3.2-2).

The following provides a discussion of the PG&E Licensing Basis as compared to the other relevant regulatory and industry documents for comparison information only:

The PG&E Quality/Code Class III generally includes the fluid systems and components identified as Safety Classes 2b and 3 in ANSI standard N18.2 Draft 1970, and Quality Group C in AEC Safety Guide 26, March 1972. However, the classification and quality standards for DCPP fluid systems and components were established prior to the existence of these documents and, therefore, do not always fall within their strict definitions. All PG&E Quality/Code Class III fluid systems and components are in accordance with the accepted industry codes and standards that were in effect during the design and construction of DCPP. If fluid systems and components were designed and constructed to codes and standards outside of the requirements of the above-mentioned documents, additional quality standards have normally been applied.

An exception exists to the above for PG&E Quality/Code Class III, Piping Symbol D piping. These are systems or portions of systems that were originally constructed as PG&E Design Class II and were subsequently upgraded to PG&E Design Class I, usually because a later requirement was updated. For such piping, the design analysis is in accordance with PG&E Design Class I criteria. All construction, repair, or replacement performed after the upgrade is in accordance with PG&E QA Class Q requirements.

3.2.2.3.4 PG&E Piping Symbol System

A system of PG&E piping symbols are used on all piping schematics and drawings to indicate the fabrication, erection, seismic qualification requirements and test criteria applicable to specific portions of each piping system. The PG&E piping symbols, including their correlation with PG&E Design Class and PG&E Quality/Code Class, are listed in Table 3.2-2.

3.2.2.4 Additional Classifications

3.2.2.4.1 Seismic Classifications

GDC 2, 1967 requires that nuclear power plant SSCs essential to the prevention of accidents be designed to withstand the effects of earthquakes. The seismic design classification of SSCs is not explicitly defined for DCPP. Instead, the seismic design requirements for an SSC are determined based on the combination of the DCPP specific classification categories:

(1) PG&E Design Class (Section 3.2.2.1)

(2) PG&E Quality Assurance Classification (Section 3.2.2.2)

(3) PG&E Quality/Code Class of Fluid Systems and Fluid Components (Section 3.2.2.3)

(4) PG&E Piping Symbol (Section 3.2.2.3.4)

(5) PG&E Instrument System Classifications (Section 3.2.2.5)

(6) PG&E Electrical System Classifications (Section 3.2.2.6)

The DCPP Q-List (Reference 8) identifies the seismic qualification requirements for individual SSCs. For an SSC that requires seismic qualification, one or more of the following design basis earthquakes is included in the applicable load combinations:

(1) The Design Earthquake (DE), as described in Section 3.7.1.1

(2) The Double Design Earthquake (DDE), as described in Section 3.7.1.2

(3) The Hosgri Earthquake (HE), as described in Section 3.7.1.3

The acceptance criteria for an SSC that requires seismic qualification are based on maintaining structural/pressure boundary integrity and may also include the ability to perform an active function, depending on the design basis function of the SSC.

The following general guidelines apply to the seismic qualification requirements for an SSC, specific details and exceptions are provided in the Q-List:

(1) All SSCs designated as PG&E Design Class I are designed to remain functional when subjected to DE, DDE, and HE seismic loads.

(2) Most SSCs designated as PG&E Design Class II or III are not designed to remain functional when subjected to DE, DDE, or HE seismic loads.

(3) Based on specific licensing requirements, certain SSCs designated as Design Class II or III are designed to remain functional when subjected to DE, DDE, and/or HE seismic loads. These licensing requirements are addressed in Sections 3.2.2.1, 3.2.2.3 and Tables 3.2-1 and 3.2-2, as applicable, and specified in the Q-List.

PG&E Design Class II or III may have some seismically qualified components (as defined in the Q-list) and have no PG&E Quality/Code Class designation. However, some PG&E Design Class II components have been seismically designed; e.g., items in the Seismically Induced Systems Interaction Program, specific components required for post-HE shutdown, (e.g. CCW header C components), and items that were designed for the DE (for specific SSC classifications refer to the Q-list). For this reason, there is not a direct correlation between PG&E Design Class and the level of seismic qualification (except that PG&E Design Class I SSCs have seismically qualified components). In addition, the classification of seismically qualified components does not indicate which of the three design basis earthquakes the SSC has been qualified for, nor whether that qualification is for passive or active function. The design basis function of the equipment determines the type of seismic qualification required. These classifications and their relationships are summarized in Tables 3.2-1 and 3.2-2.

The definitions of the three DCPP Earthquakes are included below:

Design Earthquake

The design earthquake (0.2g) is defined as the maximum size earthquake that can be expected to occur at DCPP during the life of the reactor. The design earthquake is the equivalent of the operating basis earthquake, as described in 10 CFR 100, Appendix A.

Double Design Earthquake

The double design earthquake (0.4g) is defined as the hypothetical earthquake that would produce accelerations twice those of the design earthquake. The double design earthquake is the equivalent of the safe shutdown earthquake, as described in 10 CFR 100, Appendix A.

Hosgri Earthquake

The Hosgri earthquake (0.75g) is defined as the predicted ground motion at DCPP due to a Richter magnitude 7.5 earthquake on the offshore Hosgri fault. The Hosgri earthquake does not correspond to an operating basis earthquake or safe shutdown earthquake.

3.2.2.5 PG&E Instrument System Classifications

DCPP Instrumentation is classified as Instrument Class IA, IB, IC, ID, and II depending upon the function performed. Instrument Class IA, IB, IC and ID devices have PG&E Design Class I functions or other special requirements. Instrument Class II devices have PG&E Design Class II functions. Instrument classes are defined as follows:

(1) *Instrument Class IA* - Class IA instruments and controls are those that initiate and maintain safe shutdown of the reactor, mitigate the consequences of an accident, or prevent exceeding 10 CFR Part 100 off-

site dose limits. Class IA instruments and controls enable the PG&E Design Class I systems to automatically accomplish their appropriate safety functions, or they enable operating personnel to manually accomplish appropriate safety actions when a monitored condition reaches a preset level.

(2) *Instrument Class IB* - Class IB instruments and controls are those that are required for post-accident monitoring (PAM) of Category 1 and 2 variables in accordance with Regulatory Guide 1.97, Revision 3 (refer to Table 7.5-6). Regulatory Guide 1.97, Revision 3, Category 3 instruments required for PAM are Instrument Class II. PAM instrumentation enables operating personnel to:

(a) Determine when a condition monitored by PG&E Design Class I instruments and controls reaches a level requiring manual action

(b) Assess the accomplishment of plant safety functions

(c) Assess fission product barrier integrity

(d) Provide information to indicate the operability of individual systems important to safety

(e) Assess the magnitude of radioactive materials releases from the plant

PAM instruments are further divided into five variable types (A through E) as detailed in Regulatory Guide 1.97, Revision 3. These are further detailed in Section 7.5.

(3) *Instrument Class IC* - Instrument Class IC instruments and controls have the passive function of maintaining the pressure boundary integrity (PBI) of PG&E Design Class I piping systems. Passive valve operators that are within PG&E Design Class I piping systems are Instrument Class IC and are seismically analyzed to assure structural and pressure boundary integrity of the valve operator assembly. This classification also includes instruments installed in PG&E Design Class I HVAC ducting that are required to maintain pressure boundary integrity. In addition, this classification is used for instruments that are part of seismically qualified, PG&E Design Class II systems and denotes that the instruments are required to maintain their pressure boundary integrity.

(4) *Instrument Class ID* - Instrument Class ID instruments and controls are components that have certain PG&E Design Class I attributes, but do not require conformance with all Instrument Class IA, IB or IC requirements. The Instrument Class ID designation signifies that only certain design

requirements are imposed for a component. The specific requirements for the Instrument Class ID instruments are provided on a case-by-case basis in the DCPP Q-List (Reference 8).

Instrument Class ID components are defined as:

Active components that have certain Instrument Class I attributes and are relied upon to satisfy licensing commitments or safety analyses, but do not require conformance with all Instrument Class IA and IB requirements, or Components that are electrically, mechanically or pneumatically coupled with, and whose failure may have an adverse impact on the operation of equipment with an active safety function, but do not themselves have an active safety function. These components have certain Instrument Class I requirements imposed on them to preclude their failure.

(5) *Instrument Class II* - Instrument Class II instruments and controls have PG&E Design Class II functions. However, certain instrument Class II components are subjected to some graded quality assurance requirements. This classification also includes instruments fulfilling Regulatory Guide 1.97, Revision 3, Category 3 PAM functions (refer to Table 7.5-6).

Instrument Classifications IA, IB and IC are not mutually exclusive. More than one classification may apply to a single instrument device. Where such multiple choices exist, the highest classification is assigned to the device.

3.2.2.6 PG&E Electrical System Classifications

Class 1E: The electrical equipment and components supporting PG&E Design Class I functions are shown in Table 3.2-1.

Non-Class 1E: The electrical equipment and components supporting PG&E Design Class II SSCs functions are shown in Table 3.2-1.

3.2.2.7 Summary and Comparison of PG&E Classification Systems

Generally, codes and standards were applied for construction prior to issuance of the latest codes and standards; i.e. prior to the 1971 edition of the ASME Boiler and Pressure Vessel Code, Section III, Nuclear Power Plant Components. For these fluid systems and fluid system components, the design specifications specified the accepted industry codes and standards in effect during the design and construction of DCPP (Refer to Table 3.2-2).

Table 3.2-2 shows the relationship among the following: PG&E Design Class, PG&E Quality/Code Class, PG&E Piping Symbols, seismically qualified components, and the

Original Design/Construction Codes and Standards. Table 3.2-4 shows the relationship between DCPP classification categories and non-licensing basis regulatory and industry documents.

DCPP is committed to the PG&E Classification Categories (PG&E Design Class, PG&E Quality/Code, seismically qualified components, etc) as described in this section.

3.2.3 SAFETY EVALUATION

3.2.3.1 General Design Criterion 1, 1967 - Quality Standards

As discussed in Section 3.2.2, the DCPP classification system implements quality standards that satisfy the requirements of GDC 1, 1967.

3.2.3.2 General Design Criterion 2, 1967 - Performance Standards

As discussed in Sections 3.2.2.1 and 3.2.2.4, seismically qualified components at DCPP implement performance standards that satisfy the requirements of GDC 2, 1967, with respect to earthquakes.

3.2.3.3 10 CFR 50.55a - Codes and Standards

As discussed in Sections 3.2.2.2 and 3.2.2.3 the DCPP quality/code classification system includes codes and standards that satisfy the requirements of 10 CFR 50.55(a).

3.2.3.4 10 CFR Part 50 Appendix B - Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants

As discussed in Section 3.2.2.2, the DCPP Quality Assurance Program ensures that the quality/code classification category includes requirements that satisfy 10 CFR Part 50, Appendix B.

3.2.4 REFERENCES

1. Nuclear Safety Criteria for the Design of Stationary Pressurized Water Reactor Plants. N18.2 American Nuclear Society, August 1970 Draft.

2. Quality Group Classifications and Standards for Water, Steam, and Radioactive Waste Containing Components of Nuclear Power Plants, Safety Guide 26, March 1972, Atomic Energy Commission.

3. Seismic Design Classification, Safety Guide 29, June 1972, US Atomic Energy Commission.

4. Spent Fuel Storage Facility Design Basis, RG 1.13, Nuclear Regulatory Commission.

5. Guidelines for Fire Protection for Nuclear Power Plants, BTP APCSP 9.5-1, Nuclear Regulatory Commission.

6. Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants, Regulatory Guide 1.143, October 1979, Revision 1, Nuclear Regulatory Commission.

7. PG&E Letter to the NRC, DCL-92-198 (LER 1-92-015).

8. Classification of Structures, Systems, and Components for Diablo Canyon Power Plant Units 1 and 2 (Q-List), PG&E.

9. Letter from NRC (L. F. Miller) to PG&E (G. M. Rueger), dated December 13, 1993, Subject: NRC Inspection of Diablo Canyon Units 1 and 2 (Report No. 50-275, 50-323/93-31) [pages 1 and 2].

10. Letter from NRC (A. W. Beach) to PG&E (G. M. Rueger), dated August 15, 1994, Subject: NRC Inspection Report 50-275/94-18; 50-323/94-18 (Notice of Violation) [pages 14 and 15].

11. Criteria for Accident Monitoring Instrumentation for Nuclear Power Plants, Regulatory Guide 1.97, May 1983, Revision 3, Nuclear Regulatory Commission.

3.2.5 REFERENCE DRAWINGS

Figures representing controlled engineering drawings are incorporated by reference and are identified in Table 1.6-1. The contents of the drawings are controlled by DCPP procedures.

3.3 WIND AND TORNADO LOADINGS

All seismic Design Class I structures exposed to wind forces are designed to withstand the effects of the design wind, as required by GDC 2. Although a tornado design criterion was not required for the granting of construction permits, the tornado resisting capabilities of Design Class I structures and certain other structures have been reviewed.

3.3.1 WIND LOADINGS

The design wind specified has a velocity of 80 mph based on a recurrence interval of 100 years and is in accordance with Reference 8. Section 2.3 gives information on winds recorded at the site during the period of operation of the meteorological facility. A gust factor of 1.1 was applied to the design wind. The resulting dynamic pressures, q, for the design wind velocity distribution, developed in accordance with Reference 1, are given in Table 3.3-1.

3.3.1.1 Containment Structure

Wind loads for the containment structure were developed in accordance with Reference 1 for the design wind. Methods and results of force calculations are discussed in Section 3.8.

3.3.1.2 Other Buildings

Wind loads for buildings other than the containment structure and the turbine building were developed in accordance with the International Conference of Building Officials Uniform Building Code (UBC) - 1967 Edition. This code designates the site as being in a 20 pound per square foot (psf) zone. To provide added conservatism, it was assumed for design purposes that the site is in a 25 psf zone. This resulted in the following distribution of wind loading with height above ground (the ground - or the base for wind distribution - was conservatively assumed to be at sea level):

Height Above Sea Level, ft	Wind Load (On Flat Surfaces), psf
0-29	20
30-49	25
50-99	30
100-499	40

These wind pressures were considered to act upon the gross area of the vertical projection of each Design Class I structure measured above the average level of the adjoining ground. The roof of each Design Class I structure was designed for an uplift pressure equal to three fourths of the wind pressure for the vertical projection, assumed to act over the entire roof area.

A comparison of the UBC wind pressure values with those obtained for the design wind, using the recommendations of ASCE Transactions Paper Number 3269, Wind Forces on Structures (Reference 1), is shown for the auxiliary building in Table 3.3-1. Wind values using the ASCE paper are shown in the table column labeled “1.3q” (1.3 is the drag coefficient recommended by the ASCE paper). The top of the building is 105 feet above the ground surface. The comparison shows that the UBC wind pressures are greater than those recommended in the ASCE paper up to about 70 feet above the ground surface. At greater heights, the ASCE paper wind pressures are more than the UBC pressures. Design of the building considering both the UBC and ASCE paper wind pressures is discussed in Section 3.8.

Since the Design Class II turbine building could potentially be a hazard to adjacent Design Class I structures, the turbine building is designed to withstand a design wind velocity essentially the same as that used for Design Class I structures. Due to the turbine building’s unique size, shape, and configuration, UBC and ASCE Transaction Paper Number 3269 criteria are not used to derive wind pressures for this building. The magnitude and distribution of wind pressure loads on the turbine building are based on recommendations given in the United States Navy Design Manual DM-2 (Reference 10), except for the following conservatisms:

- The base for the vertical wind distribution is taken at sea level, rather than the ground elevation at the structure as specified in DM-2.

- A uniform wind pressure based on the average elevation of the roof is considered, rather than using the wind pressure variation with height specified in DM-2.

The turbine building design wind pressures, acting on the exterior surfaces of the building, are as follows:

Main Framing	
Windward side	25 psf
Leeward side	15 psf suction
Roof	20 psf suction
Girts and Purlins	
Windward side	35 psf
Leeward side	30 psf suction
Roof	35 psf suction

3.3.2 TORNADO LOADINGS

Although a tornado design criterion was not required as a condition of the granting of a construction permit, and despite the low probability of occurrence of tornadoes in California, a review of Design Class I and certain non-Design Class I structures was undertaken. The objective of the review was to establish capabilities of the Design Class I structures as designed and constructed to withstand tornado wind pressure and the associated atmospheric pressure drop and tornado-borne missile effects. Additionally, the consequences of tornado-induced failures on the ability to safely shut down the reactor, and/or limit radioactive releases to 10 CFR Part 100 guidelines, are discussed where appropriate.

Additionally, based on the following evaluations performed subsequent to the following review, PG&E has demonstrated that there is a low probability of tornadoes occurring at DCPP:

(1) The station blackout evaluation (Reference 11), using the methodology of NUMARC 87-00 (Reference 12), determined that the annual expectation of tornadoes of severity f2 or greater (i.e., wind speeds greater than or equal to 113 mph) in events per square mile for DCPP is equal to 1.0×10^{-7}. As a result, a tornado has not been considered an initiator for a station blackout. The NRC's acceptance of the station blackout evaluation is documented in Reference 13.

(2) The Individual Plant Examination for External Events (Reference 14) determined that the annual frequency of excessive tornado winds (i.e., wind speeds greater than or equal to 200 mph) was less than 3.2×10^{-7} per year. As a result, it was judges that a tornado wind-induced scenario is an insignificant contributor to overall core damage frequency and there are no plant vulnerabilities to high winds. The NRC's acceptance of the Individual Plant Examination for External Events evaluation is documented in Reference 15.

Although the reported tornado probabilities are significantly less than the threshold for a credible event, the evaluation of DCPP's tornado effects is still considered as part of DCPP's design and licensing basis.

3.3.2.1 Applicable Design Parameters

3.3.2.1.1 Tornado Winds

As previously discussed, there is no DCPP commitment to a specific design basis tornado windspeed. However, the NRC, in the evaluation of PG&E's tornado design criteria, used the methodology of WASH-1300, Technical Basis for Internal Regional Tornado Criteria, dated May 1974, to develop a conservative estimate of a tornado windspeed appropriate for DCPP. The NRC estimated that the design basis tornado windspeed (at a probability level of 10^{-7} per year) is about 200 miles per hour for the region at the plant site. The 200 mph tornado includes a 157 mph rotational component, a 43 mph translational component, and a differential pressure of 0.86 psi applied at a rate of 0.36 psi per second. Due to the low probability, the postulated tornado is unlikely to strike the site.

Based on the NRC's estimate, PG&E has used 200 mph as the tornado windspeed for determination of the acceptability of certain vital structures, systems, and components (SSCs) required for safe shutdown of the plant.

The general equation for tornado loads is:

$$W' = \left(C_p\, q_f + C_{pi}\, q_m\right) A + P_m \qquad (3.3\text{-}1)$$

where:

- A = area exposed to wind
- C_p = effective external pressure coefficient
- C_{pi} = internal pressure coefficient
- q_f = effective external velocity pressure
- q_m = effective velocity pressure for calculating internal pressure
- p_m = equivalent elastic missile impact loading

No gust factors or variation of wind velocity with height are assumed for tornadoes in evaluating overall response. However, in evaluating small structures (plan dimension less than 80 feet, and area less than 6400 square feet) or critical portions of large structures, an appropriate gust factor is used. Pressure coefficients are taken from Bechtel Corporation Topical Reports BC-TOP-3 and BC-TOP-3A (References 2 and 9). The internal pressure coefficients used in the review include the combined effects on internal pressures from both wind and atmospheric pressure drop. The magnitude and sign of the internal pressure coefficient selected depends on the ratio of open to solid area of the exterior walls, and on which of the following combinations is critical:

(1) Maximum wind load alone

(2) Maximum wind + one-half maximum pressure drop without building depressurization

(3) Maximum pressure drop with building depressurization

(4) Maximum wind + one-half maximum pressure recovery without building repressurization

These combinations conservatively represent the effects of the transient atmospheric pressure history. In cases (2) and (4), no building internal pressure equalization is assumed in combining maximum wind with pressure drop or pressure recovery. It is physically impossible for maximum atmospheric pressure drop to occur simultaneously with maximum wind.

The coefficients relating atmospheric pressure drop to velocity pressure are applied to the resultant (rotational plus translational) wind component. This procedure is conservative because only the rotational component causes an atmospheric pressure drop.

The maximum atmospheric pressure drop depends on the wind speed and relative structure size:

- For large structures, the pressure drop is 2.1q, where q is the velocity pressure, corresponding to a pressure range of 1.2 psi to 3.4 psi for tornado wind speeds of 175 mph to 300 mph.

- For small structures and local portions of large structures, the pressure drop is 3.4q, corresponding to a pressure range of 1.9 psi to 5.4 psi for tornado wind speeds of 175 mph to 300 mph.

Velocity pressures (q) are converted to wind speeds (V) using the formula:

$$q = 0.00256\ V^2 \qquad (3.3\text{-}2)$$

where:

q = velocity pressure (psf)
V = wind speed (mph)

3.3.2.1.2 Tornado Missiles

Hypothetical tornado borne missiles considered are:

(1) 108 pound, 4- x 12-inch x 10 feet board at tornado wind velocity

(2) 76-pound, 3-inch x 10 feet schedule 40 pipe at one-third tornado wind velocity

(3) 4000 pound auto up to 25 feet above ground - frontal area 20 square feet at one-sixth tornado wind velocity

Only one missile is assumed to act at a time. Additional site-specific tornado missiles, such as siding and pull box or hatch covers, are evaluated to determine if they are more severe than the three listed hypothetical missiles. In cases where they were found to be more severe, the site-specific missiles have been included in the determination of the tornado resisting capabilities of SSCs. The maximum velocities for the site-specific missiles are determined using the methods described in Reference 16. If a site-specific missile associated with a pull box or hatch cover is found to result in the calculated tornado resisting capability of exposed SSCs falling below 200 mph, the cover is anchored to prevent it from becoming airborne at wind speeds up to 200 mph. However, capabilities of less than 200 mph for certain SSCs have been found acceptable, based on the tornado analysis described in Section 3.3.2.3 and need not be evaluated for higher wind speeds. These SSCs are identified in Table 3.3-2.

3.3.2.2 Determination of Forces on Structures

3.3.2.2.1 Tornado Wind Forces

The basic technical references employed in the tornado review are Topical Reports BC-TOP-3 and BC-TOP-3A.

The safe wind force capability is conservatively determined by deriving the uniform static load applied over the full height of the structure's projected area combined with atmospheric pressure drop at which induced stress in the critical element equals 0.9 F_y in bending and tension and 0.5 F_y in shear in the case of steel, or ACI 318-71 stress levels in the case of concrete structures. For the combined effect of wind and tornado-induced missiles, 1.0 F_y in bending and tension and 0.6 F_y in shear are used. Additionally, overall stability is checked by comparing the overturning moment of the wind forces to the resisting moment due to dead load only. Live and dead loads are considered to act simultaneously with tornado loadings. A load factor of unity is used in this load combination and is justified by the short duration and low probability of tornado loads.

3.3.2.2.2 Forces Derived From Tornado Missile Loading

Guidelines for this determination include the derived capability of structures or components for wind loads and an evaluation of the weight, weight to maximum cross-sectional area, and weight to minimum impact area of the potential missile. Tornado missiles produce impulsive load effects that are converted to equivalent elastic loads to be combined with wind loads only at locations where exterior walls protect interior Design Class I components. Missile penetration capabilities are evaluated using the modified Petry formula for concrete, and the Ballistic Research Laboratories and Stanford formula for steel. For concrete structures, the missile impact load is converted to an equivalent elastic load, P_m, using the following procedures:

(1) Estimate average impact force:

$$F_i = \frac{WV^2}{2gX} \qquad (3.3\text{-}3)$$

where:

W = missile weight
V = missile impact velocity
X = impact penetration as determined by modified Petry formula
g = acceleration of gravity

(2) Calculate duration of impact force:

$$t_i = \frac{2X}{V} \qquad (3.3\text{-}4)$$

(3) Determine dynamic load factor (DLF) versus the ratio of t_i/T, where T is the target period and a rectangular impulse is assumed

(4) Calculate equivalent static load as the product of impact force, F_i, and the DLF

(5) Calculate equivalent elastic load P_m:

Pm = Fi x DLF/10 for bending (3.3-5)

Pm = Fi x DLF/7.5 for shear (3.3-6)

The elastic indices of 10 for bending and 7.5 for shear are based on tests (3) that indicate that when using an elastic impulsive analysis, very high fictitious elastic stress will result. These tests indicate that the fictitious stresses corresponding to an elastic impulsive analysis are generally more than 10 times the usual static ultimate values.

(6) Calculate the ultimate buckling load of the missile, P_b. If P_m is less than P_b, use P_m. If P_m is greater than P_b, use P_b in evaluating structural capability.

For steel structures, missile impact loads are converted to equivalent static loads and structural response is evaluated using conventional methods, such as conservation of momentum and energy balance techniques, with due consideration for type of impact (elastic or plastic).

3.3.2.2.3 Piping Analysis

Design Class I piping in locations exposed to the weather is evaluated against the following tornado-related effects: (1) 200 mph tornado wind loading, and (2) impact by tornado-induced missiles. Primary stresses associated with the first loading is evaluated on limiting pipe spans between supports against 120 percent of the normal allowable stress, as specified by the ANSI B31.1 piping code. Active piping supports (i.e., seismic snubbers) are assumed to resist these first two effects. The second effect, tornado-induced missile impact, is evaluated via the methods described in ANSI N-177 (Reference 4). This evaluation considers the impact interaction energy to determine: (1) crushing of the pipe wall, (2) crushing of the missile, and (3) penetration of the pipe wall. The first two effects are evaluated against material yield strength, and the penetration effect is evaluated via the Ballistic Research Laboratory formula as described in “Design of Structures for Missile Impact,” Topical Report BC-TOP-9A, Revision 2, Bechtel Power Corporation, September 1974 (Reference 17).

3.3.2.2.4 Tornado Resisting Capability

Table 3.3-2 lists tornado resisting capabilities in terms of safewind velocities for Design Class I structures and Design Class II structures containing Design Class I components. In all cases, the safewind velocity includes wind pressure and associated atmospheric pressure drop effects, and is based on the element of the structure with the minimum capability. For almost all cases, the capability to resist the combined effects of a tornado and the most severe of the tornado-induced missiles is also given in terms of safewind velocity in Table 3.3-2. An expanded list of tornado-borne missiles has also been evaluated for maximum velocity developed and penetration and spalling of reinforced concrete missile barriers. This supplementary analysis is discussed in Section 3.3.2.4.

3.3.2.3 Tornado Analysis

The consequences of failure of items in Table 3.3-2 with low or intermediate resistance to tornadoes and tornado-induced missiles have been analyzed. The purpose of this analysis is to determine whether such failure would compromise the capability of shutting the plant down safely.

In this determination, it is conservatively assumed that both units are operating at full power when the tornado strikes, with no prior initiation of shutdown procedures. The tornado is generally assumed to first cause complete loss of offsite power and a turbine trip, as described in Sections 15.2.9 and 8.3 (Tables 8.3-2 and 8.3-3). The plant is assumed to proceed to hot shutdown and then to cold shutdown (if necessary), in accordance with procedures for loss of offsite power operation. The effects of failure of items in Table 3.3-2 have been analyzed for two cases: (a) following loss of offsite power, and (b) as the initiating event for plant shutdown.

Recorded tornadoes in California, and in the continental US in general, have traveled from the south or southwest to the north or northeast. In the analysis of consequences, this direction is postulated as the tornado path; and credit is taken qualitatively for protection afforded to external equipment or exposed components by structures in the postulated path. In addition, the postulated flight path of a site-specific missile, between its installed location and potential targets, is considered in the determination of the vulnerability of external equipment or exposed components.

3.3.2.3.1 Major Findings of the Tornado Analysis

The following are the major findings of the tornado analysis:

(1) The maximum postulated tornado wind velocity (300 mph) will not cause a LOCA or structural damage impairing containment integrity.

(2) All Design Class I structures, or structures housing Design Class I components, are capable of withstanding the wind effects of at least a 225 mph tornado without failure of major structural elements. Their resistance to the tornado-induced missiles corresponds to at least a 150 mph wind velocity with the exception of the fuel handling area steel structure, which has a capability of 127 mph wind velocity. Structures are arranged on the plant site and protected in such a manner that the collapse of structures that do not have tornado-resisting capability will not affect those that must withstand the tornado effects.

(3) At calculated safewind velocities of equipment and structures, certain missiles more severe than the three hypothetical missiles are generated, but these site-specific missiles have been included in the determination of the safe-wind velocities. However, in order to ensure that the calculated safe-wind velocity for certain exposed safe shutdown SSCs (e.g., the containment pipeway structure, the exposed portions of main steam leads 1 and 2, and the exposed portions of the feedwater piping) is greater than or equal to 200 mph, certain hatch and pull box covers have been anchored to prevent their becoming tornado-induced missiles.

(4) Loss of the following Design Class I equipment, with low or intermediate tornado resistance, will not compromise the capability of shutting down the plant safely. Such loss does not produce radioactive releases greater than those resulting from loss of offsite power operation. Loss of offsite power is a limiting Condition II fault discussed in Section 15.2.9:

(a) Outdoor tanks

(b) Plant vent

(c) Control room ventilation system

(d) Auxiliary building and fuel handling area ventilation systems

(e) Miscellaneous instrumentation controls and instrument conduits associated with steam and feedwater lines of steam generators 1 and 2

(f) Piping and instrumentation on the CCWS surge tank.

(g) Auxiliary feedwater system piping, valves and instrumentation in the FE and FW areas

(h) 480-V switchgear and 125-Vdc inverter room ventilation system

(i) Containment penetrations

(j) 4.16-kV switchgear/cable spreading room HVAC system

(k) 10 percent steam dump valves (PCV-19 to PCV-22)

(5) The externally located Design Class I relief valves on the main steam system are susceptible to damage in the highly unlikely event of a direct impact from one of the tornado-induced missiles. The failure of a single relief valve is a Condition II fault and will result in a safety injection signal (Section 15.2.14).

(6) The following components of the emergency power system were evaluated for tornado-induced damage:

(a) Diesel generators

(b) 4.16-kV vital switchgear and cable spreading rooms.

Such damage would not be expected to affect more than one diesel generator or more than one vital 4.16-kV bus. This degree of damage will not affect the shutdown capability of the plant, even if all offsite power is lost. However, tornado missile barriers have been provided to preclude tornado-induced damage to the diesel generators for missiles associated with a 200 mph tornado.

Additional girts have been added to the turbine building steel framing at the 4.16-kV switchgear and cable spreading rooms to prevent the exterior siding from becoming a missile during a 200 mph tornado. The analysis method used is described in Reference 9. The turbine building framing modifications are shown in Figures 3.3-1 and 3.3-2.

3.3.2.3.2 Detailed Results of the Tornado Analysis

3.3.2.3.2.1 Auxiliary Building

The major portion of the auxiliary building, except for doors and louvers, is capable of withstanding a 300 mph tornado. The rooms housing the battery room ventilation supply and exhaust fans at elevation 163 ft-6 in. are capable of withstanding a 240 mph tornado. All Design Class I equipment within the building is behind concrete walls or within concrete enclosures. The component cooling water surge tank and the 480-V switchgear/125-Vdc inverter room ventilation system located on the roof of the auxiliary building are evaluated for tornado effects in Sections 3.3.2.3.2.2 and 3.3.2.3.2.13, respectively.

Venting to limit effects of atmospheric pressure drop was not included in the design of the auxiliary building, nor assumed in the subsequent analysis of the structure. With the assumption of no venting and the selection of corresponding internal pressure coefficients, the building is conservatively shown to be capable of sustaining at least a 3.4 psi pressure drop. If venting through openings and louvers were assumed, the capability would exceed 300 mph.

The auxiliary building houses equipment for three safety-related ventilation systems. Air intake or exhaust louvers for these systems have limited tornado-resisting capability. The safety-related ventilation systems within the auxiliary building potentially affected by tornado are: (a) the main auxiliary building ventilation system (see Section 9.4.2), (b) the control room ventilation system (see Section 9.4.1), and (c) the fuel handling building ventilation system (see Section 9.4.4). Section 3.3.2.3.2.3 has additional information on fuel handling building ventilation system tornado capabilities. The consequences of tornado damage affecting these ventilation systems are discussed below.

The main auxiliary building ventilation equipment that is potentially vulnerable to tornado damage is located in the auxiliary building at elevation 140 ft. Air intake openings and equipment access panels create potential tornado vulnerability for this area, although the containment, turbine building, and fuel handling building structures provide protection from tornado missiles. Separation of the redundant auxiliary building ventilation supply fans ensures that cooling air to safe shutdown equipment would be available should a tornado generated missile damage one supply fan. Damage to the ventilation control components could cause inlet vane dampers to fail open, but supply fans would continue to run. Although intake louvers will yield and could be forced inward at tornado wind speeds, blockage of the auxiliary building intake openings is considered highly unlikely.

Control room ventilation equipment is located in the auxiliary building at elevation 154 ft-6 in. in a mechanical equipment room. Rooms to the east of the mechanical equipment room contain condenser units required for control room cooling, but not required for ventilation.

Three types of louvers are part of the control room HVAC (see Figure 9.4-1): (a) inlet louvers tied by ductwork to dampers and inlet filter banks, (b) exhaust louvers tied by ductwork to dampers, and (c) inlet and outlet louvers associated with the aircooled condensers.

At a wind speed of about 100 mph, the louver yields in bending, or the louver attachment to the louver frame yields. A deformed louver assembly could be blown inward at a higher wind speed and could damage related components, such as filters and dampers, or buckle attached ductwork. The redundant supply fans are not considered vulnerable to such damage because of their physical location in relation to the inlet and exhaust louvers, and because of the number of components (dampers, filters, etc.) in the ductwork between the louvers and the fans. Complete blockage of the ducts by the damaged louvers is highly unlikely. As a result, the control room HVAC is most likely to continue operation in Mode 1 following a tornado.

Without a simultaneous control room fire or accident, operation of other components is not required to maintain adequate temperature conditions for personnel or instrumentation in the control room during post-tornado shutdown. The redundant aircooled condensers are not considered vulnerable to damage from being hit by displaced louvers. A condenser could be damaged by a tornado-induced missile penetrating through the louvers.

Multiple missiles would be required to render inoperative more than one of the four condensers serving the control room HVAC systems of the two units. Even if all inlet ducts were blocked by damage or by loss of air to the pneumatic dampers, and if all aircooled condensers were rendered inoperative, operators would take appropriate action based on plant conditions.

Fuel handling building ventilation equipment located in Area L of the auxiliary building at elevation 100 ft is potentially vulnerable to tornado damage because of supply fan air intake openings.

These air intake openings are fully shielded to the south (Unit 1) and north (Unit 2) by the containment structures. Significant shielding is afforded these air intakes by the pipeway structure and turbine building to the west. The fuel handling building supply fans are located in a compartment beneath the air intakes and are not vulnerable to the effects of a tornado missile. In the unlikely event that a missile were to enter the fuel handling building ventilation equipment through the air intake, supply fan discharge ducting and dampers could be impacted. While damage to the ducting could occur that could degrade system performance, the supply fans would still be expected to provide adequate air flow to the fuel handling building vital equipment.

The battery room ventilation equipment in the fan rooms on the auxiliary building roof on elevation 163 ft-4 in. is nonsafety-related and damage to this equipment due to a tornado does not affect the safe shutdown capability of the plant.

3.3.2.3.2.2 Component Cooling Water System Surge Tank

The surge tank of the CCWS (Section 9.2.2) is supported horizontally at the L-line on the roof of the auxiliary building (see Figures 1.2-21 and 1.2-25). A baffle at the tank's mid-length extends about 40 percent up the tank volume to divide it into two compartments. Each compartment has its own liquid level instrumentation (see Figure 3.2-14) and a 6-inch surge line to one of the two independent vital trains of the CCWS. Each surge line is fed by a makeup line with an air-operated level control valve.

This connection is made inside the auxiliary building. The high-level alarm actuates above the top of the tank divider.

The pressurized tank vent discharges through a normally closed back-pressure control valve onto the roof of the auxiliary building. The roof is surrounded by a parapet and sloped toward a series of 4-inch roof drains. These drains feed into common 6-inch drain lines to the storm sewer and then to the discharge structure.

In case of radioactivity in the CCWS, the vent valve closes. Any overflow from the CCWS discharges through a relief valve at the top of the tank into a drain line to the auxiliary building sump. The foundation under the surge tank is sloped toward a 4-inch drain which also drains to the auxiliary building sump. The tank has an aluminum skirt from its horizontal diameter to the foundation to prevent rain water from flowing into the auxiliary building sump.

As shown in Table 3.3-2, the tank is capable of withstanding a 250 mph wind velocity, or a 200 mph wind plus associated tornado-induced missiles. The weakest element of the tank structure is the anchorage, which fails in bearing on concrete and results in only minor displacement of the tank. It is highly unlikely that such displacement would result in either rupture of the relief valve header or surge lines.

The surge lines run from beneath the surge tank, along the auxiliary building roof, and down the outside wall to elevation 140 feet-4 inches, where they enter the auxiliary building. These lines are exposed to tornado-induced missiles after they exit from beneath the surge tank until they enter the auxiliary building. The relief valve header, the vent and valve, and the two sets of liquid level instrumentation are on the east side of the surge tank and are susceptible to damage by tornado-induced missiles. Tubing, valves, and instrumentation added for surge tank nitrogen/air pressurization are also susceptible to damage by tornado-induced missiles. The tank does provide considerable protection in the postulated path of the tornado, particularly to instrumentation taps located below the operating liquid level of the tank. An exposed raceway carries the common vital conduit from the surge tank to the control room.

Failures of the raceway, the liquid level instrumentation, the nitrogen/air pressurization components, and pipes and valves are analyzed in Table 3.3-3. This analysis demonstrates that the CCWS would continue to operate satisfactorily even with postulated tornado damage. The CCWS surge tank is readily accessible from the control room, so immediate assessment of any damage following a tornado is possible.

Release of radioactivity to the discharge structure can result only if the operating portion of the CCWS is radioactively contaminated and if an instrumentation line breaks below the normal liquid level in the surge tank at a point not close to the tank.

Release of water to the auxiliary building sump or 115 ft yard drains could result if a surge line were ruptured by a tornado missile outside of the auxiliary building. Failure of the surge lines are analyzed in Table 3.3-3.

3.3.2.3.2.3 Fuel Handling Area

Analysis of the basic structure of the fuel handling area, with partial loss of siding, provides a conservative estimate of capability to withstand a wind velocity in excess of 250 mph. Under combined wind and missile load, the worst case analysis, based on elastic behavior, shows resistance to at least a 127 mph wind velocity. Because of the conservatism inherent in the assumptions for these calculations, the resistance to tornado-induced missiles is believed to be considerably higher.

Purlins, girts, siding, roofing, doors, and louvers are damaged at lower wind velocities. They are not essential to the overall structural integrity of the fuel handling area and do not produce missiles more severe than the hypothetical missiles. The metal siding and roofing do not provide significant missile protection for the building contents.

Analysis of potential water loss from the spent fuel pool shows that the water cover remaining over the fuel provides adequate protection against both fuel damage and pool liner penetration from tornado missiles. Water lost from the spent fuel pool can be replenished from on-site water supplies. Therefore, the capability of the fuel handling building ventilation system to maintain a negative pressure in the fuel handling area is not required after a tornado.

The fuel handling building ventilation exhaust ducts and fuel handling area radiation monitors have limited tornado resisting capability. Failure of these components does not affect safe shutdown capability and does not result in significant radiation releases since damage to the spent fuel does not occur as a result of a tornado.

3.3.2.3.2.4 Containment Structure

The containment structure, including equipment, personnel, and escape hatches, is capable of withstanding the combined effect of a 270 mph wind velocity and tornado-induced missiles. The containment pipeway structure is capable of withstanding the combined effect of a 200-mph wind velocity and tornado-induced missiles.

Containment pipe penetrations for main steam and feedwater lines for steam generators 1 and 2 and some non-vital and spare electrical penetrations are located within the pipeway structure. No other containment penetrations are exposed to the effects of a tornado. As described in Section 3.3.2.3.2.7, the main steam and feedwater piping have capability to withstand the combined effect of a 200-mph wind velocity and a missile. The non-vital circuits within the electrical penetrations are not required for safe shutdown of the plant. The electrical penetrations for both units are significantly shielded by the turbine building, containment structure, and the auxiliary building. In addition, the pipeway structure affords additional local protection for these penetrations. Therefore, tornado induced damage to these penetrations is considered extremely unlikely and would not affect safe shutdown capability.

Note that in order to maintain a minimum capability of 200 mph for these SSCs, certain pullbox and hatch covers have been anchored in order to prevent them from becoming tornado-induced missiles.

3.3.2.3.2.5 Plant Vent

The containment exhaust vent (plant vent) is anchored securely to the containment and can withstand loads from a wind velocity of greater than 300 mph. The internal framing supporting the duct will yield at a wind velocity of about 125 mph, but the duct remains functional at higher wind velocities. The tornado-induced missiles would damage and penetrate the vent. The plant vent handles exhaust from the auxiliary building, fuel handling areas, and penetration area ventilation fans, steam air ejector and gland steam condenser, containment purge system, and gas decay system. The plant vent contains sample probes for the plant vent gas and particulate radiation monitor. The containment purge, steam ejector line, and the gas decay system have radiation monitors prior to discharge to the plant vent.

The fuel handling area and penetration area ventilation systems do not need to be operated while the plant is brought to a safe shutdown condition. The damage to the plant vent will not prevent the auxiliary building ventilation system from exhausting. The containment purge system is normally operated only during extended plant shutdowns, and operation during safe shutdown procedures is not necessary. The remaining process flows exhausted through the plant vent are not necessary for and are unrelated to attaining safe shutdown capability. It is therefore concluded that the plant can be safely shut down without additional radiation exposure to the public, even if the plant vent and its radiation monitoring capabilities are damaged by a tornado.

3.3.2.3.2.6 Design Class I Raceways and Instrumentation

Exposed Design Class I raceways are attached to the support structure for the main steam and feedwater lines outside the containment (see Figure 3.3-3). These raceways carry control wiring to actuate various mechanical components of the steam, feedwater, and auxiliary feedwater systems. These items are shadowed from two directions as indicated in Figure 3.3-3. Some of this equipment is located in exposed instrumentation panels on the outside of the containment.

A failure analysis has been performed on these raceways and panels. Two cases were examined: (a) failure with the plant operating normally, and (b) failure with the plant already being shut down due to loss of all offsite power caused by the tornado. The results of this analysis, given in Table 3.3-4, indicate that the plant can be shut down safely.

Main steam pressure transmitters, main feedwater flow transmitters, and auxiliary feedwater flow transmitters associated with steam generators 1 and 2 are located within mechanical panels at elevation 85 ft beneath the pipeway structure. The equivalent transmitters for steam generators 3 and 4 are within the auxiliary building structure and are not vulnerable to tornado damage. In addition, main steam line radiation monitors are located adjacent to main steam lines 1 and 2 near the containment pipe penetrations.

The steam generator pressure transmitters (PT-514 to PT-516 & PT-524 to PT-526) provide a safety injection and main steam isolation signal on low steam generator pressure using two-out-of-three logic. A safety injection signal will be actuated only upon loss of at least two of the three redundant pressure transmitters associated with each steam lead. The safety injection signal can be classed as spurious and is a

Condition II fault discussed in Section 15.2.15. Steam generator pressure indicators for steam generators 3 and 4 are not affected by the tornado, and these steam generators would be used to attain safe shutdown.

The main feedwater system would receive isolation signals as a result of the safety injection signal being produced. With the main feedwater isolation valves closed and the FW pumps tripped, the main feedwater flow transmitters (FT-510, FT-511, FT-520 and FT-521) do not provide relevant information. These transmitters, while conservatively designated as Instrument Class 1A, do not have any safety-related functions.

The auxiliary feedwater flow transmitters (FT-50 and FT-77) provide control room indication and do not have any protective functions. If these transmitters are lost, steam generator level (especially wide range steam generator level, which is recorded) would provide an assessment of the status of the auxiliary feedwater system to steam generators 1 and 2. If steam generator 1 and/or 2 level cannot be controlled due to damage to auxiliary feedwater system components in the pipeway structure, auxiliary feedwater to steam generator 1 and/or 2 can be secured and the plant can be safely shutdown using steam generators 3 and 4.

The main steam line radiation monitors are provided to detect primary-to-secondary radiation releases resulting from a steam generator tube rupture accident. Such an accident condition is not assumed to occur concurrently with extreme weather conditions. Therefore, tornado-induced failure of these instruments is of no consequence and does not affect safe shutdown capability.

A seismic trip sensor located in the pipeway area is one of three sensors that generate a reactor trip signal if seismic accelerations exceed a predetermined level using two-out-of-three logic. Loss of power to these sensors will not result in a trip signal. Damage to this instrument and its resultant output signal is of no consequence and does not affect safe shutdown capability.

3.3.2.3.2.7 External Design Class I Piping and Valves

Main steam leads 1 and 2, and associated feedwater piping, penetrate the containment at elevation 129 feet. This piping is carried to the turbine building at elevation 112 feet-6 inches on a support structure external to the containment (see Figures 1.2-5 and 1.2-21 for Unit 1, and Figures 1.2-10 and 1.2-28 for Unit 2). This piping and associated steam relief valves are Design Class I up to and including the first isolation valves outside the containment. On main steam leads 1 and 2, the relief valves are located directly on the piping. On main steam leads 3 and 4, which are enclosed within the auxiliary building, the relief valves are located on separate headers on the roof of the enclosure at elevation 143 feet (see Figures 1.2-4 and 1.2-21).

Analysis of the main steam and main feedwater piping for wind stress indicates a negligible stress increase, even for the 200-mph wind velocity. The capability of this piping to withstand combined tornado wind and tornado-induced missile impact is limited to 200 mph. At this wind velocity, the piping is capable of supporting all loadings associated with the most severe tornado-induced missile without penetration of the pipe wall. In order to maintain this capability, certain pullbox and hatch covers have been anchored to prevent them from becoming tornado-induced missiles.

The 4-inch main steam piping to the AFW pump turbine is vulnerable to damage from a tornado missile. A break in this piping would be considered a minor secondary pipe break, which is a Condition III fault discussed in Section 15.3.2. A breach in this 4-inch main steam piping could cause a loss of the AFW pump 1 turbine. The motor-driven AFW pump 3 supplying steam generators 3 and 4 would still be available for plant shutdown. In addition, AFW pump 2 would be available, although there is a potential for wind induced damage to its level control valves as discussed below.

The steam generator safety relief valve spindles are considered vulnerable to direct impact by a missile. Such impact could result in severance of the spindle. However, this is not expected to result in a valve discharge or pressure boundary leakage. The relief valves represent extremely small targets for a missile and a direct hit is considered extremely unlikely. The relief valve headers for leads 3 and 4 of both units are almost completely protected by the containment and auxiliary building concrete structures, particularly from the south and southwest. The containment for Unit 1 also provides considerable protection from those directions to the relief valves on leads 1 and 2 for that unit.

The single failure of a steam generator safety relief valve is a Condition II fault (Section 15.2.14). The occurrence causes a safety injection signal.

The mechanical portions of other externally located valves associated with leads 1 and 2 are not considered susceptible to tornado or tornado missile damage. The following other events related to these valves are considered: (a) loss of air supply to main steam isolation valve, (b) rupture of the bypass line around a main steam isolation valve, (c) failure of the motor operator on a main feedwater isolation valve, and (d) loss of air supply to main feedwater control and bypass valves.

The main steam isolation valve is held open by compressed air. The valve has an integral pneumatic supply to hold the valve open (or to open it one time, if closed), upon loss of the main air supply. Such loss would, therefore, have no consequences. If the integral pneumatic supply is damaged, the valve is driven shut by an internally mounted spring. The inadvertent closing of a main steam isolation valve is similar to the inadvertent closing of a turbine stop valve, a Condition II fault whose consequences are discussed in Section 15.2-7.

The bypass around a main steam isolation valve is a 3-inch line. Its rupture by missile impact qualifies as a minor secondary system pipe break, as analyzed in Section 15.3.2. The consequences of such a break are considerably less than those of a 6-inch-diameter break, which is the limiting case in that section and is equivalent to the inadvertent opening of a steam relief valve.

Failure of the motor operator on a main feedwater isolation valve causes the valve to remain as is. The valve is normally open during plant operation. A Design Class I feedwater control/bypass valve and a Design Class I check valve are installed upstream of each main feedwater isolation valve. These valves provide additional means of isolating the feedwater line and provide the pressure boundary for operation of the auxiliary feedwater system if the isolation valve were to fail open. Therefore damage to the isolation valve operator or a loss of power that would cause it to fail "as-is" would have no affect on the integrity of the feedwater line or operation of the auxiliary feedwater system.

The air operators on the main feedwater control and bypass valves fail in the closed position upon loss of air or loss of dc power to the trip solenoid. The isolation valve and check valve installed downstream of each control/bypass valve provide additional means of isolating each feedwater line. A malfunction of the feedwater control or bypass valves, such as accidental full opening, is a highly unlikely consequence of tornado damage since these valves fail closed. However, this type of malfunction (a Condition II fault) is discussed in Section 15.2.10.

The auxiliary feedwater (AFW) system piping and valves located in the FE and FW areas, at approximately 130 ft elevation (supply to SGs 1 and 2), were identified as potentially vulnerable to tornado wind and induced missile effects. This piping has been analyzed to withstand over 300 mph wind velocity. Hence, it is very unlikely that these components will be damaged due to tornado winds. However, the AFW piping and valves in these areas are still susceptible to tornado-induced missile damage. On the basis that only one missile will occur at a time, such damage is limited to only one train of the AFW system. Thus, there will be no complete loss of AFW system function because redundant trains are available.

The following events related to AFW valves have been considered: (a) failure of the motor operator on an auxiliary feedwater control valve, (b) failure of the electro-hydraulic auxiliary feedwater level control valves, and (c) failure of the motor-operated steam supply valve to the turbine-driven auxiliary feedwater pump.

The motor-operated AFW level control valves (LCV-106 and LCV-107) for the turbine-driven AFW pump are normally open. These valves are used for remote manual throttling to maintain an appropriate steam generator level if the turbine-driven AFW pump is utilized. In case of loss of power to these valves, the turbine-driven AFW pump can be secured if necessary and secondary system decay heat removal can be accomplished using steam generators 3 and 4 and motor-driven AFW pump 3. None of these components are vulnerable to tornado damage as they are contained within the auxiliary building structure.

The electro-hydraulic AFW level control valves (LCV-110 and LCV-111) for the motor-driven pump that supplies AFW to steam generators 1 and 2 fail in the open position on loss of power. These valves are used for remote manual throttling to maintain an appropriate steam generator level if the motor-driven AFW pump to steam generators 1 and 2 is utilized. In case of loss of power to these valves, the motor-driven AFW pump can be secured if necessary and secondary system decay heat removal can be accomplished using steam generators 3 and 4 and motor-driven AFW pump 3. None of these components are vulnerable to tornado damage since they are contained within the auxiliary building structure.

Loss of remote manual control to the above noted AFW level control valves may result in some excess in feedwater injection from the AFW system. A much greater excess in feedwater injection, such as results from inadvertent full opening of a main feedwater supply valve, is a Condition II fault discussed in Section 15.2.10.

Failure of the motor operator on the steam supply valves to the turbine-driven AFW pump (FCV-37) causes the normally open valve to remain in the as-is position. The valve is not required to change position for operation of the AFW system. Therefore, damage to this valve operator or its electrical circuitry does not affect safe shutdown capability.

Loss of air supply or power to the 10 percent steam dump valves (PCV-19 to PCV-22) will cause the valves to remain in their closed position. This is the preferred failure mode, as opposed to the steam dump valves failing open causing uncontrolled depressurization of the steam generators. Steam generator pressure transmitters PT-516A and PT-526A provide nonsafety-related control signals to the 10 percent steam dump valves and could be damaged by a tornado. If the control schemes for these valves are damaged, the plant would remain at hot standby using the steam generator safety relief valves for secondary system heat removal until the secondary system is depressurized by manual operation of these valves using their handwheels. Therefore, tornado-induced damage to the 10 percent steam dump valve control scheme does not adversely affect safe shutdown capability.

The steam generator blowdown tank vent condenser and the 3/4-inch CCW supply and return piping are located outdoors at elevation 140 ft above the auxiliary building. This piping is part of CCW Header C and as such is a non-essential heat load. Tornado damage to the supply lines, return lines, or the steam generator blowdown tank vent condenser was evaluated to confirm that the CCW system leakage will be significantly less than the maximum acceptable system leakage. Therefore, tornado-induced damage to these components does not adversely affect CCW system operation.

Four-inch diameter containment hydrogen purge lines are located outdoors at elevation 140 ft above the auxiliary building. This system is used if accident conditions produce potentially explosive hydrogen concentrations inside containment. Given that extreme weather conditions do not have the capability to result in inside-containment accidents, the use of these lines would not be required after a tornado. Therefore, tornado-induced damage to these lines is of no consequence.

3.3.2.3.2.8 Turbine Building

Analysis of the basic structure of the turbine building, with partial loss of siding, provides a conservative estimate of capability to withstand a wind velocity of 225 mph. Under combined wind and missile load, the worst-case analysis based on elastic behavior shows resistance to at least 175 mph.

The 24-inch concrete walls of the turbine building can resist a combined wind and missile load corresponding to a wind velocity of 272 mph. The corresponding value for 12-inch concrete walls is 200 mph.

Purlins, girts, siding, roofing, doors, and louvers are potentially damaged by tornado wind velocities less than 175 mph. The siding system at the exterior walls (north and east for Unit 1, south and east sides for Unit 2) protecting the vital 4.16-kV switchgear and cable spreading rooms can withstand tornado winds with a velocity up to 200 mph. The roofing, siding, and louvers cannot withstand the tornado-induced missiles discussed in Section 3.3.2.1.2.

The turbine building contains major components of the emergency power system, described in Chapter 8, which are: (a) the three 4.16-kV vital buses (buses F, G, and H) for each unit, including associated switchgear, and (b) the six diesel generators, three for each unit. The possibility of tornado-related damage to these emergency power system components, and the potential effect on safe shutdown capability, has been considered and are discussed below. Such damage has potential impact on safe shutdown capability only if all offsite power is lost.

Vital 4.16-kV switchgear for the emergency power system is located at elevation 119 feet between column lines D and G (see Figures 1.2-14 and 1.2-18). Cable spreading rooms associated with this switchgear are located immediately below the switchgear at elevation 107 feet (see Figures 1.2-15 and 1.2-19). Both the switchgear and the cable spreading rooms are located above the concrete portion of the turbine building wall. Exterior turbine building walls in these areas consist of steel siding installed on a structural steel framework. Separation between components associated with each of the bus sections is provided by reinforced 8-inch concrete block partitions, as shown in Figure 3.3-4.

As shown in Figures 1.2-2 through 1.2-32, the turbine building structure provides protection from tornado-generated missiles above, below, and on two sides of the 4.16-kV switchgear and cable spreading rooms. The remaining two sides (north and east side for Unit 1, south and east side for Unit 2) are enclosed by steel siding on a structural steel framework. Some protection is provided by the structural steel, but tornado-generated missiles could penetrate the siding and damage emergency power system components. On the north and south sides (north side for Unit 1, south side for Unit 2), only components associated with bus H are directly exposed to such damage. On the east side, the ends of compartments housing components associated with buses F, G, and H are similarly exposed to tornado-generated missiles.

Concrete block partitions between vital buses minimize the possibility that tornado-generated missiles penetrating the north or south (north for Unit 1, south for Unit 2) turbine building walls would damage components associated with buses F and G.

The containment building for each unit and the auxiliary building provide a substantial amount of tornado missile protection for the east turbine building wall. In addition, the topography of the site reduces the likelihood that tornado-generated missiles would damage emergency power system components by penetrating the east wall of the turbine building.

Tornado-related damage would not be expected to affect more than one vital 4.16-kV bus. Such damage would not affect the capability for safe shutdown, even with the loss of offsite power.

The diesel generators are located in the west side of the turbine building, three between column lines 1 and 5 (Figure 1.2-16) and three between column lines 31 and 35 (Figure 1.2-20). The diesel generators in each group are separated from each other by 10-inch concrete walls. The exterior walls to the north, south, and east protecting the EDGs are constructed of minimum 12-inch concrete capable of withstanding 200 mph tornado wind and missile loads. Openings in the west walls of the turbine building between these column lines, and between floor elevations 85 and 107 feet, for diesel generators are protected by tornado missile barriers. The tornado missile barriers, consisting of closely spaced beams, are provided to protect the diesel generator compartments and their air intakes at elevation 85 ft at the west wall of the turbine building. The barriers are designed to withstand the combined effects of a 200 mph wind and tornado missiles.

The diesel generator compartment ventilation system (see Section 9.4.7) uses the diesel generator silencer rooms and an external plenum as the exhaust air flow path to the atmosphere. The external plenum, located at the west wall of the building, is integral with the turbine building structural framing and is enclosed on three sides and the bottom with corrugated metal siding. The east side of the plenum is open to the diesel generator silencer rooms at elevation 107 feet and the top is open to the atmosphere. The plenum was added in order to improve the operation of the ventilation system when the diesel engines are operating under extreme weather conditions.

The external plenum, including the steel framing, girts, siding, and turning vanes, is designed for the effects of 200 miles per hour tornado wind loads. The design is performed in accordance with the methodology given in BC-TOP-3A (Reference 9). The tornado wind speed corresponds to that used for the design of the added girts, which support the siding at the vital 4.16-kV switchgear and cable spreading rooms (see Section 3.3.2.3.1 and Table 3.3-2).

Due to its exposed location, the exhaust air plenum may be vulnerable to damage by a tornado-generated missile. The impact of a missile on the plenum could result in localized damage to the framing, which supports the plenum, or puncture of the siding, which forms the exterior boundary of the plenum. However, the plenum represents a small vulnerable area for missile impact, and is not designed for missile loads.

Since the only function of the ventilation exhaust plenum is to provide a directed flow path for the ventilation air to exhaust to the atmosphere, the maintenance of an airtight pressure boundary is not required. The primary requirement is that damage to the structure would not result in blockage of the flow path. It is unlikely that damage to the exhaust plenum by a missile would block the flow path sufficiently to have a significant impact on the function of the plenum. In the unlikely event that a tornado-generated missile did cause sufficient damage to the plenum to impact airflow, such damage would not be expected to affect more than one diesel generator and would not compromise the capability to achieve safe shutdown of the plant.

The diesel exhaust lines routed above elevation 107 ft are potentially vulnerable to tornado wind loads if the turbine building siding is blown off. Analyses demonstrate that the exhaust lines will not be affected by tornado wind loads. A tornado missile striking one of these lines has less significant consequences than the missile striking components within a 4.16-kV switchgear room.

The electrical conduits for EDG 2-3 at elevation 140 ft are routed close to the concrete floor and are shielded from tornado influences to some extent by CRPS piping and other raceways. The containment structure and the auxiliary building provide significant shielding for these raceways. The consequences of tornado damage of these conduits are no more significant than the missile striking components within a 4.16-kV switchgear room.

Banks of large diameter conduits from the 4.16-kV cable spreading rooms routed through the nonvital 12-kV switchgear room are separated by electrical division and are encased in rigid fireproofed vaults that afford some protection against tornado debris and minor missiles. This room is constructed of 12-inch reinforced concrete capable of withstanding tornado wind and missile loads. However, exhaust air louvers in the east wall provide a pathway for a tornado missile to strike banks of conduits routed directly in front of the wall penetrations. These wall penetrations are afforded a significant amount of shielding by the main and auxiliary transformers to the east. It is not credible for a tornado missile to strike more than one bank of conduits because of the spatial separation between the redundant conduits. The consequences of this tornado missile damage, which could affect one electrical division, are no more significant than a missile striking components within a 4.16-kV switchgear room.

The 4.16-kV switchgear room/cable spreading room ventilation system supply fans and supply ducting at elevation 119 ft and exhaust ducting at elevation 140 ft could be vulnerable to the effects of a tornado if the building siding is blown off. Compartment heatup analyses demonstrate that loss of this ventilation equipment does not jeopardize electrical equipment in these compartments due to elevated temperatures. Therefore, this ventilation system equipment is not required for safe shutdown of the plant.
The CCW heat exchangers and associated CCW and ASW piping, valves, and instrumentation are protected from the effects of tornado because of their location within the turbine building.

Plant protection system inputs from the main turbine first stage pressure are protected from the effects of tornado wind and missiles. The process tubing is routed in protected areas within the turbine building and is enclosed within a protective barrier. The sensing transmitters and signal circuitry are contained within the CCW heat exchanger room.

3.3.2.3.2.9 Outdoor Design Class I Tanks

Design Class I tanks located at the east end of the auxiliary building (Figure 1.2-2) include the condensate storage tank, the fire water storage and transfer tank, and the refueling water storage tank. These are concrete-protected steel tanks capable of withstanding a wind velocity of about 170 mph. They are also capable of withstanding a 150 mph wind with combined tornado-induced missiles. These capabilities do not take credit for the concrete encasement. When the concrete encasement is considered, these tanks are capable of withstanding a wind velocity of 300 mph. The anchorages that prevent the tanks from overturning have a design capacity greater than a 300 mph wind load. Based on these considerations, a tank failure producing an instantaneously large leak and flood is extremely unlikely, particularly as the tanks are concrete-protected. The 5.0-million-gallon Design Class II raw water reservoir provides a backup source of water supply.

The refueling water storage tank (Refer to Section 6.3.2.4.2) is normally used to supply borated water to the refueling canal for refueling operations when the plant is already in a safe shutdown condition. Its Design Class I function is to supply borated water to ECCS pumps and containment spray pumps following a safety injection signal.

The fire water storage and transfer tank, with a 300,000 gallon capacity (Section 9.5.1), is the shared backup source of fire water. The normal source of fire water is the shared raw water reservoir. The fire water storage and transfer tank inventory may also be used to supplement the condensate storage tank for AFW and as a source of makeup water to the spent fuel pool. While tornado-induced damage to the fire water storage and transfer tank is considered extremely unlikely, such damage would not affect safe shutdown capability of the units. The raw water reservoir provides an alternate capability for the functions performed by this tank.

The condensate storage tank (Section 9.2.6) supplies normal makeup and rejection requirements of the steam plant. Its Design Class I function is to provide 200,000 gallons for Unit 1 and 166,000 gallons for Unit 2 (out of a total capacity of 425,000 gallons) for the auxiliary feedwater system during the plant cooldown to 350°F. While tornado-induced damage to the condensate storage tank is considered extremely unlikely, such damage would not affect safe shutdown capability. The raw water reservoir provides an alternate capability for the function performed by this tank.

The raw water reservoir is the backup source of auxiliary feedwater. The reservoir (Section 9.2.3) is below grade, concrete- and PVC-lined, and divided into two compartments to permit maintenance without removing the reservoir from service.

The total potential demand on the reservoir resulting from total loss of water inventory in the condensate storage tanks for both units is 392,000 gallons. This amount of water represents less than 10 percent of the capacity of both compartments in the reservoir and about 16 percent of one compartment. Based on APED-5696, Tornado Protection for the Spent Fuel Storage Pool (Reference 5), a relatively large and shallow pool, such as the raw water reservoir, is susceptible to partial dewatering by a tornado of specific size and path. A detailed analysis of water loss has not been made since simultaneous, essentially complete loss of water from both tanks and from the reservoir is not considered credible.

3.3.2.3.2.10 Intake Structure

The intake structure and the ASW pump room vent shaft extensions are capable of withstanding the combined effect of a 240-mph wind velocity and a missile. The gantry crane for servicing equipment within the structure is susceptible to overturning at a wind velocity above 110 mph in its anchored position. In this position the crane is located near the north end of the craneway approximately 80 feet from the vent shaft above the intake structure; damage to the structure or to its components from the crane is considered extremely unlikely. The crane is kept in its anchored position when not in use.

3.3.2.3.2.11 Design Class II Outdoor Electrical Equipment

In addition to the structures listed in Table 3.3-2, the tornado-resisting capability of Design Class II outdoor electrical equipment has been evaluated. This capability ranges from greater than 250 mph for main transformers, to about 100 mph for such equipment as the isophase bus duct structure. No missiles more severe than the hypothetical missiles are generated by a tornado-induced failure of this equipment. In addition, no Design Class I equipment is endangered by any such failure.

3.3.2.3.2.12 Control Room Pressurization Equipment

The control room pressurization system (CRPS) fans, ductwork, dampers and associated controls located on the turbine building operating deck and the auxiliary building roof have the potential of being damaged by the effects of a tornado.

The CRPS is used to minimize radioactive contamination in the control room following an accident. Since a tornado is not assumed to occur concurrently with an accident or cause an accident producing significant releases, operation of the CRPS after a tornado is not required. The control room ventilation system can operate in ventilation mode 1 (normal ventilation) or ventilation mode 3 (100 percent recirculation) if the CRPS is impaired. Either mode of ventilation operation is acceptable under these circumstances. Although tornado protection of the CRPS is not required functionally, PG&E committed to design the system considering tornado missile effects.

The CRPS is of the "dual air inlet" type where two widely spaced inlets are located on opposite sides of potential radioactive gas sources. Because damage to the ducting might affect the capability of the system to protect the operators and because the duct is not considered to have a redundant counterpart, the CRPS ductwork is protected against tornado missiles.

Only one missile is assumed to act at any time (see Section 3.3.2.1.2). Missile protection is only provided where the single missile may damage both redundant components. CRPS instrumentation is installed such that a single missile will not damage both redundant components. The electrical cable and conduits are considered small targets having a low probability of impact. Also the Class I electrical conduit separation criteria provides a degree of assurance that a single tornado missile will not damage redundant circuits.

On the basis of low probability, tornado design criteria were not required for the granting of construction permits for DCPP (Reference Section 3.3). Standard Review Plan (SRP) 3.5.1.4, Revision 0, dated November 24, 1975, states:

> "...at the operating license stage, applicants who were not required at the construction permit stage to design to one of the above missile spectra...should show the capability of the existing structures and components to withstand at least missiles ‘C’ and ‘F’..."

Missiles "C" and "F" are:

	Missile	Horizontal Velocity, ft/sec
C	Steel rod, 1-in. diameter, 3-ft long, 8 lb	259
F	Utility pole, 14-in. diameter, 35 ft long, 1500 lb	241

As shown in Table 3.3-5, missile "F" will attain a maximum missile elevation of 5 feet above ground elevation. CRPS equipment located on the turbine operating deck, elevation 140 ft, is well above the 5 foot limitation and would not be affected by missile "F." Therefore, the only missile for consideration is the 1-inch diameter steel rod.

3.3.2.3.2.13 480-V Switchgear and 125-Vdc Inverter Room Ventilation System

The portion of the ventilation system for the 480-V switchgear and 125-Vdc inverter rooms (see Section 9.4.9) located on the auxiliary building roof is potentially vulnerable to the effects of a tornado. Potentially affected equipment includes the redundant supply and exhaust fans, dampers, ducting, and instrumentation and electrical raceways supporting the operation of the ventilation system.

This ventilation equipment is shielded on four sides by the containment structure, the turbine building, a ventilation fan room, and the fuel handling building superstructure. Based on this protected location, the Seismic Category I design of the system, and component redundancy, it is highly unlikely that a tornado would damage this system such that a complete loss of function would occur.

If a failure of these ventilation systems did occur, it would be apparent since the equipment temperature monitoring system would alert control room personnel to elevated temperatures in these areas. The equipment temperature monitoring system is powered from vital power and is unaffected by tornadic influences.

Compartment heatup analyses demonstrate that sufficient time is available for plant personnel to take appropriate actions, based on plant conditions, before electrical equipment function is affected from overheating.

3.3.2.4 Supplementary Analysis of Additional Tornado Missiles: Estimated Maximum Missile Velocity, Required Barrier Thickness

The following expanded list of tornado-borne missiles has also been analyzed to determine the maximum velocity attained and the thickness of a reinforced concrete missile barrier necessary to preclude perforation or the generation of secondary missiles. The previously discussed three hypothetical missiles (see Section 3.3.2.1.2) are approximately the same as items (1), (6), and (7) in the following list:

(1) 4- x 12-inch plank, 12 feet long, with a density of 50 lb/ft^3

(2) Utility pole 13.5 inches in diameter by 35 feet long with a density of 43 lb/ft^3

(3) 1-inch solid steel rod, 3 feet long, with a density of 490 lb/ft^3

(4) 6-inch Schedule 40 pipe, 15 feet long, with a density of 490 lb/ft^3

(5) 12-inch Schedule 40 pipe, 15 feet long, with a density of 490 lb/ft^3

(6) 3-inch Schedule 40 pipe, 15 feet long, with a density of 490 lb/ft^3

(7) 4000-pound automobile with a volume of 1.67 x 6 x 17 ft

The analysis employed in developing the maximum missile velocity is based on methods described in Reference 6. The tornado characteristics used as initial conditions include a constant tangential wind speed of 250 mph.

A three-dimensional, right-circular-cylinder representation is assumed for the funnel cloud. Vertical and radial velocity components are taken from Bates & Swanson (Reference 7).

The vertical component varies with elevation. The radial component varies from a peak value at the outside surface and drops abruptly to zero at the inside surface of the funnel cloud. With this model, the tornado missile forcing function and, consequently, the potential missile destructive forces do not significantly vary with elevation, since the vertical velocity component and its variation with elevation are small relative to the tangential velocity component.

The calculation of the ejection velocity of the postulated missile from the tornado vortex described above is based on solving the equation of motion for the missile. The missile is assumed to accelerate as it follows the path of the maximum tangential wind speed at the surface of the funnel cloud. It is ejected when the pressure differential and aerodynamic forces acting on the missile are overcome by centrifugal forces. Further assumptions are: (a) the missiles do not tumble, and present the maximum value of C_d A/W while in flight, and (b) the ejection velocity thus determined is the maximum velocity attained by the missile.

Modified missile velocities are calculated for those missiles incapable of being sustained after initial injection into the tornado wind field. Injection is calculated by assuming a 0.2-second impulse due to the aerodynamic lift force. From this calculation, an initial height for missile injection can be determined. Subsequent suspension and increase or decrease in elevation is determined by a new force balance including lift and gravity forces acting on the missile at the initial injection height. For the nonsustained missile, the modified horizontal velocity upon its return to the elevation of origin is then computed. Horizontal and vertical velocity components are decoupled in this latter calculation for simplicity. This approach is justified because the vertical wind speed is small compared to its horizontal component.

The required thickness of a reinforced concrete barrier to preclude perforation due to end-on impact of the additional tornado-induced missiles is computed by the modified Petry formula. The estimated thickness required to prevent the formation of secondary missiles is conservatively taken as 1.5 times the thickness required to preclude perforation. This approximation is based on a comparison of the thickness required to prevent perforation and spalling of concrete targets. These thicknesses were determined for various missile types with velocities ranging from 40 to 250 fps. Calculations were based on the Army Corps of Engineers and National Defense Research Committee formulas. Although these formulas are applicable for missile velocities exceeding 500 fps, they represented the only available basis for estimating the thickness required to prevent secondary missile generation.

The results of the supplementary analysis of additional missiles are presented in Tables 3.3-5 and 3.3-6.

3.3.3 REFERENCES

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12. Guidelines and Technical Bases for NUMARC Initiatives Addressing Station Blackout at Light Water Reactors, Nuclear Management and Resources Council, NUMARC 87-00, Revision 0.

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14. PG&E Letter DCL-94-133 to the NRC, Response to Generic Letter 88-20, Supplement 4, Individual Plant Examination of External Events for Severe Accident Vulnerabilities, June 27, 1994.

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3.4 WATER LEVEL (FLOOD) DESIGN

Safety-related structures, components, and equipment are designed to withstand the effects of potential flooding, as required by GDCs 2 and 4.

3.4.1 FLOOD ELEVATIONS

The discussion in Section 2.4 demonstrates that Diablo Creek is adequate to handle the probable maximum flood (PMF), and that yard and roof drainage designs are such that it is not possible to develop sufficient ponding to flood safety-related buildings. Thus, the depth of water at the plant location for the PMF is zero.

The intake structure is designed with an elevated air intake so that the Design Class I auxiliary saltwater pumps can operate during the design combination tsunami-storm wave runup to elevation +48 feet mean low-low water (MLLW) (+45.4 feet mean sea level (MSL)). These pumps are located below elevation zero feet MSL. The pumps are mounted at a nominal floor elevation -2.1 feet MSL, and each pump is housed in a separate watertight compartment. The location and arrangement of this equipment are shown in Figure 9.2-2.

3.4.2 PHENOMENA CONSIDERED IN DESIGN LOAD CALCULATIONS

Sections 2.4.3, 2.4.5, and 2.4.6 discuss the characteristics of the storm waves and tsunami flooding that were considered in the design of the intake structure and auxiliary saltwater pump compartments.

3.4.3 FLOOD FORCE APPLICATION

The hydraulic forces on the intake structure resulting from the design flood conditions and when the breakwaters are assumed to be degraded to MLLW were determined by hydraulic scale model tests and reported in References 1, 2, and 3. As discussed in Section 2.4.6.6, analysis of wave splash-up from the hydraulic scale model tests was performed as documented in Reference 4 to demonstrate that ingestion of sufficient seawater through the intake structure snorkels into the auxiliary saltwater (ASW) pump rooms is extremely unlikely to jeopardize operation of the ASW pumps.

3.4.4 FLOOD PROTECTION

Sections 2.4.5 and 2.4.6 describe flooding protection provisions for safety-related structures, systems, and components associated with sea wave activity and tsunami. Isolation from effects of flooding due to leakage or rupture of piping is discussed in Section 9.2.

During excavation for the Unit 1 containment structure, some seepage of ground water was encountered. Since the flow of water into the excavation was slight, no special provisions for the base slab were considered necessary. However, to provide the capability to detect the presence of water in the future, two collector loops were installed under each containment structure: one at approximate elevation 52 feet (bottom of the reactor cavity), and one at approximate elevation 74 feet (bottom of the base slab). The loops at each containment structure are connected to observation wells which may be monitored. The observation wells are capped to contain radon gas emitted from the wells. Provisions are made on the cap plate to be able to monitor the wells, and, if water should accumulate, to allow groundwater to be pumped from the wells.

3.4.5 REFERENCES

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2. F. Raichlen, Investigation of Wave Structure Interactions for the Cooling Water Intake Structure of the Diablo Canyon Nuclear Power Plant, December 1982.

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3.5 MISSILE PROTECTION

The DCPP design provides protection for PG&E Design Class I systems and components, including engineered safety features (ESF), from internally and externally generated missiles. This section discusses the design bases for DCPP missile protection and includes information on missile barriers and loadings, missile selection, selected missiles, barrier design procedures and missile barrier features.

3.5.1 DESIGN BASES

3.5.1.1 General Design Criterion 40, 1967 – Missile Protection

Engineered safety features are protected against missiles generated inside and outside of containment that might result from plant equipment failures.

3.5.1.2 Missile Protection Safety Function Requirements

(1) Protection From Missiles

PG&E Design Class I structures, systems and components inside and outside of containment are protected against the effects of missiles, which may result from equipment failures and from events and conditions outside the nuclear power unit.

3.5.2 SYSTEM DESCRIPTION

3.5.2.1 MISSILE BARRIERS AND LOADINGS

DCPP was designed and built with structures, shields, and barriers that are designed to withstand missile effects.

3.5.2.1.1 Missiles Generated Within the Containment

The principle design bases for missiles generated within the containment are that missiles generated coincident with a LOCA shall not cause loss of function of any ESF, PG&E Design Class I equipment or cause a loss of containment integrity (refer to Section 3.6). The containment is defined as the containment structure, liner and penetrations, the steam generator shell, the steam generator steam-side instrumentation connections, and the steam, feedwater, blowdown, and steam generator drain piping within the containment structure.

Other than the emergency core cooling system (ECCS) lines that must circulate cooling water to the reactor vessel, the ESFs are located outside the crane wall. The ECCS lines are routed outside of the crane wall so that the penetrations are in the vicinity of the loop to which they are attached. Physical separation together with barrier protection provided by the refueling cavity walls and various structural beams serve to minimize the potential for missiles generated in one loop from damaging adjacent loops. The portion of the steam and feedwater lines within the containment has been routed behind barriers that separate them from all reactor coolant piping.

The lower steam generator shell connecting lines are routed so that they are not in the direct path of any postulated missile.

The systems located inside the containment structure have been examined to identify and classify potential missiles. The basic approach is to ensure design adequacy against generation of missiles, rather than to allow missile formation and then try to contain their effects.

Catastrophic failure of the reactor vessel, steam generators, pressurizer, reactor coolant pump casings, or piping leading to generation of missiles is not considered credible. Massive and rapid failure of these components is not credible because of the material characteristics, inspections, quality control during fabrication, erection, operation, conservative design, and prudent operation as applied to the particular component (refer to Section 3.6). The reactor coolant pump flywheel is not considered a source of missiles for the reasons discussed in Section 5.2.6. Nuts and bolts are of no concern because of the small amount of stored elastic energy.

Components considered to have a potential for missile generation inside the reactor containment are listed as follows:

(1) Control rod drive mechanism (CRDM), drive shaft, and the drive shaft and drive mechanism latched together

(2) Certain valve bonnets

(3) Temperature and pressure sensor assemblies

(4) Pressurizer heaters

The structures designed to withstand missile effects within the containment are:

(1) The integrated head assembly missile shield provided over the control rod drive mechanisms is intended to block missiles defined in Tables 3.5-2 and 3.5-3.

(2) The pressurizer enclosure is designed to block missiles listed in Tables 3.5-4 through 3.5-6.

(3) The polar crane wall, fuel transfer canal walls, and operating floor at elevation 140 feet constitute barriers for temperature elements and valve bonnets as described in Tables 3.5-4, 3.5-5 and 3.5-6, to prevent them from reaching the containment liner.

3.5.2.1.2 Missiles Generated Outside the Containment

The ESF and PG&E Design Class I systems located outside the containment structure have been reviewed to determine the possible sources and consequences of missiles. Catastrophic failure of pressure vessels and system piping outside the containment structure is not considered credible as a source of missiles because of the conservative design, material characteristics, inspection during erection, and prudent operation. Pressurized gas containers of significant quantities are summarized in Table 3.9-11. Thrust loads resulting from a postulated failure of the largest connected pipe or manifold have been calculated to produce stresses no larger than yield in the container hold-down structure; therefore, no significant missiles are postulated from any of these containers. As shown in Table 3.9-11, other protective measures to prevent the loss of function of ESF and PG&E Design Class I equipment include compliance with Occupational Safety and Health Administration (OSHA) regulations in 29 CFR Part 1910. Many of the tanks listed in Table 3.9-11 are in locations remote from such ESF and PG&E Design Class I structures, systems and components.

Components considered to have a potential for missile generation outside the containment structure are:

(1) Main turbine

(2) Main feedwater pump turbine

(3) Auxiliary feedwater pump turbine

3.5.2.1.2.1 Missiles from the Main Turbines

Based on the main turbine design, factory test procedures, the redundancy in control systems, and routine plant testing and inspection, the probability of the generation of main turbine missiles is so remote that turbine missiles are not considered credible events. See Section 3.5.2.2.2.1 for a discussion of the main turbine missile analysis.

3.5.2.1.2.2 Missiles from the Main Feedwater Pump Turbines

Due to the location of the main feedwater pump turbines on the west side of the turbine building at elevation 85 feet, failure of these turbines and the subsequent release of a missile would not prevent PG&E Design Class I SSCs, including ESFs, from performing their design function. Protection from missiles for Class 1E cables and equipment is discussed in section 8.3.1.4.10.2.

3.5.2.1.2.3 Missiles from the Auxiliary Feedwater Pump Turbine

For DCPP Unit 1 or Unit 2, the only ESF or PG&E Design Class I equipment in the area of the turbine-driven auxiliary feedwater pump are the two make-up water transfer pumps and the two motor-driven auxiliary feed pumps. These pumps are located 30 feet from the turbine-driven pump. With this amount of separation, a rotor fragment would have to be ejected within a 6° rotational span from the turbine casing in order to impinge directly on the nearest motor-driven pump.

The disk fragments would be most likely to damage the first motor-driven auxiliary feed pump in the direct line from the turbine pump, leaving the second motor-driven auxiliary feed pump fully operational. Reactor coolant system (RCS) decay heat can be dissipated safely with the flow from one motor-driven feed pump.

A missile barrier is provided over the pump turbines to contain any potential missiles generated by disk failure.

3.5.2.1.3 Missiles Generated by Natural Phenomena

Tornado-generated missiles and protection from tornado-generated missiles are discussed in Section 3.3.2.

3.5.2.1.4 Site Proximity Missiles

Nearby industrial, transportation, and military facilities are described in Section 2.2. There are no such facilities within 5 miles of the site. The probability of missiles originating from these facilities striking PG&E Design Class I structures or components on the site is so small that missile loadings from this source have not been included in structure or component design bases.

3.5.2.2 MISSILE SELECTION

The missiles selected for each structure and the basis for their selection are discussed below.

3.5.2.2.1 Missiles Generated Within the Containment

Gross failure of a control rod drive mechanism (CRDM) housing sufficient to allow a control rod to be rapidly ejected from the core is not considered credible for the following reasons:

(1) All control rod drive mechanisms are shop pressure-tested at 3107 psig, whereas the reactor coolant system (RCS) operates at 2235 psig.

(2) The pressure housings were individually hydrotested. The lower latch housing to nozzle connection is hydrotested during hydrotest of the completed replacement reactor vessel closure head.

(3) Stress levels in the mechanisms are not affected by system transients at power, or by thermal movement of the coolant loops.

(4) The mechanism housings are made of Type 304 stainless steel. This material exhibits excellent fracture notch toughness at all temperatures that will be encountered.

(5) The CRDM housing plug is an integral part of the rod travel housing.

However, if it is postulated that the top of the rod travel housing portion of the control rod drive mechanism becomes ruptured and is forced upward by the water jet, the following sequence of events is assumed. The drive shaft and control rod cluster are forced out of the core by the differential pressure of 2500 psi across the drive shaft. (The drive shaft and control rod cluster, latched together, are assumed fully inserted when the accident starts.) After approximately 12 feet of travel, the rod cluster control spider hits the underside of the upper support plate. Upon impact, the flexure arms in the coupling joining the drive shaft and control cluster fracture, freeing the drive shaft from the control rod cluster. The control cluster is stopped by the upper support plate, but the drive shaft continues to be accelerated upward to be stopped by the integrated head assembly missile shield.

Valve stems are not considered a credible source of missiles. All of the isolation valves installed in the RCS have stems with backseats. This effectively eliminates the possibility of ejecting valve stems even if the stem threads fail.

Valves are designed against bonnet-body connection failure and subsequent bonnet ejection by means of:

(1) using the design practice of ASME B&PV Code Section VIII-1968 for bolting

(2) using the design practice of ASME B&PV Code Section VIII-1968 for flange design

(3) controlling the torque load during the bonnet body connection stud tightening process

The pressure-containing parts are designed in accordance with requirements of either ANSI B16.5 or MSS SP 66, except for the accumulator check valves which are designed to ASME B&PV Code Section III-1968 Edition.

Valve missiles are not generally postulated due to the above discussion. However, the valves in the region where the pressurizer extends above the operating deck are capable of generating missiles. The valves in this region are the pressurizer safety valves, the motor-operated isolation valves in the relief line, and the air-operated relief valves. Although failure of these valves is also not considered credible, failure of the valve bonnet-body bolts is postulated and provisions are made to ensure integrity of the containment liner from the resultant bonnet missile.

The only credible source of jet-propelled missiles from the reactor coolant piping and piping systems connected to the RCS are the temperature and pressure sensor element assemblies. The resistance temperature element assemblies are considered to be "with well" (i.e., with the piping thermowell attached) and "without well" (without the piping thermowell attached).

A temperature sensor element is installed on the reactor coolant pumps close to the radial bearing assembly. A hole is drilled in the gasket and sealed on the internal end by a steel plate. In evaluating missile potential, it is assumed that this plate could break and the pipe plug on the external end of the hole could become a missile.

In addition, it is assumed that the weldment between the instrumentation well and the pressurizer wall could fail and the well and sensor assembly could become a jet-propelled missile.

Finally, it is assumed that the pressurizer heaters could become loose and become jet-propelled missiles.

3.5.2.2.2 Missiles Generated Outside the Containment

3.5.2.2.2.1 Main Turbine

The high pressure (HP) turbines and the generators for DCPP Unit 1 and Unit 2 are manufactured by Siemens-Westinghouse Electric Corporation. The low pressure (LP) turbines A, B, and C, originally supplied by Siemens-Westinghouse, have been retrofitted with Alstom rotors and casings.

Factory test procedures, redundancy in the control system, routine testing of the main steam valves and the mechanical overspeed protection system while the unit is carrying load, make generation of missiles that might penetrate the turbine casing due to a turbine overspeed highly improbable. An LP rotor inspection program ensures that the integrity of the LP rotors is maintained and the failure probability is below the NRC allowable of 10^{-7} failures per year.

Three important criteria contribute to preventing destructive overspeed of a turbine:

(1) *Factory test procedures*

Destructive testing is performed on material specimens taken from forgings in addition to an ultrasonic test of each forging following major heat treatment. These test procedures ensure sound forgings with mechanical properties (tensile strength, yield strength, ductility, and impact strength), which meet specified levels.

(2) *Redundancy in the control system*

The overspeed protection control (OPC) system controls turbine overspeed in the event of a partial or complete loss of load, or if the turbine reaches or exceeds 103 percent of rated speed. A mechanical overspeed trip device is also provided that will automatically trip the unit at 111 percent of rated speed. An electronic trip signal is generated by the digital electrohydraulic (DEH) control system at 111.5 percent of rated speed, as redundant overspeed protection. This trip signal is set approximately 10 rpm higher than the mechanical overspeed device.

(3) *Routine testing of the main steam valves and the mechanical overspeed protection system*

Functional tests of the main turbine steam inlet valves are performed semi-annually while the unit is carrying load, per Siemens-Westinghouse instructions for the DCPP steam turbines.

References 9, 10, 11, and 12 contain an update on the industry turbine valve failure rates and assessment of the turbine destructive overspeed probabilities, and forms the basis of the turbine valve testing frequency for DCPP.

A routine LP rotor inspection program for DCPP Unit 1 and Unit 2 assures that the integrity of LP rotors is maintained. The Alstom LP rotor inspection intervals are based upon fleet and industry experience. The inspection interval for DCPP's Alstom LP rotors is conservatively based upon worst case hypothetical stress corrosion cracking (SCC) failure mechanisms, crack initiation, size and growth rates, in keeping with the OEM's methodology for determining inspection intervals.

The resulting LP turbine rotor inspection program, committed to by PG&E, eliminates the potential for operational development of a crack to a critical depth. Rotor forgings are subject to inspection and testing both at the forging suppliers and manufacturer's facilities. Thus, an LP rotor failure at a speed less than or equal to the design overspeed is not considered a credible event.

Because of the extensive overspeed and backup controls, routine testing of the turbine inlet valves and overspeed protection system, acceleration beyond design overspeed (120 percent of rated speed) to the destructive overspeed (approximately 151 percent for Alstom rotors) is not considered a credible event.

Because of the LP rotor inspection program and preoperational forging inspection described in Section 3.5.2.2.2.1, generation of a missile at design overspeed or less is not considered a credible event.

The possibility that a postulated crane fall or falling construction debris could damage elements of the turbine trip system in an earthquake has been considered in relation to the main turbine failure probability. The turbine building crane is parked away from the steam inlet valves during turbine operation to preclude damage to the valves from a postulated crane fall. Review of the vulnerability of the main turbine inlet valves and the overspeed trip system to falling debris indicates that no postulated structural debris could prevent a trip for the following reasons:

(1) The siding and roofing of the turbine building are constructed to withstand design wind loads, which are in excess of the loads they would experience during an earthquake. Therefore, falling corrugated metal roofing or siding is not considered a credible event.

(2) The main steam stop valves and the overspeed trip mechanism are located on the high-pressure turbine. Falling debris, such as rivets and small scrap metal, would not damage the stop valve bodies, actuators, or trip mechanism in a manner that would prevent valve closure.

(3) The reheat stop valves are located below the turbine deck which protects them from falling debris.

(4) The intercept valves are located above each LP turbine, but their operation is not necessary for stopping the main turbine on a turbine overspeed trip.

(5) The electrohydraulic (EH) fluid system piping that supplies high-pressure oil to the main turbine steam inlet valves runs both above and below the turbine deck. Because most of the EH piping is either below the turbine deck or sheltered by the valve bodies and inlet piping, it is highly unlikely that said debris would impact the EH piping. In the extremely unlikely event that such debris were to impact the EH piping, the piping would have to be crimped completely shut to prevent the trip system from operating. This is not considered a credible event. A partially crimped line

would not disable the trip system. A broken or punctured line would result in a loss of EH pressure resulting in HP stop valve and re-heat stop valve closure, thus stopping the main turbine.

The falling debris considered included corrugated metal roofing, rivets, and small scrap metal that may have been left after the turbine building roof was constructed.

In summary, the probability of generation of an HP turbine missile by speed in excess of the design overspeed, or of an LP turbine missile of any kind, is extremely remote. These are not considered credible events.

3.5.2.2.2.1.1 Potential Missiles

Analyses and tests regarding the generation and effects of missiles caused by the main turbine accelerating to design overspeed have been carried out by both Alstom and Siemens-Westinghouse Electric Corporation. The analyses and tests described below consider the potential sources of missiles resulting from postulated failures of the HP turbine rotor and the LP turbine rotor or last stage blade loss at design overspeed (120 percent of rated speed) and the capability of the turbine casings to contain the postulated missiles.

Experimental Results

Siemens-Westinghouse conducted a test program at its research laboratories to evaluate the missile-containing ability of its steam turbines. The tests involved spinning alloy steel discs to failure within various carbon steel containments. The discs were notched to ensure failure in a given number of segments at the desired speed. Test results were correlated with various parameters descriptive of the missile momentum and energy and the geometry of the missile and containment.

The containments were of varying geometry but all were axisymmetric and concentric with the rotation axis of the disc. They ranged in complexity from a circular cylinder to containments that approximated actual turbine construction.

From these tests, logical criteria were evolved for predicting the missile-containing ability of various turbine structures. In addition, the tests also served to determine the mode of failure that certain structural shapes common to turbine construction undergo when hit by a missile. This is important since the mode of failure has a great influence on the amount of energy absorbed by the containment.

Fracture of the Siemens-Westinghouse discs into 90, 120, and 180° segments was considered. Calculations show that the 90° fragments pose the greatest threat as external missiles.

A 120° segment has an initial translational kinetic energy 12.5 percent greater than that of a 90° segment; however it also has a 33 percent greater rim periphery, resulting in greater energy loss when penetrating the turbine casing. Therefore, 90° and 120° segments have nearly equal kinetic energy leaving the turbine casing; but since a 90° segment has smaller impact area, it represents a more severe missile.

The initial translational kinetic energy of a half disc is equal to that of a quarter disc. Because of kinematic considerations, a half disc segment will always impact with the rotor after fracture. The 180° segment, due to its larger size, will subject the stationary parts to greater deformation. As a result, the 180° segment will leave the turbine casing with lower energy than the 90° segment.

For the purpose of evaluating the missile-containing ability of the turbine structure, the discs have been postulated to fail in four quarters. Before failure, a disc has a total energy, which is purely rotational, of $1/2(I\omega^2)$, where I is the mass moment of inertia of the disc about its axis of rotation and ω is the angular velocity of the turbine at the postulated failure speed. After failure, the mass center of each fragment translates at a velocity of ωr, r being the distance from the rotation axis of the disc to the mass center of the fragment. In addition, the fragment rotates about its center of mass with an angular velocity of ω. The initial rotational energy of the disc is partitioned into both the translational and rotational kinetic energy of the fragments.

Test results and analytical considerations indicate that the translational kinetic energy of a fragment is of much greater importance than the rotational kinetic energy in predicting the ability of the fragment to penetrate the turbine casing. Rotational kinetic energy tends to be dissipated as a result of friction forces developed between the fragment surfaces and stationary parts.

These principles apply to fragments that would be generated by failure of the HP turbine rotor.

High-Pressure Turbine Analysis

The HP turbine element is of double-flow design. Steam enters at the center of the turbine element through four inlet pipes, two in the base and two in the cover. These pipes feed four double-flow nozzle chambers flexibly connected to the turbine casing. Steam leaving the nozzle chambers passes through the Rateau control stage and flows through the reaction blading. The reaction blading is mounted in the blade rings which in turn are mounted in the turbine casing. The main body of the rotor weighs approximately 100,000 pounds.

Calculations were performed to determine the effects of a postulated failure of the HP turbine rotor at design overspeed (120 percent of rated speed). These calculations show that all fragments generated by any postulated failure of the HP turbine rotor would be contained by the HP turbine blade rings and casing.

Low Pressure Turbine Analysis

Alstom has also evaluated the probability of generating LP turbine missiles, providing finite element analyses, inspection results and operating experience in References 14 through 19. Alstom rotors are far less vulnerable to SCC, as compared to the Siemens-Westinghouse rotors, due to improved design features, materials, manufacturing, and factory inspection techniques. Therefore, the Alstom rotor inspection intervals are longer than the original Siemens-Westinghouse intervals.

In Reference 14, Alstom presents criterion governing nuclear LP rotor inspection intervals. Since there are no field failures or indications of SCC in the relevant directions which might produce a missile, Alstom developed hypothetical cases to set criteria governing crack growth rate, critical crack size, and fraction of crack size allowed. In this manner, Alstom’s missile analysis is equivalent to the methodology used by Siemens-Westinghouse.

From this analysis, Alstom produced an inspection interval versus missile generation probability chart (Figure 9, Reference 14). This shows that the maximum allowable inspection interval could be as long as 25 years and still meet the minimum requirements of RG 1.115. A low pressure turbine rotor inspection program for DCPP Unit 1 and Unit 2 assures that the integrity of the LP rotors is maintained. The Alstom criteria governing LP turbine rotor inspection sets forth recommendations based on crack growth rate, critical crack size, and fraction of critical crack size allowed.

The LP turbine program inspection frequency derived from the missile analysis governs the maximum interval between inspections. DCPP will follow the vendor recommendation.

The Alstom (single) inner cylinder does not enclose the last stage blades (LSBs) making loss of a LSB an unacceptable event. Given the blade mass and projected exit velocity during a hypothetical blade loss event, it is likely the blade would penetrate the outer housing. Alstom has incorporated several LSB design features, such as tangential fir tree entry, snubber location and design, and blade frequency tuning to prevent resonant conditions which might lead to high cycle fatigue crack growth. Additionally, operational avoidance regions are procedurally enforced, improved UT factory inspection techniques will detect manufacturing defects, and routine maintenance inspections will preclude undetected crack propagation. A fracture mechanics deterministic analysis presented in Reference 15 substantiates that this type of missile generation from a LSB loss is not a credible event.

3.5.2.2.2.2 Auxiliary Feedwater Pump Turbine

The DCPP Unit 1 and Unit 2 turbine-driven auxiliary feedwater pumps are driven by Terry Steam Turbine Company turbines with a casing constructed of cast steel. The turbine disc is a solid forging.

When required to operate, the turbines run at speeds ranging from 4000 to 4260 rpm. The overspeed trip on the turbine is a centrifugal weight type, which is set at 4950 plus or minus 50 rpm.

The construction and periodic testing of the turbine make it an improbable source of missiles. It is calculated that the probability of missile generation is less than 10^{-7} events per year. Hence, using NRC criteria, such an event need not be postulated. However, if failure of the turbine did occur, the most severe missile generated is assumed to be a 90° segment of the rotor.

3.5.2.3 SELECTED MISSILES

The characteristics for each selected missile, such as origin, weight, dimensions, impact velocity and orientation, and material composition, are described below.

3.5.2.3.1 Missiles Generated Within the Containment

The characteristics of the control rod drive shaft (with the disconnect rod) missile are given in Table 3.5-2. The characteristics of the control rod drive shaft and the disconnect rod latched to the drive mechanism are given in Table 3.5-3.

The missile characteristics of the bonnets of the valves in the region where the pressurizer extends above the operating deck are given in Table 3.5-4.

The missile characteristics of the piping temperature element sensor assemblies are given in Table 3.5-5. A 10 degree expansion half-angle water jet has been assumed. The missile characteristics of the piping pressure element assemblies are less severe than those shown in Table 3.5-5.

The missile characteristics of the reactor coolant pump temperature element, the instrumentation well of the pressurizer, and the pressurizer heaters are given in Table 3.5-6. A 10-degree expansion half-angle water jet has been assumed.

3.5.2.3.2 Missiles Generated Outside the Containment

3.5.2.3.2.1 Auxiliary Feedwater Pump Turbine

As noted in Section 3.5.2.2.2.2, the probability of generating a missile from the auxiliary feedwater turbine is less than the NRC threshold value for credible events. While not required, an evaluation of the auxiliary feedwater turbine's ability to generate a missile has resulted in the following conclusions. A disc rupture at the maximum turbine speed, consistent with a single failure of the turbine control system, will not generate a missile capable of penetrating the turbine casing. However, the postulation of multiple control system failures leads to the rupture of the turbine disc at a speed of 14,000 rpm. The resulting missile is capable of penetrating the turbine casing. The missile characteristics are as follows:

Velocity = 549 fps
Impact area = 54.1 in^2
Weight = 43.2 lb (includes 13.2 lb for the turbine casing)

3.5.2.4 BARRIER DESIGN PROCEDURES

The barriers are designed to prevent penetration by the postulated missiles. The design procedures are described below:

3.5.2.4.1 Missiles Generated Within the Containment

The steam generator shell for the original steam generators (OSGs) was calculated to resist penetration of the postulated missiles using the method illustrated in ORNL-NSIC-5. The shells for the replacement steam generators (RSGs) are fabricated with materials similar to or in some cases improved from those used for the OSGs. In addition, the RSG shells are approximately as thick as the OSG shells, are made from SA-508 forging material that is equivalent to the SA-533 plate material used in the OSG shells, and do not have longitudinal weld seams in the cylindrical sections as do the OSGs.

In conclusion, the RSGs are evaluated to be at least as resistive to missile penetration as the OSGs.

The missile shield structure over the control rod drive mechanisms was evaluated using a formula derived from the Ballistic Research Laboratories (BRL) formula:

$$T = \frac{\left(E_k\right)^{2/3}}{670\ d} \tag{3.5-11}$$

where:

T = required steel wall thickness, in.
E_k = kinetic energy of missile, ft-lb
d = diameter of missile, in.

The governing missile, a 1.75-inch-diameter control rod drive shaft with a maximum kinetic energy of 60,200 foot-pounds (shield located 4 feet above housing), requires a shield thickness of 1.31 inches. However, a 2.0 inch-thick shield was provided.

The pressurizer housing and other concrete structures were evaluated for perforation by a missile using (BRL) formula:

$$P = \frac{427}{(f_c')^{1/2}} \times \frac{W\,d^{1/5}}{d^2} \times \left(\frac{V}{1000}\right)^{4/3} \tag{3.5-12}$$

where:

P	=	required thickness of concrete slab, in.
fc'	=	compressive strength of concrete, psi
W, d, V	=	weight (lb), diameter (in.), and velocity (ft/sec) of missile, respectively

The most powerful missile among those listed in Tables 3.5-4 through 3.5-6 is the 3-inch motor-operated isolation valve bonnet, weighing 400 pounds and having a velocity of 135 ft/sec and a 28 square inch impact area. This missile could perforate 6.7 inches of 5000 psi strength concrete; the minimum thickness of the target is 24 inches.

In addition to the check for perforation, the overall structural response is evaluated by using the method of Reference 4. For this purpose, the penetration depth is calculated by using the modified Petry formula (Reference 2):

$$D = K_1 A_p V' \qquad (3.5\text{-}13)$$

where:

D	=	depth of penetration, ft
K_1	=	experimentally obtained material coefficient
A_p	=	weight to impact area ratio, lb/ft
V'	=	$\lg\left(1+\frac{V^2}{215000}\right)$ = velocity factor
V	=	missile velocity, ft/sec

The calculation procedure is illustrated below for a missile believed to produce the largest response:

Missile - 3-in. motor-operated isolation valve bonnet
Weight - W = 400 lb
Impact area - A = 28 in^2 = 0.195 ft^2
Velocity - V = 135 fps

$$K_1 = 2.82 \times 10^{-3} \ \text{ft}^3/\text{lb}$$

$$A_p = \frac{W}{A} = 2{,}050 \ \text{lb}/\text{ft}^2$$

$$D = 2.82 \times 10^{-3} \times 2{,}050 \ \text{Lg}\left(1 + \frac{135^2}{215{,}000}\right) = 0.205 \ \text{ft}$$

Because the velocity is assumed to reduce linearly from the initial value of V, the time t_1 of the impulse or the duration of the dynamic force F_1 is determined by the equation:

$$t_1 = \frac{2D}{V} = \frac{2 \times 0.205}{135} = 0.003 \ \text{sec} \qquad (3.5\text{-}14)$$

The dynamic force is calculated by the equation:

$$F_i = \frac{W V^2}{2gD} = \frac{0.4 \times (135)^2}{2 \times 32.2 \times 0.205} = 550 \ \text{kips} \qquad (3.5\text{-}15)$$

The factor K, by which the value of F_i can be reduced to determine an equivalent static load P, is given by the equation:

$$K = \frac{(2\mu - 1)^{0.5} T}{\pi t_1} + \frac{1 - \frac{0.5}{\mu}}{1 + \frac{0.7T}{t_1}} = \frac{(6-1)^{0.5}(0.03)}{p(0.003)} + \frac{1 - \frac{0.5}{3}}{1 + \frac{(0.7)(0.03)}{0.003}} = 7.22 \qquad (3.5\text{-}16)$$

where:

the ductility ratio μ = 3, and the structure's natural period T = 0.03 sec

The value of the static load is then:

$$P = \frac{550}{7.22} = 76 \ \text{kips} \qquad (3.5\text{-}17)$$

3.5.2.4.2 Loadings for Missiles Generated Outside the Containment

Although the original design of certain PG&E Design Class I reinforced concrete structures outside containment included consideration of potential missile penetration and spalling effects, subsequent analyses and evaluations have determined that the generation of main turbine missiles outside of the turbine casing are not credible events.

3.5.2.5 MISSILE BARRIER FEATURES

The structures designed to protect SSCs from the effects of missiles generated within the containment are:

(1) A missile shield is provided over the control rod drive mechanisms. The shield is located 4 feet above the control rod drive mechanism housing and consists of a primary shielding element made of 2-inch steel plate that is part of the integrated head assembly structure. Refer to Figure 3.5-1.

(2) The pressurizer housing, shown in Figure 3.5-2, consists of 12-inch- and 18-inch-thick reinforced concrete walls forming an irregular polygon and a 24-inch-thick concrete slab above the top of pressurizer. A 66-inch by 105-inch opening is provided in the top slab to prevent pressure buildup. The opening is covered with grating to preclude penetration by potential missiles.

(3) The polar crane wall, fuel transfer canal walls, and operating floor at elevation 140 feet are shown on the concrete outline drawings referenced in Section 3.8.2.

A missile shield barrier is provided over the auxiliary feedwater pump turbines. The Unit 1 missile shield is a barrel-vault design and is formed from 1-inch-diameter bars sandwiched between 1/2 x 2 x 25-inch plates as shown in Figure 3.5-6. Analysis of the missile shield indicates that it is capable of retaining the missile postulated in Section 3.5.2.3.2.1. The Unit 2 missile shield is formed from horizontal layers of 4 x 1/2-inch steel bars supported on 12-inch-thick concrete walls.

3.5.3 SAFETY EVALUATION

3.5.3.1 General Design Criterion 40, 1967 – Missile Protection

Protection for engineered safety features inside and outside of containment is provided against missiles that might result from plant equipment failures. Refer to Sections 3.5.2.1, 3.5.2.2, 3.5.2.3, 3.5.2.4, and 3.5.2.5.

3.5.3.2 Missile Protection Safety Function Requirement

(1) Protection From Missiles

PG&E Design Class I structures, systems and components, both inside and outside of containment, are protected against the effects of missiles, which may result from equipment failures and from events and conditions outside the nuclear power unit. Refer to Sections 3.5.2.1, 3.5.2.2, 3.5.2.3, 3.5.2.4, and 3.5.2.5.

3.5.4 TESTS AND INSPECTIONS

A routine LP rotor inspection program for DCPP Unit 1 and Unit 2 ensures that the integrity of LP rotors is maintained. The LP turbine rotors are inspected in order to identify cracks which may lead to missile generation. The inspection interval is based on maintaining a missile generation probability below the NRC guidance of 10^{-7} events per year. Section 3.5.2.2.2.1 discusses LP turbine inspection.

The turbine overspeed protection system helps ensure that the turbine does not accelerate to destructive overspeed should a failure occur. Section 3.5.2.2.2.1

discusses testing of the main steam intercept valves and other turbine overspeed protection devices.

3.5.5 REFERENCES

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11. R. L. Haessler, Final Update and Evaluation of BB-95/96 Turbine Valve Failure Data Base, WOG TUTF-00-014, October 16, 2000.

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13. NRC letter to PG&E, Leak-Before-Break Evaluation of Reactor Coolant System Piping for DCPP Units 1 and 2, from Sheri R. Peterson to Gregory M. Rueger, March 2, 1993.

14. Alstom Report: "Diablo Canyon – 1200 MW at 1800 RPM, LP-Rotor Missile Analysis", DC 6020487-37.

15. Alstom Report: "Diablo Canyon Missile Analysis: Customer Questions about Last Stage Blades", DC 6020487-36.

16. Alstom Study: "ND56R Geometrical Description and Mechanical Integrity of the L-1 Blade", DC 6020487-40.

17. Alstom Study: "ND56R Geometrical Description and Stress Calculations of the L-0 Blade", DC 6020487-41.

18. Alstom Study: "Diablo Canyon Stress Corrosion Cracking (SCC) Initiation at the Bottom of Rotor Grooves", DC 6020487-43.

19. Alstom Study: "Diablo Canyon – 1200 MW @ 1800 RPM; Stress Analysis of the LP-Rotors", DC 6020487-29.

3.5.6 REFERENCE DRAWINGS

Figures representing controlled engineering drawings are incorporated by reference and are identified in Table 1.6-1. The contents of the drawings are controlled by DCPP procedures.

3.6 PROTECTION AGAINST DYNAMIC EFFECTS ASSOCIATED WITH THE POSTULATED RUPTURE OF PIPING

Special measures have been taken in the design and construction of the plant to protect the public against the consequences of dynamic effects associated with postulated piping ruptures both inside and outside the containment. The plant is designed so that a postulated piping failure will not cause the loss of needed functions of safety-related systems and structures, and so that the plant can be safely shut down in the event of such failure.

For moderate energy systems, protection from the jet spray and flooding effects due to critical cracks is incorporated into the design. This section presents the design bases and design measures established for DCPP for protection against these dynamic effects in conformance with 10 CFR 50, Appendix A, GDC 4.

3.6.1 SYSTEMS IN WHICH DESIGN BASIS PIPING BREAKS OCCUR

3.6.1.1 High-Energy Piping Inside Containment

The following systems have been evaluated with regard to the dynamic effects of pipe whip and blowdown reactive forces associated with a ruptured pipe:

(1) Reactor coolant system (RCS)

- (a) Primary reactor coolant loops (see Section 3.6.2.1.1.1)
- (b) Pressurizer surge line
- (c) Pressurizer spray line
- (d) Pressurizer relief and safety valve lines
- (e) Drains greater than 1-inch in diameter

(2) Chemical and volume control system (CVCS)

- (a) Charging line and auxiliary spray line
- (b) Reactor coolant pumps seal water injection
- (c) Letdown line
- (d) Excess letdown line
- (e) Reactor coolant pumps - seal vent and leakoff, greater than 1 inch in diameter

(3) Safety injection system (SIS)

(a) Accumulator injection lines

(b) Safety injection lines

(4) Residual heat removal (RHR) system

(a) Residual heat removal supply

(b) Residual heat removal return

(5) Turbine steam supply system

(a) Main steam lines

(b) Feedwater lines

(c) Steam generator blowdown lines

3.6.1.2 High-Energy Piping Outside Containment

The following criteria and definitions apply to the selection of high-energy piping systems outside containment for evaluation of the dynamic effects associated with postulated pipe rupture:

(1) (a) All systems having a service temperature greater than 200°F or an operating pressure greater than 275 psig are considered

(b) Open crack breaks are postulated to occur in the most adverse locations in piping having fluid temperature or pressure greater than the above

(c) Design basis breaks, in addition to crack breaks, are postulated in those portions of high-energy systems where both temperature and pressure exceed these levels

(d) The criteria for determining the location of design basis breaks are defined later in this section

(2) Piping that is either encased in concrete or protected by barriers from safety-related SSCs is considered to be adequately designed

(3) Piping that is physically located so that unrestrained motion (pipe whip) could occur in any direction about a plastic hinge formed after a pipe

rupture, but that could not impact any safety-related structure, component, or system is considered to be adequately designed

The systems that contain high-energy lines located outside of the containment in which both open crack and design bases breaks are postulated to occur are:

(1) Condensate system

(2) Feedwater system

(3) Turbine steam supply system (main steam, steam generator blowdown system, and auxiliary feedwater turbine steam supply piping up to stop valve FCV 95)

(4) Extraction steam and heater drip system

(5) Chemical and volume control system

(6) Turbine and generator associated systems

Open crack breaks are also postulated in the auxiliary steam system, as it contains piping at a temperature that exceeds 200°F and in the auxiliary feedwater piping (pressure greater than 275 psig).

No design basis breaks or crack breaks are postulated downstream of stop valve FCV-95 in the auxiliary feedwater pump turbine steam supply because it is not pressurized during startup, shutdown, or normal plant operating conditions.

3.6.1.3 Moderate-Energy Piping

Through-wall cracks are postulated to occur in moderate energy systems, i.e., any pipe larger than 1-inch in diameter with fluid temperature less than or equal to 200°F and pressure less than or equal to 275 psig, to determine the effect of resulting spray or flooding on safety-related equipment.

The RHR system, and its associated normally pressurized branch piping is analyzed as a moderate-energy system because the RHR system is only a high-energy system for a very small period of time. The system is also seismic Design Class I, dual-purpose, designed and constructed to meet ANSI B31.7, Class II requirements.

3.6.2 DESIGN BASIS PIPING BREAK CRITERIA

3.6.2.1 General Criteria

3.6.2.1.1 High-Energy Piping Inside Containment

The containment and all essential equipment within the containment, particularly components of the reactor coolant pressure boundary and other safety-related components, are protected against the dynamic and environmental effects of pipe whip resulting from postulated rupture of piping. The criteria for minimizing these effects are described in Section 3.6.5. In March 1993, the DCPP leak-before-break evaluation (Reference 11) was accepted by the NRC (Reference 12), so that the dynamic effects associated with breaks in the main reactor coolant loop piping are no longer part of the DCPP design basis and no longer need to be protected against, although protection from the dynamic effects of RCS branch line and other high energy line breaks must still be provided (see Section 3.6.2.1.1.1).

Engineered safety features (ESFs) are provided for core cooling and boration, pressure reduction, and activity confinement in the event of a loss of reactor coolant or a steam or feedwater line break accident, to ensure that the public is protected in accordance with 10 CFR 100 guidelines. These safety systems have been designed to provide protection for an RCS pipe rupture of a size up to and including a double-ended severance of an RCS main loop.

3.6.2.1.1.1 Reactor Coolant System Main Loop Piping (Leak-Before-Break)

In November 1984, the NRC issued NUREG-1061 assessing the applicability of leak-before-break analysis to nuclear power plant piping systems. Effective May 1986, the NRC revised 1971 General Design Criterion 4 to allow the use of leak-before-break methodology for excluding the dynamic effects of postulated ruptures in reactor coolant loop piping in PWRs from the design basis. A draft revision to Standard Review Plan 3.6.3 was subsequently issued outlining the scope of the plant specific evaluation required to obtain NRC acceptance of the use of leak-before-break exclusion. Westinghouse performed the required evaluation for the DCPP main reactor coolant loops (Reference 11), and PG&E submitted the evaluation to the NRC on March 16, 1992, requesting elimination of the dynamic effects of postulated ruptures in the main reactor coolant loop piping from the DCPP design basis. On March 2, 1993, the NRC accepted the analysis and granted permission to eliminate the dynamic effects of those breaks from the DCPP design basis.

The scope of application of the DCPP leak-before-break exemption is limited in nature. It applies to the dynamic effects of breaks in the main reactor coolant loop piping only, and can be used only for purposes of exempting consideration of the dynamic loads resulting from such breaks in the equipment and structural design bases, and for exempting consideration of the dynamic effects of those breaks in the protection of equipment. It applies to the main RCS loop piping only, and does not apply to breaks in branch lines off the main loop lines. Hence, the effects of pipe whip, pipe break reaction forces, jet impingement, decompression waves within the ruptured pipe, missile generation, and dynamic subcompartment pressurization due to main loop piping breaks no longer have to be considered in the DCPP design basis (Reference 13); however, those effects resulting from branch line breaks still must be considered. The inclusion of pipe whip restraints and jet impingement barriers in the design of the plant to protect equipment in containment from the effects of pipe whip and jet impingement resulting from main reactor coolant loop pipe breaks is no longer required. In addition, dynamic LOCA loads resulting from main reactor coolant loop breaks now need not be included in the dynamic load combinations for equipment and structural analyses, although those loads resulting from branch line breaks must still be included. Snubbers on steam generators and reactor coolant pumps whose only design function is the mitigation of thrust loads associated with main reactor coolant loop pipe breaks are no longer required. However, the use of the leak-before-break exemption is specifically not allowed for purposes of accident analysis or for purposes of environmental qualification of equipment. Static post-LOCA peak containment pressure resulting from main RCS loop breaks must still be included in load combinations where that pressure is a consideration in the structural design. The ECCS and ESF systems must still be capable of mitigating the effects resulting from the original design basis double-ended guillotine breaks in the main reactor coolant loops, and the equipment required to mitigate a LOCA must still be environmentally qualified to the conditions resulting from such breaks.

The acceptability of the leak-before-break evaluation is based on a number of reactor coolant system design and operating characteristics which are expected to remain fairly constant over the life of the plant and which are controlled by procedures and licensing and design basis documents. These include main coolant loop piping materials and mechanical properties, as-built configuration, assumed seismic loads and load combinations, reactor coolant system chemistry, and reactor coolant system operating parameters. To maintain the validity of the leak-before-break evaluation, these characteristics should not change significantly in a nonconservative direction. If such a change occurs or is made by design or procedure change, its effect on the assumptions made in the leak-before-break analysis should be evaluated. The leak-before-break analysis also assumes that the DCPP reactor coolant system leak detection system has the capability to detect an increase in reactor coolant system leakage into the containment of 1 gpm. The current design basis for this system indicates that it has this capability (see Section 5.2.7). Operability of this system is controlled by the plant Technical Specifications.

As a part of the Unit 2 Steam Generator Replacement Project, the reactor coolant loop piping was reanalyzed, which resulted in new reactor coolant loop loads. These new loads, along with steam generator replacement related temperature changes, were used to determine the impact on the existing DCPP leak-before-break analysis. The results determined that the leak-before-break recommended margins remained satisfied. Therefore, the conclusions reached in the existing Reference 11 report remain valid with the replacement steam generator installation.

3.6.2.1.1.2 Reactor Coolant System Connected Piping

The piping connections to the primary reactor coolant loops fall into the general categories illustrated in Figure 3.6-1. These categories are defined by the direction of flow to or from the primary reactor coolant loops and by the associated valve configuration. A rupture of these lines conceivably could cause uncontrolled loss of reactor coolant depending on the precise location of the break and the line configuration.

In establishing the dynamic effects criterion, uncontrolled loss of reactor coolant is assumed to occur for a pipe break out to the restraint of the second automatic isolation valve (Case II, Figure 3.6-1) on outgoing lines, and out to and including the second check valve on incoming lines normally with flow (Case III, Figure 3.6-1). This criterion takes credit for only one of the two valves performing its intended function. However, for the letdown line pipe break analysis, credit for successful closure of isolation valves LCV-459 and LCV-460 is not required (they would be considered to be fail-as-is valves in Figure 3.6-1, Case II). The HELB analysis showed that accident mitigation and plant shutdown could be successfully accomplished for a letdown line break downstream of these valves even in the event they should not close. For normally closed isolation or incoming check valves (Cases I and IV, Figure 3.6-1), uncontrolled loss of reactor coolant is assumed to occur for pipe breaks on the reactor side of the valve.

It is assumed that a break of the piping associated with ESFs does not occur during the injection phase following a loss of coolant. During the recirculation phase, a leak or equivalent break resulting in a maximum flow of 50 gpm is assumed. This value is based on the flow that could result from the complete failure of a RHR pump seal. The radiation dose analysis for this seal failure is discussed in Section 15.5.

3.6.2.1.1.3 Other High-Energy Piping Inside Containment

Breaks were postulated at all terminal ends and all high stress points in accordance with RG 1.46 (Reference 1). Subsequent to receipt of NRC approval on March 13, 1987, break locations were postulated based on stress level as recommended in NRC BTP MEB 3-1 (Reference 8). Subsequent to the issuance of NRC Generic Letter 87-11 on June 19, 1987, break locations are postulated as recommended in BTP MEB 3-1 Rev. 2 (Reference 9).

3.6.2.1.2 High-Energy Piping Outside Containment

The criteria that apply to the evaluation of the dynamic effects associated with postulated high-energy pipe rupture outside the containment are in accordance with those given in References 4, 5, and 6.

3.6.2.1.3 Moderate-Energy Piping

The moderate-energy lines were reviewed to determine the effects of an assumed leak at the worst location with respect to the equipment required for safe shutdown. The effects of water spray on the equipment and potential compartment flooding were considered.

3.6.2.2 Specific Criteria

The specific criteria for determining pipe break location, type, and area are discussed below.

3.6.2.2.1 Primary Reactor Coolant Piping

3.6.2.2.1.1 Locations Where Design Basis Piping Breaks Are Postulated to Occur

To provide integrity and design adequacy of the primary reactor coolant loop piping and equipment supports in the event of a highly improbable pipe rupture accident, a number of pipe rupture locations were postulated in the original design. The primary reactor coolant loop was analyzed for the design pipe breaks listed in this section and shown in Figure 3.6-4. However, as discussed in Section 3.6.2.1.1.1 above, due to the acceptance of the DCPP leak-before-break evaluation by the NRC (Reference 12), the dynamic effects of breaks in the main reactor coolant loop piping no longer have to be analyzed; only the effects from RCS branch line breaks have to be considered.

The piping of the reactor coolant loops was designed to ANSI B31.1. Design was completed prior to the issuance of RG 1.46 and the nuclear piping Codes B31.7 and the ASME Code III, to which the break criteria of the Westinghouse Report WCAP-8082 (Reference 3) specifically apply. Consequently, these documents were not available when the discrete break locations for the reactor coolant loop (RCL) were determined. However, a comparison of the postulated break locations for the RCL and those of WCAP-8082 shows that the break locations are similar and provide protection equivalent to the criteria of RG 1.46. (Again, however, due to the acceptance of the DCPP leak-before-break evaluation by the NRC, the dynamic effects of breaks in the main reactor coolant piping no longer have to be analyzed; only the effects from RCS branch line breaks have to be considered. Since the breaks postulated for the original analyses are more severe than those that are now required to be considered, the original analyses are conservative.)

These discrete break locations and types were determined by an engineering approach that employed, as its basis, stress and fatigue analyses, system considerations, operational characteristics, and loading conditions. The breaks in the hot and cold legs were placed in the straight run outside of the primary shield wall. These circumferential breaks were chosen so as to allow full double-ended pipe separation and full discharge flow rather than limited area breaks and limited flow which would be obtained from a break inside the shield wall or at the reactor vessel nozzles. These RCL break locations were chosen for the piping dynamic analysis. The criteria used to determine the break locations on the RCL provided equivalent conservatism and result in equivalent protection to the criteria of RG 1.46. The dynamic analyses of the primary RCL piping and equipment supports for each of these break locations provided assurance of the protection of public health and safety. (Again, however, due to the acceptance of the DCPP leak-before-break evaluation by the NRC, the dynamic effects of breaks in the main reactor coolant piping no longer have to be analyzed; only the effects from RCS branch line breaks have to be considered. Since the breaks postulated for the original analyses are more severe than those that are now required to be considered, the original analyses are conservative.)

3.6.2.2.1.2 Types of Breaks and Break Areas Assumed for Postulated Primary Coolant Loop Failure for Piping Dynamic Analysis

The types of breaks assumed in the original analysis for postulated primary coolant loop failure for the piping dynamic analysis included:

(1) Straight portion of hot leg piping - guillotine

(2) Straight portion of cold leg piping - guillotine

(3) Steam generator inlet nozzle - guillotine

(4) Steam generator outlet nozzle - guillotine

(5) Reactor coolant pump inlet nozzle - guillotine

(6) 50° elbow - longitudinal

(7) Flow entrance to the 90° elbow - guillotine

(8) RHR primary loop connection - guillotine

(9) Safety injection/primary coolant loop connection - guillotine

(10) Pressurizer surge/primary coolant loop connection - guillotine

(11) Loop closure weld in crossover leg - guillotine

The break length for the longitudinal breaks was considered to be equal to two pipe diameters. For the breaks listed in this section, the break area is conservatively assumed to be equal to the cross-sectional area of the pipe. However, as discussed in Section 3.6.2.1.1.1 above, due to the acceptance of the DCPP leak-before-break evaluation by the NRC (Reference 12), the locations, types, and areas of breaks in the main reactor coolant loop piping no longer have to be defined for purposes of performing dynamic effects analyses. As a consequence, only branch line breaks (8), (9), and (10) above remain within the design basis for dynamic effects and require dynamic effects analyses.

A break at each of the reactor vessel inlet and outlet nozzles was postulated in addition to those listed above. These breaks were assumed to be limited in area by the primary shield wall restraints. The purpose of postulating breaks at these locations was to determine mass and energy release information for use in the evaluation of the asymmetric pressure loading on the reactor vessel. Under the leak-before-break exemption, these breaks and the resulting asymmetric pressure loadings no longer need to be considered.

3.6.2.2.2 Other High-Energy Piping Inside Containment

Breaks were postulated at all terminal ends and all high stress points in accordance with RG 1.46. Subsequent to receipt of NRC approval on March 13, 1987, break locations were postulated based on stress level as recommended in NRC BTP MEB 3-1. Subsequent to the issuance of NRC Generic Letter 87-11 on June 19, 1987, break locations are postulated as recommended in BTP MEB 3-1 Rev. 2.

3.6.2.2.3 High-Energy Piping Outside Containment

The selection of design basis breaks in each of the systems identified in Section 3.6.1.2 is generally based on results of the piping stress analyses. These analyses consider effects of pressure, deadweight, thermal expansion during normal operation, upset and test conditions, and the DE. Subsequent to receipt of NRC approval on March 13, 1987, break locations were postulated based on stress level as recommended in BTP MEB 3-1.

Subsequent to the issuance of NRC Generic Letter 87-11 on June 19, 1987, break locations are postulated as recommended in BTP MEB 3-1 Rev. 2. Where such stress analyses were not available, the breaks were postulated to occur at such fittings that could result in the most severe consequences in such systems. Crack breaks were postulated to occur in the most adverse orientations and locations throughout the piping.

3.6.2.2.4 Moderate-Energy Piping

Through-wall cracks were assumed to have an area equal to 1/2d x 1/2t, where "d" is the nominal pipe diameter and "t" is the nominal pipe wall thickness.
The control room ventilation and pressurization system is not required to be protected from moderate-energy pipe breaks from either unit. In the event that an MELB occurs, and the control room becomes uninhabitable due to the impairment of the HVAC system, safe shutdown can be controlled from the hot shutdown remote control panel.

3.6.3 DESIGN LOADING COMBINATIONS

3.6.3.1 High-Energy Piping Breaks Inside Containment

The design loading combinations, the design condition categories, and design stress limits applied to components, supports, and pipe whip restraints of essential components and piping of high-energy fluid systems within the containment, are described in Sections 3.9 and 5.2. Section 5.2 addresses Class A components while Section 3.9 addresses the remaining components. A discussion of potential missiles is presented in Section 3.5.

In the original design, to achieve an adequate primary RCL design, a dynamic analysis, as described in Section 3.6.4.1, was performed on the RCL/support system described below for the pipe break cases discussed in Section 3.6.2.2.1.2 to determine component and component support loadings. However, as discussed in Section 3.6.2.1.1.1 above, due to the acceptance of the DCPP leak-before-break evaluation by the NRC (Reference 12), dynamic analyses to determine the RCS component and component support loadings resulting from breaks in the main reactor coolant loops are no longer separately required. Only dynamic analyses for RCS branch line breaks are still required. The support structure design analyses discussed in the following three subsections were based on the original design requirements, and do not reflect the subsequent leak-before-break exemption. Since the breaks postulated for the original analyses are more severe than those that are now required to be considered, the original analyses are conservative.

3.6.3.1.1 Reactor Vessel Support Structure

The reactor vessel support structure is designed to resist thrusts that are considered to originate from the following three sources: (a) the reactions of the blowdown forces in the primary RCL piping that are eventually transmitted to the reactor nozzles, (b) the forces within the reactor pressure vessel shell acting on the reactor internals and shell wall, and (c) the effects of reactor vessel cavity asymmetric pressurization.

The superposition of these effects, in time-history form, permits accurate determination of the loads transmitted to the reactor vessel support structure. The design and details of the reactor vessel support structure are further discussed in Section 5.5.13.

3.6.3.1.2 Steam Generator Support Structure

The steam generators are supported in a manner that allows for thermal expansion of the equipment from cold to operating condition. Each steam generator is supported such that the rupture of steam, feedwater, blowdown, or instrument piping as a result of thrust forces created by the rupture of a primary RCL pipe is prevented. The steam generators are also supported in a manner that prevents rupture of a primary RCL pipe as a result of thrust forces created by the rupture of a steam or feedwater line.

Guides and restraints are employed, where required, to limit the motion of the steam generators under the reaction forces that result from a primary RCL pipe break, to a distance that is compatible with the flexibility of the steam and feedwater piping. Also, the motion of the steam generators, under the reaction forces due to a steam or feedwater pipe break, is limited to a distance that is compatible with the flexibility of the primary reactor coolant piping. The design and details of the steam generator support structures are further discussed in Section 5.5.13.

3.6.3.1.3 Reactor Coolant Pump Support Structure

Each reactor coolant pump is supported in a manner that would limit its displacement short of the primary shield, secondary shield, steam generator, steam generator supports, equipment and piping in adjacent loops, and hot leg of the affected loop as a result of a rupture occurring in either the pump suction or discharge piping. The design and details of the reactor coolant pump support structures are further discussed in Section 5.5.13.

3.6.3.2 High-Energy Piping Breaks Outside Containment

Piping response analyses are performed on high-energy piping systems at those postulated break locations for which unrestrained pipe motion about a plastic hinge could impact or endanger vital systems. Factors and criteria that are considered are:

(1) The dynamic nature of the loading.

(2) Pipe impact effects due to gaps in piping restraints.

(3) Nonlinear (elastic-plastic) pipe and restraint material properties and the effect of rapid strain rate on material properties.

(4) For circumferential breaks in a pipe, whip occurs upon attainment of 50 percent of uniform ultimate strain at a plastic hinge due to loading from the blowdown reactive forces. The pipe whip is characterized by unrestrained motion of the pipe about the hinge in the direction governed by the vector thrust of the break force.

(5) For longitudinal breaks, failure occurs upon attainment of a hinge mechanism with 50 percent of uniform ultimate strain on each hinge.

(6) Lower-bound piping material properties are used for the prediction of pipe whips.

(7) Both lower- and upper-bound piping material properties are used for the prediction of loads on anchors and restraints.

(8) Piping loads on the rupture restraints are limited to the equivalent of 50 percent of uniform ultimate strain in the restraint materials. The nonlinear material properties of the restraint are considered.

(9) The load combinations and allowable limits used in evaluating Design Class I concrete structures for the effects of the postulated high-energy pipe breaks are discussed in Section 3.8.

(10) Load combinations and allowable limits used in evaluating those Design Class I steel structures outside the containment whose function is to provide protection against the effects of the postulated high-energy pipe break are discussed Section 3.8.

3.6.4 DYNAMIC ANALYSES

The dynamic effects of pipe breaks described previously in Sections 3.6.1 and 3.6.2 were analyzed using the methods described below.

3.6.4.1 Reactor Coolant Loop Piping Breaks

In the original design, the primary coolant loop pipe break hydraulic analysis described below in Sections 3.6.4.1.1, 3.6.4.1.2, and 3.6.4.1.3 was performed to determine the resulting dynamic thrust and jet forces and dynamic asymmetric pressure loadings. However, as discussed in Section 3.6.2.1.1.1 above, due to the acceptance of the DCPP leak-before-break evaluation by the NRC (Reference 12), the dynamic effects of breaks in the main reactor coolant loop piping no longer have to be considered in the design basis analyses. Analyses such as those described below are now only required for RCS branch line breaks. Since the breaks postulated for the original analyses are more severe than those that are now required to be considered, the original analyses are conservative.

3.6.4.1.1 Thrust-Time Relationship

The blowdown forces caused by a rupture of a primary RCL pipe include the specific jet thrust at the break location and the internal hydraulic forces resulting from the acceleration of the fluid within the broken and unbroken loops.

Hydraulic forcing functions are calculated for the ruptured and intact RCLs as a result of a postulated LOCA. These forces result from the transient flow and pressure histories in the RCS. The calculation is performed in two steps. The first step is to calculate the transient pressure, mass flow rates, and thermodynamic properties as a function of time. In the second step, the results obtained from the hydraulic analysis, along with input of areas and direction coordinates, are used to calculate the time-history of forces at appropriate locations in the RCLs.

The analysis is performed on integrated analytical models including the steam generator and reactor coolant pump, the associated supports and restraints, and the attached piping. An elastic-dynamic, three-dimensional model of the reactor coolant loop is constructed. The boundary of the analytical model is, in general, the interface between the foundation concrete and the support structure. The deformation of the reinforced concrete foundation supports is considered, where applicable to the RCL model.

The steps in the analytical method are:

(1) The initial deflected position of the RCL model is defined by applying the initial steady state condition of the unbroken reactor coolant loop model.

(2) Natural frequencies and normal modes of the broken loop are determined.

(3) The initial deflection, natural frequencies, normal modes, and time-history forcing functions are used to determine the time-history forcing dynamic deflection response of the lumped mass representation of the RCL.

(4) The forces imposed on the equipment supports and restraints by the loop are obtained by multiplying the support stiffness matrix and the time-history of the displacement vector at the support point.

(5) The time-history dynamic deflections at mass points are treated as an imposed deflection condition on the ruptured loop RCL model and internal forces, deflections, and stresses at each end of the members on the RCL piping systems are computed.

The results are used to verify the adequacy of the piping, equipment supports, and restraints.

The hydraulic model represents the behavior of the coolant fluid within the RCS. Key parameters calculated by the hydraulic model are pressure, mass flow rate, and density. These parameters are supplied to the thrust calculation, together with appropriate plant layout information, to determine the time-dependent loads exerted by the fluid on the loops. In evaluating the hydraulic forcing functions during a postulated LOCA, the pressure and momentum flux terms are dominant. The inertia and gravitational terms are taken into account in evaluation of the local fluid conditions in the hydraulic model.

The blowdown hydraulic analysis provides the basic information concerning the dynamic behavior of the reactor core environment for the loop forces, reactor kinetics, and core cooling analysis. This analysis requires the ability to predict the flow, quality, and pressure of the fluid throughout the reactor system.

The MULTIFLEX 3.0 computer code performs a comprehensive, space-time dependent analysis of a LOCA and is designed to treat all phases of the blowdown. The stages are: (a) a subcooled stage where the rapidly changing pressure gradients in the subcooled fluid exert an influence upon the RCS internals and support structures, and (b) a two-phase depressurization stage, and (c) a saturated stage.

The code employs a one-dimensional analysis in which the entire RCS is divided into control volumes. The fluid properties are considered uniform and thermodynamic equilibrium is assumed in each element. Pump characteristics, pump coastdown and cavitation, and core and steam generator heat transfer, including the W-3 DNB correlation, in addition to the reactor kinetics, are incorporated in the code. The MULTIFLEX code is described and referenced in Section 5.2.1.10.2.

The blowdown hydraulic loads on primary loop components are computed from the following equation:

$$F = 144A \left[\left(P - 14.7 \right) + \frac{\left(\dot{m} \right)^2}{\left(144\, \rho\, g\, A_m^2 \right)} \right] \quad (3.6\text{-}1)$$

which includes both the static and dynamic effects. The symbols and units are:

F	=	force, lb_f
A	=	aperture area, ft^2
P	=	system pressure, psia
$\dot{m}$	=	mass flow rate, lb/sec
ρ	=	Density, lb_m/ft^3
g	=	gravitational constant = 32.174 ft/sec^2
A_m	=	mass flow area, ft^2

In the model to compute forcing functions, the main RCL system is represented by a similar model as employed in the blowdown analysis. The entire loop layout is described in a global coordinate system. Each node is fully described by: (a) blowdown hydraulic information, and (b) the orientation of the steamlines of the force nodes in the system, which includes flow areas and projection coefficients along the three axes of the global coordinate system. Each node is modeled as a separate control volume with one or two flow apertures associated with it. Two apertures are used to simulate a change in flow direction and area. Each force is divided into its x, y, and z components using the projection coefficients. The force components are then summed over the total number of apertures in any one node to give a total x force, total y force, and total z force. These thrust forces serve as input to the piping/restraint dynamic analysis.

The dynamic analysis of RCLs employs displacement method, lumped parameter, and stiffness matrix formulation and assumes that all components behave in a linear elastic manner.

3.6.4.1.2 Jet Dynamic Force

A jet dynamic force will result from any of the pipe breaks postulated above. The force, caused by the momentum change of fluid flowing through the break, is a function of the upstream fluid conditions, fluid enthalpy, source pressure, pipe flow restrictions, friction, and dimensions. Structural barriers and physical separation by plant layout have been used in the design to limit the effects of impingement. Where necessary, the jet forces resulting from the pipe break have been computed using the following method:

Jet dynamic forces on structures are calculated:

$$F_j = C_j\,(1.26\,PA) \qquad (3.6\text{-}2)$$

where:

F_j	=	jet dynamic force acting on a structure
Cj	=	factor to account for the dynamic nature of the load. In determining the value of Cj, inelastic behavior is assumed
P	=	system operating pressure
A	=	cross-sectional area of pipe

The above loads were considered in the structural design as described in Section 3.8.

3.6.4.1.3 Asymmetric Pressure Loading

Pressure differentials may develop between structural subcompartments as a result of reactor coolant pipe breaks. Evaluation of these pressure differentials is given in Section 6.2.1.

3.6.4.2 Other High-Energy Piping Breaks Inside Containment

Blowdown forces and jet impingement forces due to the postulated piping breaks in lines inside containment (other than the RCLs) were calculated from the formula:

$$F_B = C_T\,PoA \text{ (see Reference 2)} \qquad (3.6\text{-}3)$$

where:

F_B	=	steady state blowdown force, lb
CT	=	steady state thrust coefficient
	=	2.0 for subcooled water, or 1.26 for saturated steam/water (see Reference 2)
Po	=	line pressure at time 0
A	=	cross-sectional flow area of pipe with a full-area break

Moments required to form a plastic hinge were calculated from the formula:

$$M_p = \frac{K S_y I}{R_o} \qquad (3.6\text{-}4)$$

where:

M_p = plastic moment
K = M_p/M_y where M_y is the moment to produce yielding on the extreme fiber
S_y = yield stress of material at temperature
I = moment of inertia of piping = $\pi/4 \ (R_o^4 - R_i^4)$
R_o = outside radius of pipe
R_i = inside radius of pipe
K = 2.5 for materials in the piping systems

3.6.4.3 High-Energy Piping Breaks Outside Containment

Analysis to determine the effects of a rupture of high-energy piping breaks outside containment was originally reported in Reference 7. Pipe break effects analyzed include pipe whip, jet impingement, compartment pressurization, water flooding, and the environmental effects of pressure, temperature, and humidity.

The analyses on pipe whip, jet impingement, and water flooding were reverified to reflect the current as-built condition, and modifications were made as necessary. In this reanalysis, in addition to the method used in Reference 5 of Reference 7, the methods as described in ANSI/ANS 58.2 are also used to verify the jet impingement temperature.

The analyses on compartment pressurization and the environmental effects of pressure, temperature and humidity have been re-done and the results of these analyses totally supersede the analyses as reported in Reference 7. These analyses and their results are reported in Sections 3.6.4.3.2 and 3.6.4.3.3. Because of the similarity of Units 1 and 2, these results can also apply to Unit 2.

3.6.4.3.1 Blowdown Forces

Fluid blowdown thrust time-histories resulting from a pipe rupture are determined using PRTHRUST, a computer code derived from RELAP3. The assumptions used for these analysis together with representative mathematical models and typical results are presented in Reference 7.

Pipe whip analyses of the main steam and feedwater piping, between the containment and the turbine building, resulting from ruptures at the identified locations were determined using computer program PIPERUP. PIPERUP determines the nonlinear, elastic-plastic response of three-dimensional piping restraint systems to the fluid blowdown force time-histories defined above. Gaps between the piping and rupture restraints, as well as nonlinear properties of the restraints are included in the analysis.

A description of the analytical methods used in the analyses, mathematical models of the piping systems, and representative results are also presented in Reference 7.

3.6.4.3.2 Compartment Pressurization

A definition of the worst case overpressure condition is required for the areas subjected to high-energy line breaks in order to evaluate the structural capability of essential structures required to maintain their integrity following the break. The potentially affected areas of the plant were modeled as compartments connected by vent paths that communicate with the break compartment in the multicompartment computer program FLUD (PCFLUD) or GOTHIC. This program calculates the conditions that will exist in the compartments over the duration of release of mass and energy from the postulated break. Figures 3.6-5 through 3.6-15 illustrate the compartment designations for the various models used in the analysis. Because of the similarity between Units 1 and 2, the compartment designations also apply to Unit 2.

3.6.4.3.2.1 Main Steam and Feedwater Piping

The controlling design basis break locations in the main steam piping between the containment and turbine stop valves in regard to compartment overpressure are as follows:

(1) Auxiliary building, Area GW, containment penetration

(2) Auxiliary building, Area GW, G-line anchor

(3) Turbine building, Area D, elevation 85 feet

(4) Turbine building, Area D, elevation 140 feet

Mass and energy release were calculated for each of the above breaks assuming the worst case operating conditions with the flow constrictions at the containment penetration and G-line anchor considered in addition to the available means of detection and automatic isolation. The consequent pressurization of compartments was determined and the results are illustrated in Figures 3.6-16 through 3.6-19. Pressurization effects due to feedwater line breaks are bounded by the above results for the main steam lines.

3.6.4.3.2.2 Other High-Energy Piping

The controlling break locations for other high-energy lines are as follows:

(1) Chemical and volume control system (DER in letdown line)

- Auxiliary building, Area K, letdown heat exchange room

(2) Auxiliary steam supply system (crack)

- Auxiliary building, Area J, auxiliary feedwater pump rooms

- Auxiliary building, Area K, elevation 100 feet open area

(3) Turbine steam supply system (DER in line 593)

- Auxiliary building, Area L, radiation monitor room

Mass and energy release were calculated for each of the above breaks assuming the worst case operating conditions and considering the available means of detection and isolation system. In the case of the CVCS break, an automatic detection/isolation system as described in Section 7.6 is utilized to minimize the release. The consequent pressurization of compartments was determined and the results are illustrated in Figures 3.6-20 through 3.6-26. Pressurization effects due to other high-energy line breaks in Area GE/GW and the turbine building are bounded by the results for the main steam line breaks.

3.6.4.3.3 Environmental Conditions

A definition of the worst case environmental conditions of temperature and humidity is required for the area subjected to the high-energy line breaks to evaluate essential safety-related equipment required to shutdown the plant following the break.

3.6.4.3.3.1 Main Steam and Feedwater Piping

The controlling break locations in the main steam piping between the containment and turbine stop valves for consideration of adverse environmental effects are as follows:

(1) Auxiliary building, Area GW, elevation 115 feet (crack break)

(2) Turbine building, Area D, elevation 85 feet (DER)

(3) Turbine building, Area D, elevation 140 feet (crack break)

Mass and energy release were calculated for each of the above breaks assuming the worst case operating conditions. The consequent compartment temperature and relative humidity were determined using the computer program FLUD (PCFLUD) or GOTHIC. Bounding temperature profiles for the affected areas are illustrated in Figures 3.6-28 through 3.6-33. The maximum relative humidity of 100 percent was applied in the evaluation of essential safety-related equipment. New MSLB analyses accounting for the elimination of the boron injection tank (BIT) utilized the computer program LOFTRAN to determine the mass and energy releases in the GE/GW compartment. Bounding temperature profiles for the RSGs were calculated using the computer program GOTHIC and are summarized in the table below:

Compartment	Peak Temperature, °F[a]
GW 115 ft	453.7
GE 115 ft	329.1
GW 100 ft	356.5
GE 100 ft	306.3
GW 85 ft	261.7
Pipeway	425.5

[a] For clarity, only peak instantaneous temperature, as determined over the analysis time duration (2000 second maximum time duration), for the limiting case for each analyzed compartment, is included herein. Complete analysis results, including pertinent temperature profiles for each compartment for each of the 60 base case scenarios included in the analysis, reside in applicable PG&E calculations.

3.6.4.3.3.2 Other High-Energy Piping

The controlling break locations for the other high-energy lines for consideration of adverse environmental effects are those identified in Section 3.6.4.3.2.2. The consequent temperature and relative humidity in the affected compartments were determined in the FLUD analysis. Bounding temperature profiles for the affected areas are illustrated in Figures 3.6-34 through 3.6-44. The maximum relative humidity of 100 percent was applied in the evaluation of essential safety-related equipment.

3.6.5 PROTECTIVE MEASURES

3.6.5.1 Piping Breaks

3.6.5.1.1 High-Energy Piping Breaks Inside Containment

It is essential that the equipment support structures (reactor pressure vessel, steam generator, and reactor coolant pump) be protected from the impact of large whipping pipes or be designed to resist such impact. This protection is accomplished by separation of equipment and piping, or by providing pipe restraints to prevent the formation of a plastic hinge mechanism. With the acceptance of the DCPP leak-before-break analysis by the NRC (Reference 12), protection of equipment and support structures from the whipping of main reactor coolant loop piping is no longer required, although protection from the dynamic effects of RCS branch line and other high energy line breaks must still be provided. If any branch pipes are supported from equipment support structures, the reaction force resulting from a rupture of these lines is considered in designing the equipment supports. Small pipes are assumed to cause no significant damage to equipment supports.

To ensure the continued integrity of the vital components and the engineered safety systems, consideration is given to the consequential effects of the pipe break itself in order to meet the following criteria:

(1) The minimum performance capabilities of the engineered safety systems must not be reduced below that required to protect against the postulated break.

(2) The containment[a] leaktightness must not be decreased below the design value, if the break leads to a loss of reactor coolant.

(3) An RCS pipe break must not cause a steam-feedwater system pipe break and vice versa.

The fluid discharge from ruptured piping would produce reaction and thrust forces in the piping systems. The effects of these forces have been considered in ensuring that the general criteria and performance of engineered safety systems are satisfied.

In addition to the three criteria on the consequential effects of the pipe break itself, as given above, propagation of damage must be limited in type and/or degree as follows:

(1) A pipe break that is not a loss of reactor coolant must not cause a loss of reactor coolant, or a steam or feedwater line break.

(2) Branch lines connected to the RCS are defined as "large" if they have an inside diameter greater than 4 inches.

Large piping is restrained so that:

(a) propagation of the break to the unaffected loops is prevented

(b) propagation of the break in the affected loop is permitted to occur but must not exceed 20 percent of the area of the line that initially ruptured

(3) Branch lines connected to the RCS are defined as "small" if they have an inside diameter equal to or less than 4 inches.

Small lines are restrained or arranged to meet the following requirements in addition to (1) and (2) above:

(a) Break propagation must be limited to the affected leg.

(b) Propagation of the break in the affected leg is permitted, but must be limited to a total break area of 12.5 square inches (4-inch inside diameter).

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[a] The containment is defined here as the containment structure liner and penetrations, the steam generator shell including instrumentation connections, the feedwater, blowdown, and steam generator drain lines within the containment structure.

(c) Damage to the high-head safety injection lines connected to the other leg of the affected loop or to the other loops must be prevented.

(d) Propagation of the break to high-head safety injection lines connected to the affected leg must be prevented if the line break results in a loss of core cooling capability due to a spilling injection line.

(4) Where restraints are necessary to prevent impact causing unacceptable damage to essential systems and components, they are installed such that a plastic hinge (unrestrained rotation) on the pipe at the two support points closest to the break will not be formed (see Section 3.6.5.2). Pipes are allowed to form plastic hinges in areas or arrangements where:

(a) Whipping free sections cannot reach equipment or other pipes for which protection is required.

(b) Protective barriers prevent the whipping pipe from impacting components or pipes requiring protection.

(c) The internal energy of the pipe is insufficient to impair the function of any equipment or structure, i.e., the operating temperature is less than 200°F and the operating pressure is below 275 psig.

Pursuant to RG 1.46, lines hitting equal or larger size lines of the same schedule will not cause failure of the line being hit, e.g., failure of a 2-inch line will not cause subsequent failure of another 2-inch line or a line of larger size. The reverse, however, is assumed to be probable; i.e., a 4-inch line, should it fail and whip as a result of the fluid discharged through the break, is assumed to cause failure of smaller lines, such as neighboring 3-inch or 2-inch lines, unless additional justification is provided.

3.6.5.1.2 High-Energy Piping Breaks Outside Containment

The safety-related features requiring protection from high-energy pipe breaks outside containment are discussed in Reference 7.

3.6.5.2 Pipe Restraint Design Criteria

3.6.5.2.1 Inside Containment

Where the requirements as outlined above cannot be satisfied by judicious routing of the piping, pipe whip restraints are designed and located as outlined below. Note that with the acceptance of the DCPP main RCS loop leak-before-break analysis by the NRC (Reference 12), pipe whip restraints are not required on main reactor coolant loop piping since breaks in this piping are no longer assumed to occur for evaluation of dynamic effects. Pipe whip effects due to breaks in RCS branch lines and other high energy lines must still be evaluated.

3.6.5.2.1.1 Location of Pipe Whip Restraints

Restraints are located at each zone of the piping over 1 inch in size where formation of a plastic hinge could endanger a structure, component, or system vital to safety. Design was completed prior to issuance of RG 1.46 in May 1973. The piping design on all of these systems is based on ANSI B31.1-1967.

3.6.5.2.1.2 Design of Restraint Structures

Restraint structures consist of steel rods, U-bolts, crushable bumpers, and steel frames. In determining their design load, the pipe rupture restraints are considered independent of dead and live load supports and of seismic restraints.

In equation form,

$$Y = Y_r \tag{3.6-5}$$

where:

Y = section strength required to resist design loads

Y_r = the equivalent static load on a pipe rupture restraint generated by the reaction on the broken high-energy pipe during a postulated break. The load, Y_r, includes a dynamic impact factor (DIF). Unless otherwise justified, the DIF equals 2 for restraints where rods, U-bolts, or crushable bumpers are provided, and 3 for restraints without U-bolts, rods, or crushable bumpers and a gap larger than 1/2 inch; Y_r = (DIF) (K)(P)(A), where K is the thrust coefficient.

Maximum strain of steel rods and U-bolts is limited to 50 percent of the strain corresponding to the maximum tested ultimate strength of the material. Crushable bumper deformation is limited to two-thirds of the effective crushing length of the bumper. For nonlinear analysis of steel frames, the allowable stress is the minimum specified yield stress of the material. Due to the high rate of strain that the restraint would experience after pipe rupture, the static yield strength of the frame material is increased by 5 percent.

3.6.5.2.1.3 Summary of Pipe Whip Effects

Lines that are classified as high-energy have been evaluated against RG 1.46 as the minimum design criteria. Tables 3.6-1 and 3.6-2 are checklists of pipe whip effects from postulated pipe ruptures inside the containment.

3.6.5.2.2 High-Energy Piping Breaks Outside Containment

Pipe restraint design criteria for high-energy piping outside containment are given in Reference 7.

3.6.5.3 Protective Provisions for Vital Equipment

3.6.5.3.1 High-Energy Piping Breaks Inside Containment

The piping is routed so that whipping of two free sections cannot reach equipment or other pipes for which protection is required. Barriers are used, where available, to prevent the whipping pipe from impacting equipment or piping requiring protection. For example, the crane wall, operating floor, and refueling cavity walls serve as barriers between the RCLs and the containment liner. Note that with the acceptance of the DCPP leak-before-break analysis by the NRC (Reference 12), protection of vital equipment from whipping of the main reactor coolant loop piping is not required since breaks in this piping are no longer assumed to occur for evaluation of dynamic effects. Pipe whip effects due to breaks in RCS branch lines and other high energy lines must still be evaluated. Except for ECCS lines attached to main RCLs, the engineered safety features are located outside of the crane wall. The ECCS lines that penetrate the crane wall are routed outside of the crane wall so as to penetrate it in the vicinity of the loop to which they are attached. The results of analyses demonstrate that pipe whip resulting from a postulated break in an ECCS line inside the crane wall will not cause damage in excess of that allowed by the established criteria.

3.6.5.3.2 High-Energy Piping Breaks Outside Containment

The piping anchors, rupture restraints, and restraint attachments are analyzed as described in Reference 7. Safety-related building structures are analyzed in accordance with the design load combinations described in Section 3.8.

Affected equipment and conduits have also been investigated to ensure their operability in the post-break temperature, pressure, and humidity environment[a].

3.6.5.3.3 Moderate-Energy Piping Breaks

The effects of spray or flooding from postulated ruptures of moderate-energy piping were evaluated to indicate probable loss of function of any safety-related equipment. If the loss of function of the equipment could be tolerated due to system redundancies, equipment environmental qualification, etc., no action was required. If the loss of function could not be tolerated, a design modification was required to protect the

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[a] The pressure transmitter PT 432 is incorrectly designated as essential in Section 6.1.1.4 of Reference 7. This transmitter is not essential for the satisfactory operation of the auxiliary feedwater system.

vulnerable equipment. Following a detailed review and plant walkdown, no modifications were required (Reference 10).

3.6.5.4 Pipe Whip Restraints and Spray Barriers

3.6.5.4.1 Inside Containment

For piping inside containment, the analysis shows that the locations of restraints are adequate to prevent unacceptable damage from all whips that could result from postulated breaks (see Figure 3.6-2 and Tables 3.6-1 and 3.6-2). Restraint structures consist of ASTM A304 steel rods or U-bolts, or crushable bumpers, and steel frames of ASTM A36, ASTM A441, or ASTM A516 Grade 70 material. Bumpers are 1-1/2 inch square, thin-walled A151 Grade 1010 carbon steel tubing welded together to form a multi-cell cross-section. Typical restraint structures are shown in Figures 3.6-3A to 3.6-3C. Tested properties of the ASTM A304 material are as follows:

Elastic modulus	28,000 ksi
Yield strength	34 to 46 ksi
Tensile strength	76 to 94 ksi

3.6.5.4.2 Outside Containment

For piping outside the containment, the analysis shows that the locations of restraints are adequate to prevent unacceptable damage from all whips that could result from postulated breaks. A typical restraint is shown in Figure 3.6-3.

3.6.5.4.3 Moderate-Energy Piping Breaks

Based on a detailed review and a plant walkdown, it was determined that no impingement shields (i.e., barriers) were necessary to protect vital equipment from moderate-energy pipe breaks (Reference 10). Due to system redundancy and equipment environmental qualification, the postulated MELBs would not affect the capability to achieve a safe shutdown.

3.6.5.5 Differences Between Unit 1 and Unit 2 Pipe Break Protection Features Outside Containment

DCPP Units 1 and 2 were designed to be mirror images of each other. This similarity allowed use of the same type of analysis to determine the consequence of a pipe break outside containment. The two units were not built simultaneously, which accounts for some of the design differences between them.

The major difference in protection on Unit 2 was that, for several locations, jet impingement barriers were eliminated from the Unit 2 design. However, for these locations, safety-related equipment was separated from piping systems whose failure could produce jets. Other main differences are in the change in the Unit 2 reheater drain system and location of essential equipment, instrumentation, and electrical conduit.

The reheater drain piping systems of Units 1 and 2 in the turbine building are not mirror images of each other.

Table 3.6-6, Pipe Break Protection Features on Unit 2 Different from Unit 1, describes the design features of the units that are different, compares the protection features of the units, and justifies the protection features employed for Unit 2.

3.6.6 REFERENCES

1. NRC RG 1.46, Protection Against Pipe Whip Inside Containment, May 1973.

2. Moody, ASME Paper G9-HT-31, 1969.

3. WCAP-8082, Pipe Breaks for the LOCA Analysis of the Westinghouse Primary Coolant Loop, Westinghouse Nuclear Energy Systems, June 1973.

4. Letter dated December 18, 1972, including the attachment "General Information Required for Consideration of the Effects of a Piping System Break Outside Containment" (Docket Nos. 50-275 and 50-323) from A. Giambusso of the AEC to F. T. Searls of PG&E.

5. Letter dated January 29, 1973, including the errata sheet for "General Information Required for Consideration of the Effects of a Piping System Break Outside Containment" (Docket Nos. 50-275 and 50-323) from P. Kniel of the AEC to F. T. Searls of PG&E.

6. Enclosure 3 to letter dated August 13, 1973, "Structural Design Criteria for Evaluating the Effects of High Energy Pipe Breaks on Category I Structures Outside Containment" (Docket Nos. 50-275 and 50-323) from A. Giambusso of the AEC to F. T. Searls of PG&E.

7. Nuclear Services Corporation, Evaluation for Effects of Postulated Pipe Break Outside Containment for Diablo Canyon Unit 1, Revision 2, June 1974. (Subsequent to Revision 2, the following references were issued to complete the overall justification of the effects of postulated pipe breaks outside containment: Nuclear Services Corporation, Evaluation for the Effects of Postulated Pipe Break Outside Containment for Diablo Canyon Unit 1, Revision 3, April 13, 1977; Pacific Gas and Electric Company, Design Criteria Memoranda T-29 and T-12.)

8. Standard Review Plan 3.6.2, BTP MEB 3-1, Postulated Break and Leakage Locations in Fluid System Piping Inside and Outside Containment, July 1981.

9. Standard Review Plan 3.6.2, BTP MEB 3-1 Rev. 2, Postulated Break and Leakage Locations in Fluid System Piping Inside and Outside Containment, June 1987.

10. "Reanalysis of Moderate Energy Line Break (MELB) Requirements," PG&E Memorandum, January 18, 1991.

11. WCAP-13039, Technical Justification for Eliminating Large Primary Loop Pipe Rupture as the Structural Design Basis for DCPP Units 1 and 2, Westinghouse Electric Corporation, November 1991.

12. Letter dated March 2, 1993, "Leak-Before-Break Evaluation of Reactor Coolant System Piping for DCPP Units 1 and 2," (Docket Nos. 50-275 and 50-323), from Sheri R. Peterson of the NRC to Gregory M. Rueger of PG&E.

13. NRC Inspection Manual, Part 9900: 10 CFR Guidance, LBB Analysis, "Definition of Leak-Before-Break Analysis and Its Application to Plant Piping Systems," issued September 26, 1996.

14. Deleted in Revision 19

3.6.7 REFERENCE DRAWINGS

Figures representing controlled engineering drawings are incorporated by reference and are identified in Table 1.6-1. The contents of the drawings are controlled by DCPP procedures.

3.7 SEISMIC DESIGN

3.7.1 SEISMIC INPUT

This section describes the Design Earthquake (DE), the Double Design Earthquake (DDE), and the Hosgri Earthquake (HE).

In addition to the above three design-basis earthquakes, Pacific Gas and Electric (PG&E) conducted a program to reevaluate the seismic design for DCPP in response to License Condition 2.C(7) of the DCPP Unit 1 Operating License (DPR-80), which stated, in part: "PG&E shall develop and implement a program to reevaluate the seismic design bases used for the Diablo Canyon Power Plant."

PG&E's reevaluation effort in response to the license condition was titled the "Long Term Seismic Program" (LTSP). PG&E prepared and submitted to the NRC the "Final Report of the Diablo Canyon Long Term Seismic Program" in July 1988 (Reference 19). The NRC reviewed the Final Report between 1988 and 1991, and PG&E prepared and submitted written responses to NRC questions resulting from that review. In February 1991, PG&E issued the "Addendum to the 1988 Final Report of the Diablo Canyon Long Term Seismic Program." (Reference 20) In June 1991, the NRC issued Supplement 34 to the Diablo Canyon Safety Evaluation Report (SSER) (Reference 21), in which the NRC concluded that PG&E had satisfied License Condition 2.C(7) of DPR-80. In the SSER the NRC requested certain confirmatory analyses from PG&E, and PG&E subsequently submitted the requested analyses. The NRC's final acceptance of the LTSP is documented in a letter to PG&E dated April 17, 1992 (Reference 22).

The LTSP contains extensive databases and analyses that update the basic geologic and seismic information. However, the LTSP material does not alter the design bases for DCPP. In SSER 34 (Reference 21), the NRC states, "The Staff notes that the seismic qualification basis for Diablo Canyon will continue to be the original design basis plus the Hosgri evaluation basis, along with associated analytical methods, initial conditions, etc."

PG&E committed to the NRC in a letter dated July 16, 1991 (Reference 23), that certain future plant additions and modifications, as identified in that letter, would be checked against insights and knowledge gained from the LTSP to verify that the plant margins remain acceptable.

A completed listing of bibliographic references to the LTSP reports and other documents are provided in References 19, 20, and 21.

3.7.1.1 Design Bases

The regulatory requirements listed below constitute additional Design Bases as defined in 10 CFR 50.2 and NEI 97-04, Appendix B (2000) and include: GDCs, applicable federal regulations, and system safety functional requirements.

3.7.1.1.1 General Design Criterion 2, 1967 - Performance Standards

Those systems and components of reactor facilities that are essential to the prevention of accidents which could affect the public health and safety, or to mitigation of their consequences, shall be designed, fabricated, and erected to performance standards that will enable the facility to withstand, without loss of the capability to protect the public, the additional forces that might be imposed by natural phenomena such as earthquakes. The design bases so established shall reflect (a) appropriate consideration of the most severe of these natural phenomena that have been recorded for the site and the surrounding area, and (b) an appropriate margin for withstanding forces greater than those recorded to reflect uncertainties about the historical data and their suitability as a basis for design.

Subject matter related to meeting GDC 2, 1967 falls under the following general categories:

Seismic Input

- Design Response Spectra
- Design Response Spectra Derivation
- Critical Damping Values
- Bases for Site Dependent Analysis
- Soil-Supported Category I Structures
- Soil-Structure Interaction

Seismic System Analysis

The seismic system analyses address the following considerations:

- Seismic Analysis Methods
- Natural Frequencies and Response Loads
- Procedures Used to Lump Masses
- Rocking and Translation Response Summary
- Methods Used to Couple Soil with Seismic-System Structures
- Development of Floor Response Spectra
- Differential Seismic Movement of Interconnected Components
- Effects of Variations on Floor Response Spectra
- Use of Constant Vertical Load Factors
- Methods Used to Account for Torsional Effects

- Comparison of Responses
- Methods to Determine PG&E Design Class I Structure Overturning Moments
- Analysis Procedure for Damping
- Combination of Components of Earthquake Motion for Structures

Seismic Subsystem Analysis

The seismic subsystem analyses address the following considerations:

- Determination of number of earthquake cycles
- Basis for selection of forcing frequencies
- Procedure for combining modal responses
- Root mean square basis
- Design criteria and analytical procedures for piping
- Basis for computing combined response
- Amplified seismic responses
- Use of simplified dynamic analysis
- Modal period variation
- Torsional effects of eccentric masses
- Piping outside Containment Structure
- Interaction of other piping with PG&E Design Class I Piping
- Field location of supports and restraints
- Seismic analyses for fuel elements, control rod Assemblies and control rod drives

Seismic Instrumentation Program

The seismic instrumentation program, addresses the following:

- Safety Guide 12, March 10, 1971, “Instrumentation for Earthquakes”
- Location and description of instrumentation
- Control Room operator notification
- Comparison of Measured and Predicted Responses

Seismic Design Control

- Equipment Purchased Directly by PG&E
- Equipment Supplied by Westinghouse

3.7.1.1.2 Seismic Design Safety Function Requirement

(1) Design Earthquake and Double Design Earthquake Damping Values

The damping values provided in Section 3.7.1.4 shall be used for the seismic evaluation of PG&E Design Class I SSCs for the DE and DDE.

3.7.1.1.3 Safety Guide 12, March 10, 1971 – Instrumentation for Earthquakes

The seismic instrumentation is designed to fulfill the requirements of Safety Guide 12, March 10, 1971.

3.7.1.1.4 Regulatory Guide 1.61, October 1973 - Damping Values for Seismic Design of Nuclear Power Plants

As noted in subsection 3.7.1.4, it is acceptable to use the damping values that are defined in Table 3.7.1.4, consistent with the recommendations of Regulatory Guide 1.61, October 1973, "Damping Values for Seismic Design of Nuclear Power Plants," for the seismic evaluation of PG&E Design Class I SSCs for the Hosgri Earthquake, and the reactor coolant loop piping for the Design Earthquake and Double Design Earthquake.

3.7.1.1.5 Regulatory Guide 1.61, Revision 1, March 2007 - Damping Values for Seismic Design of Nuclear Power Plants

Damping values for the seismic design and analysis of the integrated head assembly (IHA) and control rod drive mechanism (CRDM) pressure housings that are consistent with the recommendations of Regulatory Guide 1.61, Revision 1, “Damping Values for Seismic Design of Nuclear Power Plants,” are used in the seismic qualification.

3.7.1.1.6 NUREG-0660 (Item II.C.3), May 1980 - NRC Action Plan Developed as a Result of the TMI-2 Accident

Item II.C.3 - Review of Safety Classifications and Qualifications - The object of the DCPP Seismically Induced Systems Interaction Program is to identify and eliminate any potential undesirable, seismically-induced interactions between PG&E defined targets and sources.

3.7.1.1.7 License Condition 2.C(7) of DCPP Facility Operating License DPR-80 Revision 44 (LTSP), Element (4)

License Condition 2.C (7), “Seismic Design Bases Reevaluation Program (SSER 27 Section IV.5)” requires that PG&E shall develop and implement a program to reevaluate the seismic design bases used for the Diablo Canyon Nuclear Power Plant.

Elements 1, 2 and 3 of this license condition apply to the geological and seismological aspects of seismic sources and motion and are discussed in Section 2.5

Element 4 of this license condition requires that PG&E shall assess the significance of conclusions drawn from the seismic reevaluation studies in Elements 1, 2 and 3, utilizing a probabilistic risk analysis and deterministic studies, as necessary, to assure adequacy of seismic margins.

3.7.1.1.8 ASME Code Case N-411

The use of damping values from ASME Code Case N-411 is acceptable for piping reanalysis associated with the snubber reduction program and for future pipe stress analysis work, with certain restrictions as set forth in the NRC letter dated April 7, 1986.

3.7.1.2 Design Response Spectra

Section 2.5.3 provides a discussion of the earthquakes postulated for the DCPP site and the effects of these earthquakes in terms of maximum free-field ground motion accelerations and corresponding response spectra at the plant site. Since the development of the seismic inputs for DCPP predates the issuance of 10 CFR Part 100, Appendix A, “Seismic and Geologic Siting Criteria for Nuclear Power Plants,” the following DCPP earthquakes are plant specific.

3.7.1.2.1 Design Earthquake

The maximum vibratory accelerations at the plant site associated with the DE would result from either Earthquake B or Earthquake D-modified, depending on the natural period of the vibrating body. Response acceleration spectra curves for horizontal free-field ground motion at the plant site from Earthquake B and Earthquake D-modified are presented in Figures 2.5-20 and 2.5-21, respectively.

For design purposes, the response spectra for each damping value from Earthquake B and Earthquake D-modified are combined to produce an envelope spectrum. The acceleration value for any period on the envelope spectrum is equal to the larger of the two values from the Earthquake B spectrum and the Earthquake D-modified spectrum. Vertical free field ground accelerations, and the vertical free-field ground motion response spectra are assumed to be two-thirds of the corresponding horizontal spectra.

The DE is the hypothetical earthquake that would produce these horizontal and vertical vibratory accelerations. The DE corresponds to the operating basis earthquake (OBE), as described in Appendix A to 10 CFR 100 (Reference 7).

3.7.1.2.2 Double Design Earthquake

To ensure adequate reserve energy capacity, PG&E Design Class I structures and equipment are reviewed for the DDE. The DDE is the hypothetical earthquake that would produce accelerations twice those of the DE. The DDE corresponds to the SSE, as described in Appendix A to 10 CFR Part 100 (Reference 7).

3.7.1.2.3 Hosgri Earthquake

PG&E was requested by the NRC to evaluate the plant's capability to withstand a Richter magnitude 7.5 earthquake centered along an offshore zone of geologic faulting, generally referred to as the Hosgri Fault. This evaluation is discussed in the various chapters when it is specifically referred to as the Hosgri evaluation or Hosgri event evaluation.

Acceleration response spectra curves for horizontal and vertical free field ground motion at the plant site from the HE are the Newmark and Blume spectra described in Section 2.5 and presented in Figures 2.5-29 through 2.5-32. The vertical free field response spectra are two-thirds of the corresponding horizontal spectra.

3.7.1.3 Design Response Spectra Derivation

3.7.1.3.1 Design Earthquake

The free-field ground motion acceleration time-histories used in the dynamic analyses of the containment structure, auxiliary building, turbine building, and intake structure, for the DE, are developed by the following procedure: The response spectra for 2 percent damping for Earthquake B and Earthquake D-modified are enveloped to produce a single response spectrum (DE intensity). A time-history is then developed that produces a spectrum with no significant deviation from the smooth DE-envelope spectrum. This procedure eliminates undesirable peaks and valleys that exist in the response spectrum calculated directly from Earthquake B and Earthquake D-modified records.

3.7.1.3.2 Double Design Earthquake

A similar procedure is used to obtain a free-field ground motion acceleration time-history for the DDE. The free-field ground motion acceleration time-histories for the DE and DDE are shown in Figures 3.7-1 and 3.7-2, respectively. Comparison of the response spectrum computed from the time-history with the smoothed envelope spectrum is shown in Figure 3.7-3 (2 percent damping) and in Figure 3.7-4 (5 percent damping). These spectra are calculated at period intervals of 0.01 seconds, which adequately define the spectra.

3.7.1.3.3 Hosgri Earthquake

For the HE evaluation of containment structure, auxiliary building, turbine building, and intake structure acceleration time-histories were derived from the smooth spectra by utilizing an iterative procedure computer program (Reference 15). The horizontal input motions are reduced from free-field motions to account for the presence of the structures that have large foundations. These reduced inputs have been derived by spatial averaging of acceleration across the foundations of each structure by the Tau filtering procedure (Reference 12). The resulting horizontal response spectra for these structures are shown in Figures 3.7-4A through 3.7-4F. The Blume and Newmark spectra were used as the basic horizontal inputs while the vertical input corresponded to the Newmark free field response spectrum (Reference 15).

For HE evaluation of outdoor water storage tanks and smaller structures, the horizontal design response spectra are the free-field horizontal response spectra. HE vertical design response spectra are the free-field vertical response spectra. For design purposes, the Newmark spectra are used, or alternately the Blume spectra are used, with adjustment in certain frequency ranges as necessary so that they do not fall below the corresponding Newmark spectra.

Acceleration time-histories used in the analysis of the containment and intake structures, auxiliary building, and turbine building are shown in Figures 3.7-4G through 3.7-4M. Comparison of the response spectrum computed from each time-history with the corresponding design response spectrum for 7 percent damping is shown in Figures 3.7-4N through 3.7-4T.

3.7.1.4 Critical Damping Values

The specific percentages of critical damping used for PG&E Design Class I SSCs, and the PG&E Design Class II turbine building and intake structure are listed in the following table:

Type of Structure	% of Critical Damping		
	DE	DDE	HE
Containment structures and all internal concrete structures	2.0	5.0	7.0
Other conventionally reinforced concrete structures above ground, such as shear walls or rigid frames	5.0	5.0	7.0
Welded structural steel assemblies	1.0	1.0	4.0
Bolted or riveted steel assemblies	2.0	2.0	7.0
Mechanical components (PG&E purchased)	2.0	2.0	4.0

Type of Structure	% of Critical Damping DE	DDE	HE
Vital piping systems (except reactor coolant loop)[a]	0.5	0.5	3.0[b]
Reactor coolant loop[a][c]	1.0	1.0	4.0
Replacement Steam Generators[f]	2.0	4.0	4.0
Integrated Head Assembly[g]	4.9	6.85	6.85
CRDMs[h]	5.0	5.0	5.0
Foundation rocking (containment structure only)[d]	5.0	5.0	NA[e]

(a) ASME Code Case N-411 damping may be used provided it is applied to all earthquake cases and used in response spectrum modal superposition analysis. When used, pipe displacements are checked for adequacy of clearances and pipe mounted equipment accelerations are verified against project qualification criteria.
For equipment and components modeled inline, damping should be consistent with Regulatory Guide 1.61, October 1973; a composite damping value may be used for the analysis of these piping systems.

A log of calculations is kept that indicates which calculations have used Code Case N-411 damping.

Request for NRC approval for the use of ASME Code Case N-411 was made in letter DCL-86-009, dated January 22, 1986. NRC approval was granted by letter on April 7, 1986

(b) Two percent of critical damping is used for piping less than or equal to 12 inches in diameter.

(c) Although a damping value of 1 percent is used for the DE and DDE analyses of the reactor coolant loop (RCL) piping, the maximum permissible damping values for DE and DDE are those specified in Regulatory Guide 1.61, October 1973. The NRC approved a higher damping value (4%) for the Hosgri Earthquake. Damping values of greater than 4 percent have been measured experimentally for the RCL in full-size power plants (Reference 8). These testing programs have been reviewed and approved by the NRC.

(d) Five percent of critical damping is used for structures founded on rock for the purpose of computing the response in the rocking mode, and 7 percent of critical damping is used for the purpose of computing the response in the translation mode.

(e) Analysis utilizes fixed base.

(f) These values are valid for replacement steam generator (RSG) internals and shell components up to the RSG nozzle to pipe/tube connections in the RCS, MS, and FW systems and the interface between the RSG shell and upper and lower lateral and lower vertical supports. The restrictions imposed by WCAP 7921-AR (Reference 8) shall be observed when applying these values. (Reference 27)

[g] Damping values for the IHA are based on Regulatory Guide 1.61, Revision 1 (Reference 31), Tables 1 and 2, using a weighted average for "Welded Steel or Bolted Steel with Friction Connections" and "Bolted Steel with Bearing Connections". See PG&E Document 6023227-19 (Reference 30) for computation of weighted average value. Computation of weighted average value was approved in Reference 32.

[h] Damping values for the CRDMs are based on Regulatory Guide 1.61, Revision 1, (Reference 31) as approved in Reference 33.

3.7.1.5 Bases for Site-Dependent Analysis

Site conditions used to develop the shape of site seismic design response spectra are described in Section 2.5.2.

3.7.1.6 Soil-Supported PG&E Design Class I Structures

All PG&E Design Class I plant structures are founded on rock or on concrete fill.

3.7.1.7 Soil-Structure Interaction

Soil-structure interaction effects are considered as described in Section 3.7.2.1.7.

3.7.1.8 Hosgri Evaluation

The criteria and methods used to review the major structures for response to the HE are discussed in this chapter. A comparison of the DE and the DDE criteria with the HE evaluation criteria is given in Table 3.7-1 for the containment and auxiliary building, Tables 3.7-1A for the turbine building, 3.7-1B for the intake structure, and 3.7-1C for the outdoor water storage tanks, respectively.

3.7.2 SEISMIC SYSTEM ANALYSIS

In accordance with Revision 1 to RG 1.70, paragraphs under the headings below Seismic Analysis Methods and Description of Seismic Analyses, apply to all seismic analysis performed, i.e., both seismic system analysis and seismic subsystem analysis. Paragraphs under subsequent headings in this section provide discussion of specific topics applicable to seismic system analysis. Discussion of specific topics applicable to seismic subsystem analysis is provided in Section 3.7.3. The seismic analyses of PG&E Design Class I SSCs are based on the input motions associated with the DE, DDE, and HE described in Section 3.7.1.

3.7.2.1 Seismic Analysis Methods

Primarily, four dynamic methods of seismic analysis are used for the evaluation of PG&E Design Class I SSCs: time-history modal superposition, response spectrum modal superposition, response spectrum single-degree-of-freedom, and the method for rigid equipment and piping. The concept of modal analysis and each of the four methods of seismic analysis listed above are discussed in subsequent sections.

In cases where a different method of analysis is used a description of the analysis method is provided in the FSAR section associated with its application. Specific exceptions include:

(1) Mechanical equipment whose seismic adequacy is verified by testing as described in Section 3.9

(2) Electrical and instrumentation equipment whose seismic adequacy is verified as described in Section 3.10

(3) Certain PG&E Design Class I piping less than 2-1/2 inches in diameter that is restrained according to criteria described in Section 3.7.2.2.4

(4) Reactor internals, fuel elements, control rod drive assemblies, and control rod drives, as described in Section 3.7.3.15.

3.7.2.1.1 Modal Analysis

The structure, system, or component is represented as a mathematical model that is in the form of lumped masses interconnected by springs or finite elements. The mathematical model typically has one, two, or three degrees of freedom for each lumped mass or node point, but could have as many as six degrees of freedom for each lumped mass or node point.

Each multiple-degree-of-freedom (multidegree) system has the same number of normal modes as it has degrees of freedom. The characteristics of a normal mode of vibration is that, under certain conditions, the multidegree system could vibrate freely in that mode alone, and during such vibration the ratio of displacements of any two masses is constant with time. These ratios define the characteristic shape of the mode. For any vibration of the multidegree system, the motion in any of the individual normal modes can be treated as an independent single-degree-of-freedom system, and the complete motion of the multidegree system can be obtained by superimposing the independent motions of the individual modes.

The natural frequencies and characteristic shapes are determined by solution of the equations of motions for free vibrations.

3.7.2.1.2 Time-History Modal Superposition

The time-history of response in each mode is determined from the acceleration time-history input by integration of the equations of motion. The modal responses are combined by algebraic sum to produce an accurate summation at each step.

3.7.2.1.3 Response Spectrum Modal Superposition

The response spectrum is a plot, for all periods of vibration, of the maximum acceleration experienced by a single-degree-of-freedom vibrating body during a particular earthquake. The response spectrum modal superposition method of analysis applies to multidegree systems and is based on the concept of modal analysis. The modal equation of motion for a multidegree system is analogous to the equation of motion for a single degree of freedom. The maximum response in each mode is calculated, and modal responses (displacements, accelerations, shears, moments, etc.) are combined by the square root of the sum of the squares (SRSS) method.

3.7.2.1.4 Response Spectrum, Single-Degree-of-Freedom

Many components can be accurately represented by a single-degree-of-freedom mathematical model. The response spectrum method of analysis is applicable and the concept of modal analysis is not required.

3.7.2.1.5 Method for Rigid Equipment and Piping

When it can be shown that a sub-system is rigid, a static analysis may be performed. The zero period acceleration obtained from the applicable response spectra curve may be used in static calculations.

3.7.2.2 Description of Seismic Analyses

3.7.2.2.1 PG&E Design Class I Structures

Dynamic analyses by the time-history modal superposition method were performed for the containment structure and the auxiliary building to generate acceleration time-histories at specific points in the structures, and response spectra were calculated from these. In order to provide for possible variations in the parameters used in the dynamic analyses, such as mass values, material properties, and material sections, the calculated spectra were modified. For DE and DDE analyses, it is estimated that the calculated period of the structure could vary by approximately 10 percent, and to account for this the peaks of the spectra were correspondingly widened. Similarly, for HE analyses, peaks of the spectra are widened 5 percent on the high frequency side, due to a lesser need for peak broadening as the structural stiffnesses based on the actual material strengths are used, and 15 percent on the low frequency side (Reference 15). The modified spectra, known as "smooth spectra," are used in the design of PG&E Design Class I equipment and piping located in the containment structure and auxiliary building.

A detailed analytical static model of the auxiliary building was used to distribute the seismic inertial forces and moments to various walls, diaphragms, and columns, as described in Section 3.8.2.4.

In addition to the containment structure and auxiliary building, the dynamic analyses of the PG&E Design Class I containment polar crane, pipeway structure, and outdoor water storage tanks are discussed in the following subsections.

Allowable stresses for PG&E Design Class I structures are presented in Section 3.8.

3.7.2.2.1.1 Containment Structure Model

(1) DE and DDE events

The containment structure calculations relative to responses to DE and DDE events are performed with a computer program for analysis of axisymmetric structures by the finite element method. The foundation rock mass and the containment structure are modeled as one structure system to consider the effect of rock-structure interaction, as shown in Figure 3.7-5. The boundary dimensions of the model are selected such that they do not have a significant effect on the response of the structure. The exterior shell and internal structure are modeled using shell elements with four degrees of freedom at each nodal point. The weight of mechanical equipment in the structure is included in the calculation of equivalent mass density for the structure elements. Values of elastic constants for the rock mass and their variation with depth are based on field measurements made at the plant site (Refer to Section 2.5).

In the seismic analysis of the finite element model, for DE and DDE, the motions at the boundary of the rock mass are required as input. These boundary motions are derived using procedures described in the following steps:

(a) The finite element model of the rock mass only (without the structure) is subjected to a unit impulse acceleration acting at the rock mass boundaries. As a result, the acceleration time-history (impulse response that reflects the rock mass properties) is obtained at the center nodal point on the surface of the rock mass.

(b) The impulse response function, together with the desired free-field ground motion, is used as input to a deconvolution program. The required boundary motion is obtained as the output. This boundary motion, when used as input to the nodes along the horizontal and vertical boundaries of the rock mass model, produces a time-history at the center nodal point on the surface of the model that is equivalent to the free-field motion. To check the accuracy of the derived boundary motion, the rock mass without the structure is analyzed using this motion as input, and the computed free-field ground motion at the center nodal point on the surface of the rock mass is obtained. The computed free-field spectrum is calculated for this surface motion and compared with the DE- or DDE-smoothed spectrum. Due to approximations involved in the analytical methods used to derive the boundary motions, the computer spectra show slight deviations from the desired smoothed spectra. To account for these deviations, the structural response results are then conservatively scaled upward by appropriate correction factors.

The boundary motions derived from the procedure described above are used to complete the analysis of the containment structure.

To substantiate that the coupling effect is small at the reactor pressure vessel (RPV) elevation, two floor response spectra were generated for a decoupled interior concrete structure model and a coupled RPV and the interior concrete structure model, respectively. The RPV model is a simplified one-degree-of-freedom system, with its natural frequency matching the fundamental mode of the DCPP vessel. The RPV model is attached to Node 2 of the interior concrete structure model at the vessel support elevation by the spring of the vessel model.

Floor response spectra for the decoupled and the coupled models were very similar, indicating that the coupling effect at this low elevation is very small. More importantly, the response spectra magnitude of the decoupled model is consistently higher than the coupled model between 0.05 to 0.40 seconds, and is equal at all other natural periods. This shows that, indeed, the decoupled model is more conservative.

(2) HE event

The dynamic analysis for HE is performed for exterior shell, interior concrete structures, and the annulus steel structure. The description of these structural components is given in Section 3.8.1.

The elements used in the analysis of exterior shell consist of annular rings of shell elements as shown in Figure 3.7-5A. A typical shell element has four degrees-of-freedom as shown in Figure 3.7-7. The axisymmetric model is used to compute the translational response of the structure due to the horizontal and vertical ground motion. The horizontal base motion inputs are applied directly at the top of the base mat. A response spectrum analysis of the axisymmetric model is used to determine the maximum shell forces and moments, as shown in Section 3.7.2.2 (Reference 15). Since the center of mass and the center of rigidity coincide, the translational analysis does not yield any torsional response. The torsional responses are obtained from separate lumped mass models as shown in Figure 3.7-5B. These lumped mass models account for actual (geometric) eccentricity as well as 5 percent and 7 percent accidental eccentricities.

The responses from axisymmetric model and the lumped mass models are combined at the mid-thickness of the exterior shell by absolute sum for 5 percent eccentricity and by SRSS for 7 percent eccentricity (Reference 15).

The dynamic analysis of containment internal structure is divided into two parts: concrete interior structure and annulus steel structure.

(a) The concrete interior structure mainly comprised of reactor cavity walls and crane wall is represented by an axisymmetric model as shown in Figure 3.7-5A. The weight of mechanical equipment in the structure is included in the calculation of equivalent mass density for the structure elements (Reference 15). Because the center of mass and the center of rigidity coincide, the analysis does not yield torsional modes. Therefore, a separate lumped mass model, as shown in Figure 3.7-5C, is used to consider torsional response.

Figure 3.7-5D is used to compute vertical responses of the concrete interior structures due to the HE. The lumped mass stick with model points 1, 7, 18, 29, and 40 represent the concrete walls. The annulus steel is modeled by five frames located along the circumference as shown. This model was developed at an early stage of the project to estimate vertical responses of both annulus steel and concrete structures from the HE. However, subsequently detailed models were developed for the annulus steel as described later and the model of Figure 3.7-5D is used for the vertical analysis of the concrete interior structures only. The vertical response is determined by a time-history, modal superposition analysis (Reference 15).

The models of Figures 3.7-5A, 3.7-5C, and 3.7-5D represent concrete interiors up to elevation 140 feet which is the operating floor of the containment. The secondary shield walls housing the steam generators do extend above elevation 140 feet; however, the mass of these walls above elevation 140 feet is small compared to the total concrete mass and, therefore, lumping the mass at elevation 140 feet of the walls that extend above elevation 140 feet has little effect on the dynamic behavior of concrete internals below elevation 140 feet.

(b) Several models are developed for the vertical dynamic analysis of the annulus steel. Each model represents a steel frame with a column at the outside perimeter, crane wall at the inside perimeter, and the radial beam. Figure 3.7-5E represents a typical model.

The horizontal responses of the annulus steel are considered to be the same as the concrete interior structures as computed from model of Figure 3.7-5A. This consideration is supported by:

- The study results showing that the amplification above 20 Hz for the annulus steel is negligible; and
- The modal analysis of steel frames shows that the first mode of vibration, which is the predominant mode, is approximately 20 Hz.

For the HE, the analytical models are considered fixed as shown in Figures 3.7-5A, 3.7-5B, 3.7-5C, and 3.7-5D. The fixed base assumption for the Containment Structure uncouples the exterior and interior structure responses and allows a separate, independent analysis of each structure (Reference 15). The analysis is performed using the input motions as specified in Section 3.7.1.2. These spectra are to be used for all equipment placed on top of the base slab at elevation 91 ft (Reference 15).

3.7.2.2.1.2 Containment Polar Crane

The polar crane as described in Sections 9.1.4 and 3.8.1 is an overhead gantry crane, supported by the crane wall inside the containment.

For the DE, DDE, and HE nonlinear time-history analyses are performed for the crane to consider the possibility of wheel uplift and/or slack in the hook cable. The crane structure model is shown in Figure 3.7-7A. Structural members are represented as beam elements; wheel assemblies as nonlinear gap elements with compression stiffness only, and a hook cable is represented as a truss element with no compression capability. A step-by-step integration procedure is employed to determine the response. The time-step for integration is 0.005 sec. Seismic input is provided by simultaneous, independent time-histories in three directions (two horizontal and one vertical). These time-histories are developed at the top of the crane wall from the dynamic analysis described in Section 3.7.2.2.1.1 above.

3.7.2.2.1.3 Pipeway Structure

To obtain seismic responses in the pipeway structure, a combined model is used consisting of containment exterior shell, pipeway-framing members, and the mainsteam and feedwater piping which are supported by the framing members. The three-dimensional pipeway structure model consists of steel platforms supported on structural steel columns, containment shell and auxiliary and turbine buildings. This structure is represented in the model by beam elements. Oversized holes are provided to support pipeway structure beams on the auxiliary and turbine buildings. Accordingly, the model is decoupled from auxiliary and turbine buildings in the horizontal direction.

The horizontal coupling between pipeway framing model and containment model is achieved by rigid links. The main steam and feedwater lines are included in the model since they represent significant masses for the pipeway structure.

The combined containment-pipeway structure model was excited by acceleration time-history at the containment base.

(1) DE and DDE events

Equivalent static analyses of the pipeway structure are performed for the DE and DDE events as described in Section 3.8.6. The adequacy of these analyses is confirmed by a time-history dynamic analysis.

(2) HE event

The response spectra are generated using the time-history dynamic analysis method. The effect of accidental torsion is included as discussed for the containment structure model in Section 3.7.2.2.1.1. These response spectra are used for qualification of equipment and components. The structural qualification is performed using the response spectrum dynamic modal superposition method for the Unit 1 pipeway structure and using the equivalent static method for the Unit 2 pipeway structure.

3.7.2.2.1.4 Auxiliary Building

For the DE, DDE, and HE dynamic time-history analyses of the auxiliary building are performed with a computer program for analysis of a spring and lumped mass model. Two horizontal models and a vertical model, shown in Figure 3.7-13, are used. Each model is fixed at the base (elevation 85 feet). The part of the structure between elevations 60 ft and 85 ft is below grade and therefore is not lumped as a separate mass but it is assumed to be part of the foundation mass (Reference 15). Each horizontal model consists of five lumped masses with two degrees of freedom at each mass point, one translational degree of freedom in the horizontal direction, and one rotational degree of freedom about the vertical axis.

For HE the Blume and Newmark acceleration time histories correspond to the smooth response spectra used to represent the east-west and north-south horizontal components of the Hosgri base-motion inputs to the Auxiliary Building (Reference 15). The vertical model for HE evaluation consists of five lumped masses with one translational degree of freedom in the vertical direction at each mass point. The vertical input is represented by the Newmark free-field acceleration time history, which corresponds to the smooth response spectra used as the vertical component of the Hosgri ground motion (Reference 15).

For DE, DDE, and HE the masses are represented as the mass of the slab plus one-half of the walls immediately above and below the slab, with an appropriate live load on each floor to account for the effect of small pieces of equipment, concrete pads for equipment, tanks, pumps, and incidental weight not otherwise considered. Weights of cranes, storage tanks, and other large pieces of equipment are included at the appropriate mass points. Location of the centers of masses and rigidities are calculated to consider torsional modes of vibration. It was found that the centers of mass and rigidity at each level were very close for the east-west and vertical directions. Hence these small eccentricities were ignored in the model. The actual eccentricities in the structure for the north-south direction were more substantial, and these were included in the model. The inclusion of both actual and accidental eccentricity into the horizontal model for HE requires that the torsional mass moment of inertia be included. The vertical model requires that the rotational mass be included (Reference 15). Mass moments of inertia and torsional rigidities are calculated by conventional structural analysis methods.

For DE, DDE, and HE the soil at elevation 100 feet is represented by soil springs as shown in Figure 3.7-13. The stiffnesses of these foundation springs are derived by considering the case of a rigid plate on a semi-infinite elastic half-space with a horizontal surface (References 2, 3, and 4). The auxiliary building is a broad-based and comparatively low-rise structure, and therefore rocking is insignificant.

For HE evaluation, dynamic time-history analysis of flexible floor slabs is performed using finite element models composed of plate elements. Columns supporting the slabs are represented by springs. In each model, masses of slab, equipment, piping, and other items are concentrated at appropriate nodal points. A typical flexible slab model is shown in Figure 3.7-13A. Input excitation is the vertical acceleration time-history at the slab supports, obtained from the vertical analysis of the auxiliary building model.

Dynamic time-history analyses of the fuel handling area crane support structure for the DE, DDE, and HE are performed using one model to represent six end-bay frames and a second model to represent six middle bay frames. Each model is fixed at its base and uses beam and truss elements to represent all significant structural members. Structure masses are concentrated at appropriate nodal points. The model representing the middle bay frames is shown in Figure 3.7-13B. Input excitations are translational and rotational acceleration time-histories at elevation 140 feet obtained from analysis of the auxiliary building model.

3.7.2.2.1.5 Outdoor Water Storage Tanks

The condensate storage tanks, refueling water storage tanks, and firewater and transfer tank are PG&E Design Class I steel tanks with exterior concrete encasements (Reference 15).

(a) HE event

The axisymmetric and 3-D SAP IV mathematical models used in the HE finite element analysis are shown in Figures 3.7-14, 3.7-15, 3.7-15A, and

3.7-15B. The axisymmetric model using the AXIDYN computer program is used to analyze the effects of gravity loading, hydrostatic pressure, structure inertial forces, and hydrodynamic loads consisting of impulsive and convective pressures caused by the seismic event. The fluid impulsive effects are modeled as effective fluid inertia masses attached to appropriate concrete elements (Reference 13). The 3-D SAP IV model is used to assess the effects of the nonaxisymmetric vault opening on the stresses in and around the opening area. The loads determined from dynamic analysis using axisymmetric model are input as static loads in the 3-D SAP IV model. All tanks except the firewater and transfer tank are analyzed as fixed base models.

The exterior tank of the firewater and transfer tank is analyzed as a fixed base, whereas the inner steel tank is pinned at the base in the finite-element analyses.

For horizontal direction, a response spectrum, modal superposition analysis is performed with an axisymmetric model to determine the combined dynamic effects of structure inertial forces and impulsive pressures due to the horizontal earthquake. Gravity, hydrostatic pressure, and convective pressure loads are analyzed statically. The tanks analyzed are refueling water storage tank and firewater and transfer tank. No additional analysis is done for condensate tank since it is similar to refueling water storage tank.

For the SAP IV nonaxisymmetric model, an equivalent, static, lateral load analysis based on accelerations computed from the axisymmetric model analysis is performed for the refueling water storage tank to determine the structure response maxima. The results of this analysis are applicable to other outdoor water storage tanks because they have similar vault openings and are of comparable size. The axisymmetric analyses have shown that responses of these tanks are generally similar to refueling tank.

Since the fundamental period is approximately 0.033 sec in the vertical direction, the empty tanks are determined to be rigid in that direction. Considering the possibility that fluid may not act as a rigid mass during vertical motion, effects of the vertical earthquake are obtained by scaling the results of the analysis for gravity loading and hydrostatic pressure by a factor of 1.0 for the HE (2/3 x 0.75 x amplification factor of 2).

(b) DE and DDE events

For the DE and the DDE, HE finite-element analysis results are used as the basis for evaluation. The HE responses are adjusted by the ratio of peak spectral accelerations for the DE, or the DDE, and by appropriate damping ratios.

3.7.2.2.2 PG&E Design Class II Structures Containing PG&E Design Class I Equipment

The turbine building and the intake structure are PG&E Design Class II. However, PG&E Design Class I equipment is located inside:

(a) Turbine Building: component cooling water (CCW) heat exchangers, 4.16kV vital switchgear, emergency diesel generators, and other PG&E Design Class I systems

(b) Intake Structure:auxiliary saltwater (ASW) pumps, piping, and instrumentation

In order to provide assurance that the function of the PG&E Design Class I equipment will not be adversely affected, these structures are reviewed to ensure that they would not collapse in the unlikely event of an HE. The vulnerability of the main turbine steam valves to seismically induced falling debris is reviewed and is described in Section 3.5.

The structural evaluation of the turbine building and intake structure for the HE earthquake was performed using the response spectrum dynamic modal superposition method. In addition, a time-history dynamic analysis is performed to generate DE, DDE, and HE response spectra.

3.7.2.2.2.1 Turbine Building

Turbine building horizontal analyses use one model to represent the Unit 1 portion of the building, which extends from column line 1 to 19, and a second model to represent the Unit 2 portion of the building, which extends from column line 19 to 35. The models are fixed at the base and are composed of truss, beam, and plane stress elements. The Unit 1 horizontal model is shown in Figures 3.7-15C and 3.7-15D. The Unit 2 horizontal model is similar.

Four models representing different areas of the building are used to represent the building in the vertical direction. The models are fixed at the base and consist of plate, beam, and truss elements. Three of the models are three-dimensional extending the full building height and width, and together represent the building from column lines 1 to 17 and 19 to 35. The fourth model is two-dimensional extending to elevation 140 feet only and represents the building between column lines 17 and 19. The vertical model used to represent the building between lines 1 and 5 and between lines 31 and 35 is shown in Figures 3-7-15E and 3.7-15F. Additional models are used to represent bridge crane effects. Analyses consider that both the Unit 1 and the Unit 2 bridge cranes may be located in the Unit 1 or the Unit 2 portion of the building with one of the cranes lifting 135 tons.

In generating the floor response spectra for Class I equipment for the HE, two analyses are made, elastic and inelastic using the lower bound of material strength. The final floor response spectrum is the envelope of the two spectra.

Structural evaluation of the turbine pedestal for the HE earthquake is performed using the response spectrum dynamic modal superposition method. The evaluation has shown the available gap between the turbine pedestal and the turbine building is adequate to prevent component interaction during a seismic event Three-dimensional fixed base models are used to evaluate loading of the pedestals in the horizontal and vertical directions. The model shown in Figure 3.7-15G represents the Unit 1 turbine pedestal. The Unit 2 model is similar. Pedestal members are modeled as beam elements with rigid joints to account for the stiff zones at beam-column intersections. Pedestal and turbine-generator masses are included at appropriate nodal points.

3.7.2.2.2.2 Intake Structure

The seismic analysis of the intake structure was carried out by initially separating the structure into two basic parts: (a) the pump-deck base, consisting of the massive land-side portion of the structure, from elevation -31.5 feet to the -2.1-foot pump-deck level; and (b) the remainder of the structural system. The analysis demonstrated that the massive pump-deck base below the 2.1-foot level would not amplify the ground motion. Hence, the pump-deck base need not be considered in the analysis of the remainder of the structure.

The three-dimensional mathematical model is used for the north-south and east-west/vertical analysis. Figures 3.7-15H and 3.7-15I show a typical finite element model. The model is fixed at the base and uses typical finite-element methods of discretization suitable for the structural system. Floor slabs and walls are modeled as flat-plate elements primarily to capture in-plane behavior. The slabs are shown to be rigid in the vertical direction by a separate simplified analysis. Some thick shear walls near the symmetry plane of the structure in the east-west direction are modeled as three-dimensional solid elements. There are six degrees of freedom for each node - three translational and three rotational degrees of freedom.

For the north-south analysis, the effect of the virtual mass of contained water has been considered by including the total mass of water tributary to the transverse flow straighteners (or piers). This method is considered reasonable because the relatively short distance between piers inhibits the tendency of the water to slosh and thereby reduce its virtual mass. A high-tide condition, with sea level at elevation +3.4 feet (MSL), is assumed for the analysis.

For the east-west/vertical analysis, the effect of water due to an earthquake is considered negligible because it is assumed that the water can flow in and out of the structure and will exert relatively little force on the structure.

Static and dynamic lateral earth pressures on the east wall of the intake structure are considered in the calculation of the in-place shear stress for the east-west walls and roof slabs. The earth pressure influence is combined by SRSS method with the seismic forces.

3.7.2.2.3 PG&E Design Class I Mechanical Equipment

3.7.2.2.3.1 Reactor Coolant Loop

The RCLs and their support systems are analyzed for seismic loads (DE, DDE, and HE) based on a three-dimensional, multi-mass elastic dynamic model, as discussed in Section 5.2. Table 3.7-24 shows the fundamental mode frequency ranges for RCL primary equipment (steam generator, reactor coolant pump, and reactor pressure vessel). The stress analyses for faulted condition loadings of these components from a HE are provided in Section 5.2.1.15. The analyses of the reactor internals, fuel elements, control rod drive assemblies, and control rod drives are described in Section 3.7.3.15.

3.7.2.2.3.2 Other PG&E Design Class I Mechanical Equipment

PG&E Design Class I mechanical equipment is grouped into: (a) equipment purchased directly by PG&E, and (b) equipment supplied by Westinghouse.

(1) Equipment purchased directly by PG&E

Equipment is considered rigid if all natural periods are equal to or less than 0.05 seconds for the DE and the DDE, and 0.03 seconds for the HE. Rigid equipment is designed for the maximum acceleration of the supporting structure at the equipment location. Flexible equipment is analyzed by response spectrum methods. Hydrodynamic analysis of rigid tanks is performed using the methods described in Reference 6. Flexible tanks were analyzed by the methods described in Reference 13.

Load combinations and allowable stresses for PG&E Design Class I equipment are given in Section 3.9.

(2) Equipment supplied by Westinghouse Electric Corporation

The seismic response of PG&E Design Class I piping and components is determined by response spectrum methods. The system is evaluated for the simultaneous occurrence of one horizontal and the vertical seismic input motions. For each mode, the results for the vertical excitation are added absolutely to the separate results for the north-south or east-west directions. The larger of the two values so determined at each point in the model is considered as the earthquake response. Details of the response spectrum analyses are as follows:

(a) If a component falls within one of the many categories that has been previously analyzed using a multi-degree-of-freedom model and shown to be relatively rigid, the equipment specification for the component is checked to ensure that the equivalent static g-values specified are larger than the building floor response spectrum values and therefore are conservative. Equipment is considered to be rigid relative to the building if all natural periods are equal to or less than 0.05 seconds for the DE and the DDE, and 0.03 seconds for the HE.

(b) If the component cannot be categorized as similar to a previously analyzed component that has been shown to be relatively rigid, an analysis is performed as described below.

PG&E Design Class I mechanical equipment, including heat exchangers, pumps, tanks, and valves, are analyzed using a multi-degree-of-freedom modal analysis. Appendages, such as motors attached to motor-operated valves, are included in the models. The natural frequencies and normal modes are obtained using analytical techniques developed to solve eigenvalue-eigenvector problems. A response spectrum analysis is then performed using horizontal and vertical umbrella spectra that encompass the appropriate floor response spectra developed from the building time-history analyses.

The simultaneous occurrence of horizontal and vertical motions are included in the analyses. These response spectra are combined with the modal participation factors and the mode shapes to give the structural response for each mode from which the modal stresses are determined. The combined total seismic response is obtained by adding the individual modal responses utilizing the SRSS method.

Under certain conditions, the natural frequency of the equipment is not calculated. Under those conditions, using the appropriate damping value, the peak value of acceleration response curve is used to calculate the inertia forces. This method of calculation is termed the pseudo-dynamic method.

Components and supports of the RCS are designed for the loading combinations given in Section 5.2. Components are designed in complete accordance with the ASME Boiler and Pressure Vessel Code, Section III, Nuclear Vessels, and the USAS Code for Pressure Piping (Refer to Section 5.2 for details of code years and addenda to specific components). The allowable stress limits for these components and supports are also given in Section 5.2. The loading combinations and stress limits for other components and supports are given in Section 3.9.

The Hosgri evaluation of the RCS is discussed in Section 5.2. All loop piping, components and supports of the RCS satisfy criteria demonstrating qualification for the HE.

3.7.2.2.4 PG&E Design Class I Piping

3.7.2.2.4.1 Criteria

The following criteria determine the type of seismic analysis performed for PG&E Design Class I piping:

(1) 2-1/2 inches in diameter and larger

Seismic analysis is performed by the response spectrum, modal superposition method.

(2) Less than 2-1/2 inches in diameter

Seismic analysis is performed by the response spectrum modal superposition method for all Unit 2 piping. In Unit 1, piping less than 2-1/2 inches in diameter was analyzed by sampling criteria in which systems representing the worst case configurations or reflecting generic concerns were selected for analysis by the response spectrum modal superposition method. The remainder was qualified by criteria that limit the periods of free vibration to values that assure only moderate amplification of piping responses.

3.7.2.2.4.2 Modeling

Three dimensional mathematical models are used in the response spectrum modal superposition analyses. A typical mathematical model is shown in Figure 3.7-26.

Valves and valve operators are included where appropriate in the piping models as eccentric masses. Pipe supports, restraints and equipment having a natural frequency of 20 Hz or greater are modeled as being rigid restraints. Where PG&E Design Class II piping connects to PG&E Design Class I piping, sufficient PG&E Design Class II piping is included in the model to assure qualification of the PG&E Design Class I piping and code boundary.

3.7.2.2.4.3 Allowable Stresses

Load combinations and allowable stresses for PG&E Design Class I piping are given in Section 3.9.

3.7.2.3 Natural Frequencies and Response Loads

The natural frequencies and seismic response results summarized in the following sections for the major plant structures are representative of the seismic analyses performed for the operating license review (Reference 18), but may not reflect minor changes associated with subsequent plant modifications.

3.7.2.3.1 Containment Structure

(1) DE and DDE

The natural periods for all significant modes of the containment structure are listed in Table 3.7-2. The corresponding mode shapes are shown in Figure 3.7-6. The shell forces and moments in a typical element of the model are defined in Figure 3.7-7.

The containment structure seismic analysis provides acceleration time-histories, maximum absolute accelerations, displacements, shell forces and moments, total shears, and total overturning moments. These maximum response values are listed in Tables 3.7-3 through 3.7-8 for the nodal points indicated in Figure 3.7-5.

Acceleration response spectra for the containment are calculated from the acceleration time-histories, and corresponding smooth spectra are prepared. Typical smooth spectra are shown in Figures 3.7-8 through 3.7-12.

(2) HE

The natural periods and significant modes of vibration are listed in Table 3.7-8A. Modes having a period of vibration less than 0.03 sec (frequency greater than 33 Hz) are considered to be insignificant. The percentage of the modal participation factors is defined as (Reference 15):

$$\%PF_i = \frac{PF_i}{\sum_{i=1}^{n} PF_i} \times 100$$

where:

$\%PF_i$	=	percent participation factor in ith mode
PF_i	=	participation factor in ith mode
n	=	number of significant modes used in the analysis

As shown in Table 3.7-8A three sets of periods are given for the exterior shell:

(a) Translational mode determined from model of Figure 3.7-5A

(b) Torsional and translational mode determined from Figure 3.7-5B

(c) Vertical modes determined from Figure 3.7-5D

Table 3.7-8B gives the horizontal and vertical maximum absolute accelerations and Table 3.7-8C gives the maximum relative horizontal and vertical displacement. Table 3.7-8D gives the maximum shell forces and moments. Tables 3.7-8E and 3.7-8F give the maximum total shear forces, overturning moments, torsional moments, and axial forces for the containment shell.

The horizontal floor response spectra, including the effects of accidental torsion of the structure, at the inside face of the exterior shell are shown in Figures 3.7-12A and 3.7-12B. To develop these spectra, the translational spectra are combined with the torsional spectra from the 5 percent and 7 percent accidental eccentricities. The combined translational and torsional spectra are then combined on an SRSS basis with the horizontal component due to the vertical input to yield the spectra shown in Figures 3.7-12A and 3.7-12B. The horizontal floor response spectra are applicable in any horizontal direction due to the axisymmetry of the structure (Reference 15).

The vertical floor spectra are shown in Figures 3.7-12C and 3.7-12D. These spectra are based on a dynamic vertical analysis of the exterior structure using a vertical ground motion equal to 2/3 the Newmark 7.5M Hosgri free-field (tau = 0) ground motion combined with the vertical components resulting from each of the Blume and Newmark horizontal ground motion inputs (Reference 15). Tables 3.7-8G and 3.7-8H show the accelerations, displacements, stress, and moments for the containment interior structures as a result of the horizontal dynamic analysis.

For the interior structure, the Newmark input generally produces a higher structure response than does the Blume input. Figures 3.7-12E through 3.7-12G show the response spectra for the interior structure at elevation 140 feet, which is the operating floor for the containment. The spectra are for the horizontal, torsional, and vertical response. The horizontal and torsional response spectra at the Containment centerline are applicable to both the north-south and the east-west directions due to the assumed axisymmetry of the structure. However, for points located away from the centerline, the horizontal acceleration in the tangential direction is based on the combination of the horizontal and torsional response spectra as follows (Reference 15):

$$A_{MAX\ TANG} = |A_{HORIZONTAL}| + \left|\frac{X}{g}\ A_{TORSION}\right|$$

Where:

$A_{MAX\ TANG}$ = Maximum horizontal acceleration applied tangentially at the location under consideration in g's.

$A_{HORIZONTAL}$ = Maximum horizontal acceleration of the center of mass in g's.

$A_{TORSION}$ = Torsional acceleration at the center of mass in radians/sec^2.

X = Distance in feet from the center of mass of the Containment Interior Structure to the location under consideration perpendicular to the direction of $A_{MAX\ TANG}$.

g = Gravitational constant of 32.2 ft./sec^2.

For $A_{MAX\ RADIAL}$ (Maximum horizontal acceleration applied radially at the location under consideration) there is no torsional component and $A_{MAX\ RADIAL} = A_{HORIZONTAL}$.

The same procedure is used to calculate maximum relative horizontal displacements, as well the horizontal response for the seismic analysis of equipment and piping supported on the interior structure.

The vertical floor response spectra are based on a dynamic vertical analysis of the interior structure using a vertical input equal to 2/3 the Newmark 7.5M Hosgri free-field ground motion.

For the annulus structural steel frames, a separate vertical dynamic analysis is carried out for each frame as shown in Figures 3.7-12H and 3.7-12I for Units 1 and 2, respectively. Tables 3.7-8I and 3.7-8J list the frequencies and participation factors for frame number 6 which is a typical annulus steel radial frame. After the response spectra are generated in the vertical direction for each radial frame, they are enveloped according to their locations. As shown in Figures 3.7-12H and 3.7-12I, the annulus is divided into the five major sectors (called sector frames) and the response spectra for any sector frame at a given elevation are derived from enveloping the response spectra of radial frames located in that sector. Typical enveloped response spectra are shown in Figures 3.7-12J and 3.7-12K. As discussed earlier, the annulus structure does not amplify the horizontal motion of the interior concrete.

Therefore, the horizontal spectra for the concrete interior structures are used for the annulus steel. Table 3.7-8K lists the natural frequencies for horizontal seismic motion. As mentioned in Section 3.7.2.2, the first mode frequencies are approximately 20 Hz or higher and, therefore, for the rationale given earlier, the annulus is considered rigid in the horizontal direction.

3.7.2.3.2 Containment Polar Crane

Maximum displacements for various nodes for the polar crane are given in Table 3.7-8L. The member forces and bending moments are shown in Tables 3.7-8M and Table 3.7-8N. The typical response spectra are shown in Figures 3.7-12L and 3.7-12M.

3.7.2.3.3 Pipeway Structure

The modal analysis indicates that the minimum frequency of the model is 1.6 Hz and there are 100 modes below 33 Hz indicating many closely spaced modes. The containment structure and the piping modes are included in the results since a composite model is analyzed as discussed in Section 3.7.2.2. The mode shapes indicate there are no global structural modes of the pipeway structure itself; instead, there are many local modes.

The input horizontal acceleration time-histories are scaled up by a factor of 1.06 to approximate the accidental eccentricity of masses. Five input cases are considered for the seismic analysis: The Blume horizontal time-history in E-W and N-S direction, the Newmark horizontal time-history in E-W and N-S direction, and the Newmark time-history in the vertical direction. Typical response spectra for pipeway structure are shown in Figures 3.7-12N through 3.7-12S.

3.7.2.3.4 Auxiliary Building

The natural periods for all significant modes of the auxiliary building are listed in Tables 3.7-9 through 3.7-11. Frequencies for significant modes of the fuel handling crane support structure are listed in Tables 3.7-11A and 3.7-11B.

Acceleration response spectra for the auxiliary building are calculated from the acceleration time-histories at the mass points and corresponding smooth spectra are developed. Typical spectra are shown in Figures 3.7-16 through 3.7-25 and 3.7-21A through 3.7-21I.

Maximum absolute accelerations, relative displacements, story shears, overturning moments, and torsional moments in the auxiliary building are listed in Tables 3.7-12 through 3.7-23. Maximum absolute accelerations and relative displacements in the fuel handling crane support structure are listed in Tables 3.7-8O and 3.7-8P; the displacements are obtained from static analysis of the detailed model described in Section 3.8.2.4.

3.7.2.3.5 Turbine Building

Natural frequencies of vibration in the horizontal direction in all significant modes of the Unit 1 portion of the building, for the condition where two bridge cranes are centered near column line 10.6, are listed on Table 3.7-23A. Corresponding horizontal frequencies for the Unit 2 portion of the building are similar. Natural frequencies of vibration in the vertical direction for all significant modes of the building between column lines 1 and 5 are listed on Table 3.7-23B. Corresponding vertical frequencies for the Unit 2 portion of the building are similar.

HE floor acceleration response spectra for the turbine building are calculated from acceleration time-histories at the mass points using both the Blume and Newmark ground motion as the horizontal input, and the Newmark ground motion as the vertical input (Reference 15). Typical spectra are shown in Figures 3.7-25A through 3.7-25M.

Maximum absolute accelerations and relative displacements in the Unit 1 portion of the building are listed in Tables 3.7-23C and 3.7-23D. Corresponding accelerations and displacements in the Unit 2 end of the building are similar.

Natural periods for all significant modes of the turbine pedestal model are listed in Table 3.7-23E. Maximum relative displacements of the pedestal model are listed in Table 3.7-23F.

3.7.2.3.6 Intake Structure

The natural periods and participation factors for all significant modes of the intake structure having periods of vibration less than 0.03 sec (frequency greater than 33 Hz) are listed in Tables 3.7-23G. Acceleration response spectra for the intake structure are calculated from the acceleration time-histories at the selected mass points, and corresponding smooth spectra are developed as specified in Figure 3.7-4A. Typical spectra are shown in Figures 3.7-25N through 3.7-25T. Maximum absolute acceleration, relative maximum displacements are listed in Table 3.7-23H.

3.7.2.3.7 Outdoor Water Storage Tanks

The natural periods for significant modes of the refueling water storage tanks and fire water and transfer tank are listed in Tables 3.7-23I and 3.7-23J.

3.7.2.4 Procedures Used to Lump Masses

3.7.2.4.1 Structures

The mass of the structure is assumed to be concentrated at particular locations on the model. These locations coincide with either floor levels, significant points where dynamic response is required as input for piping and equipment, nodal points in the finite element model, or any other points required to accurately define the natural frequencies and mode shapes for the significant modes. The torsional effect for containment, auxiliary building, turbine building, and intake structure is considered as discussed in Section 3.7.2.11.

3.7.2.4.2 Equipment and Piping

The mass of the equipment and piping systems is assumed to be concentrated at particular locations on the model. These locations coincide with either actual masses such as pumps, motors, valve restraints and anchors, or any other points required to accurately define the natural frequencies and mode shapes of the significant modes.

3.7.2.5 Rocking and Translational Response Summary

Methods used to consider soil-structure interaction for PG&E Design Class I structures are described in Section 3.7.2.2.

3.7.2.6 Methods Used to Couple Soil with Seismic-System Structures

The procedures used to represent the containment structure and surrounding rock mass as a finite element model, and the procedures used to derive the stiffnesses of foundation springs for the auxiliary building are described in Sections 3.7.2.2.1.1 and 3.7.2.2.1.4, respectively.

3.7.2.7 Development of Floor Response Spectra

Floor response spectra are developed using time-history modal superposition analyses as described in Section 3.7.2.3.

3.7.2.8 Differential Seismic Movement of Interconnected Components

Components and supports of the RCS are designed for the loading combinations and stress limits given in Section 5.2. The loading combinations and stress limits for other components and supports are given in Section 3.9.

3.7.2.9 Effects of Variations on Floor Response Spectra

Consideration of the effects on floor response spectra of possible variations in the parameters used for the structural analysis is discussed in connection with the development of smooth spectra in Section 3.7.2.2.1.

3.7.2.10 Use of Constant Vertical Load Factors

The PG&E Design Class I structures are heavy, massive, reinforced concrete, rigid-type structures and are founded on competent hard rock. For such structures, insignificant amplification of vertical motions can be expected, the critical factor in design being the response of the structures to horizontal earthquake motions. The containment structure and auxiliary building including Class I systems and components are designed for DE and DDE, using a vertical static coefficient equal to two-thirds of the peak horizontal ground motion, unless otherwise noted. For the HE, a dynamic analysis in the vertical direction is carried out as discussed in Sections 3.7.2.2.1 and 3.7.2.2.4.

3.7.2.11 Method Used to Account for Torsional Effects

The containment structure is essentially axisymmetric and therefore has insignificant torsional response. The torsional response of the auxiliary building is calculated by use of a combined translational and torsional mathematical model in the seismic system time-history modal superposition analysis, as described in Section 3.7.2.2.4.

For the HE evaluation of PG&E Design Class I structures, the effect of accidental torsion is included as an additional eccentricity in the mathematical models. The greater of these two cases is considered for accidental torsion:

(a) torsional response due an eccentricity equal 5 percent of the building dimension in the direction perpendicular to the applied loads combined with the translational response by the absolute summation method

(b) torsional response due an eccentricity equal 7 percent of the building dimension in the direction perpendicular to the applied loads combined with the translational response by the SRSS method

For HE evaluation of the PG&E Design Class II turbine building, turbine pedestals and the intake structure, a torsional response is calculated by the use of finite element models which include both translation and torsion. In addition, the effect of accidental eccentricity is accounted for by a 10 percent increase in the structural responses for the turbine building and intake structure. For the turbine pedestal, a static torsional moment corresponding to a 5 percent eccentricity is added to the dynamic analysis in each horizontal direction.

3.7.2.12 Comparison of Responses

Time-history analyses only are performed for PG&E Design Class I structures. Response spectrum analyses are not performed because the time-history produces spectra that represent reasonably the criteria response spectra.

3.7.2.13 Methods for Seismic Analysis of Dams

There are no dams associated with the DCPP.

3.7.2.14 Methods to Determine PG&E Design Class I Structure Overturning Moments

The maximum overturning moments for PG&E Design Class I structures are determined as part of the time-history modal superposition analyses. Vertical earthquake is considered to act concurrently with the maximum horizontal overturning moments.

3.7.2.15 Analysis Procedure for Damping

Structures are analyzed using modal superposition techniques, and element or material-associated damping ratios are given in Section 3.7.1.4. "Composite" or modal damping ratios in structural systems comprised of different element material types are selected based on an inspection of the significant mode shapes, and on the assumption that the contribution of each material to the composite effective modal damping is proportional to the elastic energy induced in each material. The following criteria and procedures are applied on a-mode-by mode basis to evaluate and conservatively determine composite damping values:

(1) Where a particular mode primarily indicates response of a single element type, the damping ratio corresponding to that element type is assigned to that mode. Where all but a negligible amount of the elastic energy is induced in, for example, concrete or rock, the damping ratio appropriate to these materials is applied. Similarly, where a lightly damped material exhibits a major portion of the elastic energy of the mode, a conservative choice is made to use the damping ratio of that material for that mode. In most cases for this plant, the modes are well defined according to material types; composite damping values can be selected on the basis of a visual inspection of mode shapes and no additional numerical computations are required.

(2) In a few instances, the above criteria cannot be applied because a particular mode indicates response of several element types. The damping ratio for that mode is conservatively estimated based on the degree of participation of the different elements. Table 3.7-10 lists the participation factors for the auxiliary building. The elastic energy induced

in the different elements is estimated and the composite damping values assigned in proportion to the elastic energy.

(3) Mass-weighted composite modal-damping is used for the DE and DDE analysis of the turbine building.

The approach described above is consistent with currently accepted techniques, and in all cases the damping values are selected conservatively. The use of this approach results in design that can conservatively resist the seismic motions postulated for the DCPP.

3.7.2.16 Combination of Components of Earthquake Motion For Structures

For DE and DDE analysis maximum structural response due to one horizontal and the vertical component of earthquake motion are combined by the absolute sum method. For HE analysis the maximum structural responses due to each of the three components of earthquake motion are typically combined by the SRSS method. For the PG&E Design Class I Containment Structure and Auxiliary Building the two horizontal components are taken as equal, although it is generally recognized that the second component is smaller than the first. Alternately, the maximum structural response due to one horizontal component of earthquake motion is combined with the vertical component by the absolute sum method if it can be shown that the results of this combination are essentially equivalent to the results determined by the SRSS combination method (Reference 15).

3.7.3 SEISMIC SUBSYSTEM ANALYSIS

3.7.3.1 Determination of Number of Earthquake Cycles

Where fatigue is a criterion, it is assumed that there are 20 occurrences of the DE, each producing 20 cycles of maximum response.

3.7.3.2 Basis for Selection of Forcing Frequencies

PG&E Design Class I equipment and piping is analyzed by the response spectrum method or the pseudo-dynamic method, using floor response spectra, unless it can be shown to be rigid, as discussed in Section 3.7.2.1. Accordingly, a special procedure to avoid certain frequencies is not needed.

3.7.3.3 Procedure for Combining Modal Responses

The method and procedure for combining modal responses are described in Sections 3.7.2.1.3 and 3.7.3.4.

3.7.3.4 Square Root of the Sum of the Squares

When PG&E Design Class I piping systems are analyzed by the response spectrum modal superposition method, all modal responses, including closely spaced modes, are combined by the SRSS method to obtain total response.

A study was conducted to evaluate the effects of combining modes with closely spaced modal frequencies by the absolute sum method. For closely spaced modes, the combined total response was obtained by taking the absolute sum of the closely spaced modes and then taking the SRSS with all other modes. Twenty-nine piping systems were studied, representing approximately 10 percent of the total number of piping systems analyzed. Of these 29 piping systems, 8 systems had no closely spaced frequencies and 8 systems had closely spaced frequencies which were in the rigid period range and therefore required no further study.

The remaining 13 systems had some modal frequencies in the flexible range that could be termed closely spaced. Of these, 5 systems had low seismic stresses with an adequate margin of safety, so that any possible increase in seismic stresses due to a combination of closely spaced frequencies by the absolute sum method would not affect the safety of the piping systems. In addition, 6 systems had closely spaced frequencies, but study of the mode shapes revealed that the seismic stresses would not be significantly affected by the absolute sum of these modal responses.

For the 2 remaining systems, it was not possible to positively conclude that the effects of combining the modes with closely spaced frequencies by absolute sum would be minimal by inspecting the stresses or mode shapes. Therefore, these 2 systems were reanalyzed by computer, and it was found that if the seismic responses of the modes with closely spaced frequencies were combined by the absolute sum method, the increase in stress would be less than 1 percent.

It was therefore concluded that the combination of modal responses of piping systems by the SRSS method is adequate and conservative.

3.7.3.5 Design Criteria and Analytical Procedures for Piping

Stresses induced in PG&E Design Class I piping from relative movement of anchor points (points where all degrees of freedom are fixed), whether due to building or equipment movement, are considered with stresses calculated in the piping response spectrum modal superposition analyses.

PG&E has developed specific guidelines for the design of PG&E Design Class I pipe supports that account for such items as allowable deflections, forces, gaps, and moments imposed on the supports. Allowable stresses and loads are described in more detail in Section 3.9.

A study (Reference 9) has also been performed to evaluate the stresses in piping systems, assuming failure of a single hydraulic or mechanical pipe snubber during a seismic event. Results of the study indicate that the probability of a snubber failing to snub and causing a pipe failure was sufficiently low that no additional design restraints had to be imposed.

As an additional control, hydraulic snubbers are visually inspected and functionally tested. These surveillance requirements are detailed in the DCPP Equipment Control Guidelines (see Chapter 16).

At the request of the NRC in April 18, 1984, in its order to modify Facility Operating License No. DPR-76, PG&E developed a program to review the small and large bore pipe supports for the specific concerns raised by that order.

The specific items requested by the NRC were as follows:

(1) PG&E shall complete the review of all small-bore piping supports which were reanalyzed and requalified by computer analysis. The review shall include consideration of the additional technical topics, as appropriate, contained in License Condition No. 7 below.

(2) PG&E shall identify all cases in which rigid supports are placed in close proximity to other rigid supports or anchors. For these cases PG&E shall conduct a program that assures loads shared between these adjacent supports and anchors result in acceptable piping and support stresses. Upon completion of this effort, PG&E shall submit a report to the NRC Staff documenting the results of the program.

Design procedures were revised to address this issue.

(3) PG&E shall identify all cases in which snubbers are placed in close proximity to rigid supports and anchors. For these cases, utilizing snubber lock-up motion criteria acceptable to the staff, PG&E shall demonstrate that acceptable piping and piping support stresses are met. Upon completion of this effort, PG&E shall submit a report to the NRC Staff documenting the results.

Design procedures were revised to address this issue.

(4) PG&E shall identify all pipe supports for which thermal gaps have been specifically included in the piping thermal analyses. For these cases the licensee shall develop a program for periodic inservice inspection to assure that these thermal gaps are maintained throughout the operating life of the plant. PG&E shall submit to the NRC Staff a report containing the gap-monitoring program.

Rather than establishing a gap-monitoring program, the piping analysis and procedures were modified to eliminate the thermal gaps in the analyses.

(5) PG&E shall provide to the NRC the procedures and schedules for the hot walkdown of the main steam system piping. PG&E shall document the main steam hot walkdown results in a report to the NRC Staff.

(6) PG&E shall conduct a review of the "Pipe Support Design Tolerance Clarification" program (PSDTC) and "Diablo Problem" system (DP) activities. The review shall include specific identification of the following:

(a) Support changes, which deviated from the defined PSDTC program scope;

(b) Any significant deviations between as-built and design configurations stemming from the PSDTC or DP activities; and

(c) Any unresolved matters identified by the DP system.

The purpose of this review is to ensure that all design changes and modifications have been resolved and documented in an appropriate manner. Upon completion PG&E shall submit a report to the NRC Staff documenting the results of this review.

(7) PG&E shall conduct a program to demonstrate that the following technical topics have been adequately addressed in the design of small and large-bore piping supports:

(a) Inclusion of warping normal and shear stresses due to torsion in those open sections where warping effects are significant.

(b) Resolution of differences between the AISC Code and Bechtel criteria with regard to allowable lengths of unbraced angle sections in bending.

(c) Consideration of lateral/torsional buckling under axial loading of angle members.

(d) Inclusion of axial and torsional loads due to load eccentricity where appropriate.

(e) Correct calculation of pipe support fundamental frequency by Rayleigh's method.

(f) Consideration of flare bevel weld effective throat thickness as used on structural steel tubing with an outside radius of less than 2T.

The above considerations were incorporated in the applicable design procedures.

All of the above specific concerns were addressed and resolved to the satisfaction of the NRC.

3.7.3.6 Basis for Computing Combined Response

As a minimum, mechanical equipment is designed for a vertical static coefficient equal to 2/3 of the peak horizontal ground motion for DE and DDE analysis. For HE analysis, specific vertical floor response spectra are used. Horizontal and vertical responses are combined by absolute sum. The responses of PG&E Design Class I equipment analyzed with the response spectrum method are combined by SRSS for HE (Reference 15).

Equipment is reviewed for a vertical force determined from a response spectrum, as described in Sections 3.7.2.2.3, 3.7.3.16, and 5.2.

The horizontal and vertical responses of PG&E Design Class I piping are determined from the two-dimensional response spectrum modal superposition analyses described in Section 3.7.2.2.4. Response spectra at the applicable piping support attachment elevations are enveloped to obtain the final design response spectra. The vertical and one horizontal response are combined by absolute sum on the modal level. Modal responses are combined by the SRSS method. The two two-dimensional results are then enveloped to obtain the total response. Figure 3.7-26 shows a typical piping mathematical model. Figure 3.7-29 illustrates the derivation of the design response spectra for a typical piping system.

In many cases, earthquake piping stresses due to DDE are not directly calculated. Instead, the results from the DE piping analysis are doubled to represent the DDE. This approach was chosen because review of the design spectra showed that the DDE accelerations did not exceed twice the DE accelerations. Since pipe stress is linear with accelerations, this approach is conservative.

3.7.3.7 Amplified Seismic Responses

Components that can be adequately characterized as a single-degree-of-freedom system are considered to have a modal participation of one.

3.7.3.8 Use of Simplified Dynamic Analysis

All methods of seismic analysis used for PG&E Design Class I structures, components, systems, and piping are described in Section 3.7.2.

Two methods of dynamic seismic analysis are used for PG&E Design Class I components and piping that are different than multiple-degree-of-freedom, modal analysis methods. The first of these is the response spectrum, single-degree-of-freedom method used for components whose dynamic behavior can be accurately represented by a single-degree-of-freedom mathematical model. The second of these is the method for rigid components where the component is designed for the maximum acceleration experienced by the supporting structure at the location of support, if all natural periods of the component are less than, or equal to, 0.05 seconds for DE and DDE, or 0.03 seconds for HE in piping analysis.

The pseudo-dynamic method of analysis is used for certain items of mechanical equipment as described in Section 3.7.2. The basis for this method is described in Section 3.7.2.2.3.

Certain Unit 1 PG&E Design Class I piping less than 2-1/2 inches in diameter is restrained according to criteria described in Section 3.7.2.2.4.

3.7.3.9 Modal Period Variation

Consideration of the effects on floor response spectra of possible variations in the parameters used for structural analysis is discussed in connection with the development of smooth spectra in Section 3.7.2.2.1.

3.7.3.10 Torsional Effects of Eccentric Masses

Where appropriate, valves and valve operators are included as eccentric masses in the mathematical models for piping seismic analysis, as described in Section 3.7.2.2.4.

3.7.3.11 Piping Outside Containment Structure

The procedures used to determine piping stresses resulting from relative movement between anchor points (points where all degrees-of-freedom are fixed) are discussed in Section 3.7.3.5. The forces exerted by piping on anchor points, including the containment structure penetrations, are included in the evaluation of stresses for PG&E Design Class I structures.

Buried PG&E Design Class I piping is confined by sand backfill in rock trenches. The piping material is ASTM A-53 or A-106 carbon steel.

3.7.3.12 Interaction of Other Piping With PG&E Design Class I Piping

Mathematical models for PG&E Design Class I piping seismic analyses normally originate and terminate at anchor points. Where PG&E Design Class II piping connects to PG&E Design Class I piping sufficient PG&E Design Class II piping is included in the mathematical model to assure qualification of the PG&E Design Class I piping and code boundary.

3.7.3.13 Seismically Induced Systems Interaction (SISI) Program

PG&E developed a program to consider seismically-induced physical interactions between PG&E Design Class II or III SSCs and PG&E Design Class I SSCs. The methodology and results of the original interaction study are presented in Reference 10 and are summarized as follows. The objective of the program was to establish confidence that when subjected to seismic events of severity up to and including the HE, SSCs important to safety shall not be prevented from performing their intended safety functions as a result of physical interactions caused by seismically induced failures of PG&E Design Class II or III SSCs. In addition, safety-related SSCs shall not lose the redundancy required to compensate for single failures as a result of such interactions.

To accomplish the program, PG&E defined as targets all SSCs required to safely shut down the plant and maintain it in a safe shutdown condition, and certain accident-mitigating systems. Initial plant operating modes of normal operation, shutdown, and refueling were considered in the selection of the target equipment. All PG&E Design Class II or III SSCs were defined as sources.

Interactions between source and target equipment were postulated by an interdisciplinary Interaction Team. The Interaction Team postulated interactions during walkdowns of the target equipment, using previously established guidelines and criteria. The Interaction Team also recommended resolutions to the postulated interactions. The findings of the Interaction Team were evaluated during a subsequent office-based technical evaluation. Any modifications deemed necessary were reviewed after completion by the Interaction Team to ensure that no new interactions were created by the modifications themselves.

The program was subjected to an independent audit by PG&E's Quality Assurance Department and a review by an Independent Review Board which reported its findings to a managing consultant who, in turn, reported his findings to PG&E management. The NRC documented acceptance of the SISI Program in SSER 31(Reference 41).

To ensure that the objective of the SISI Program is met on an ongoing basis, plant modifications and housekeeping and maintenance activities are reviewed for their potential to create seismically induced systems interactions. The SISI Program Manual (Reference 40) provides the technical guidance to perform SISI evaluations.

3.7.3.14 Field Location of Supports and Restraints

Seismic supports and restraining devices for PG&E Design Class I piping are located as follows:

3.7.3.14.1 Two Inches in Diameter and Less

Field-routed and vendor-furnished piping 2 inches and less in diameter is supported by the piping installation contractor's field personnel in accordance with criteria supplied by PG&E's engineering staff on Approved for Construction drawings. These criteria specify size, type, spacing, and permissible locations for seismic supports and restraining devices. Prior to initial fuel loading, the completed installation of this piping was reviewed by an experienced piping engineer from PG&E's engineering staff to ensure compliance with the criteria and the observance of good design practice.

3.7.3.14.2 Larger Than 2 Inches in Diameter

The size, type, and location of each support or restraining device on each line is shown on Approved for Construction drawings.

The procedures followed during development of the Approved for Construction drawings provide assurance that the field location and the seismic design of supports and restraining devices are consistent with the assumptions made in the seismic analysis. These procedures are:

(1) The locations of supports and restraining devices are established on preliminary drawings.

(2) The locations shown on the preliminary drawings are used to develop the mathematical model for the seismic analysis, and the seismic analysis is performed. If the results show piping stresses higher than allowable, adjustments are made in the location, and/or the type of support or restraining device, and the seismic analysis is repeated.

(3) The reactions calculated as part of the seismic analysis, combined with other loads, are used for final design of piping supports and restraining devices.

(4) When the design is complete, drawings are issued as Approved for Construction to the piping installation contractor. Installation of supports and restraining devices is in accordance with Approved for Construction drawings.

3.7.3.15 Seismic Analyses for Fuel Elements, Control Rod Assemblies, Control Rod Drives, and Integrated Head Assembly

3.7.3.15.1 Reactor Vessel Internals Evaluation - DE, DDE, and HE

Nonlinear dynamic seismic analysis of the reactor pressure vessel (RPV) system includes the development of the system finite element model and the synthesized time history accelerations. Both of these developments for the seismic time history analysis are discussed below.

The basic mathematical model for seismic analysis is essentially similar to a LOCA model in that the seismic model includes the hydrodynamic mass matrices in the vessel/barrel downcomer annulus to account for the fluid-solid interactions. On the other hand, the fluid-solid interactions in the LOCA analysis are accounted through the hydraulic forcing functions generated by Multiflex Code (Reference 3). Another difference between the LOCA and seismic models is the difference in loop stiffness matrices. The seismic model uses the unbroken loop stiffness matrix, whereas the LOCA model uses the broken loop stiffness matrix. Except for these two differences, the RPV system seismic model is identical to that of LOCA model.

The RPV system finite element model for the nonlinear time history dynamic analysis consists of three concentric structural sub-models connected by nonlinear impact elements and linear stiffness matrices. The first sub-model, shown in Figure 3.7-27A, represents the reactor vessel shell and its associated components. The reactor vessel is restrained by four reactor vessel supports (situated beneath alternate nozzles) and by the attached primary coolant piping. Also shown in Figure 3.7-27A is a typical RPV support mechanism.

The second sub-model, shown in Figure 3.7-27B, represents the reactor core barrel, thermal shield, lower support plate, tie plates, and the secondary support components for Unit 1 (PGE); whereas, for Unit 2 (PEG) the second sub-model is shown in Figure 3.7-27C (core barrel with neutron pads instead of thermal shield).

These sub-models are physically located inside the first, and are connected to them by stiffness matrices at the vessel/internals interfaces. Core barrel to reactor vessel shell impact is represented by nonlinear elements at the core barrel flange, upper support plate flange, core barrel outlet nozzles, and the lower radial restraints.

The third and innermost sub-model, shown in Figure 3.7-27D, represents the upper support plate assembly consisting of guide tubes, upper support columns, upper and lower core plates, and the fuel. The fuel assembly simplified structural model incorporated into the RPV system model preserves the dynamic characteristics of the entire core. For each type of fuel design the corresponding simplified fuel assembly model is incorporated into the system model. The third sub-model is connected to the first and second by stiffness matrices and nonlinear elements.

As mentioned earlier, fluid-structure or hydroelastic interaction is included in the reactor pressure vessel model for seismic evaluations. The horizontal hydroelastic interaction is significant in the cylindrical fluid flow region between the core barrel and the reactor vessel annulus. Mass matrices with off-diagonal terms (horizontal degrees-of-freedom only) attach between nodes on the core barrel, thermal shield and the reactor vessel. The mass matrices for the hydroelastic interactions of two concentric cylinders are developed using the work of Reference 36. The diagonal terms of the mass matrix are similar to the lumping of water mass to the vessel shell, thermal shield, and core barrel. The off-diagonal terms reflect the fact that all the water mass does not participate when there is no relative motion of the vessel and core barrel. It should be pointed out that the hydrodynamic mass matrix has no artificial virtual mass effect and is derived in a straight-forward, quantitative manner.

The matrices are a function of the properties of two cylinders with the fluid in the cylindrical annulus, specifically, inside and outside radius of the annulus, density of the fluid, and length of the cylinders. Vertical segmentation of the reactor vessel and the core barrel allows inclusion of radii variations along their heights and approximates the effects beam mode deformation. These mass matrices were inserted between the selected nodes on the core barrel, thermal shield, and the reactor vessel as shown in Figure 3.7-27E.

The seismic evaluations are performed by including the effects of simultaneous application of time history accelerations in three orthogonal directions. For the DE, DDE and HE, the Westinghouse generated synthesized time history accelerations at the reactor vessel support were used. The detailed seismic analyses results of the RPV system are documented in Reference 34.

The WECAN computer code, which is used to determine the response of the reactor vessel and its internals, is a general-purpose finite element code. In the finite element approach, the structure is divided into a finite number of discrete members or elements. The inertia and stiffness matrices, as well as the force array, are first calculated for each element in the local coordinates. Employing appropriate transformations, the element global matrices and arrays are assembled into global structural matrices and arrays, and used for dynamic solution of the system equations.

The results of the nonlinear seismic dynamic analyses include the transient displacements and impact loads for various elements of the mathematical model. These displacements, impact loads, and linear component loads (forces and moments) are then used by cognizant organizations for detailed component evaluations to assess the structure of the reactor vessel, reactor internals, and the fuel. Note that the linear component forces and moments are not the direct output from the modal superposition analysis but rather are obtained by post-processing the data saved from the nonlinear time history analysis.

From the modal analysis (free vibration analysis), the system eigenvalues and eigenvectors are saved to be used later in the modal superposition analysis. The validity of a complex system structural model is generally verified by comparing the calculated fundamental frequency of the system with the available test data frequency. The fundamental core barrel frequency of a four-loop thermal shield core barrel is known from test data to be approximately 6.6 to 7.0 Hz. The results of Diablo Canyon Unit 1 modal analysis show that the core barrel fundamental beam mode frequency is close to 7.0 Hz, thereby verifying the applicability of the system model for the desired analysis.

Note that the preceding paragraphs describe RPV and internals system dynamic analyses for which the WECAN computer code was used. Current analyses (such as the dynamic analyses performed in support of the replacement vessel head project) utilize the ANSYS computer code. The methodology used to develop the ANSYS system models is consistent with the methodology used to develop historic WECAN models. The direct time integration method is used in ANSYS to solve the dynamic equations of motion for the system; whereas the nonlinear mode superposition method is used in WECAN to solve the dynamic equations of motion for the system.

3.7.3.15.2 Fuel Assembly Evaluation

The fuel assembly design adequacy under DDE and HE conditions was assessed through a combination of mechanical tests and analyses. The information obtained from the fuel assembly and component structural tests provided the fundamental mechanical constants for the finite element model used in the fuel analysis.

The analysis of the fuel is performed in two steps. The first step involves analysis of the detailed reactor core model, which includes the reactor vessel, internals, and a simplified model of the fuel (Figures 3.7-27A thru 3.7-27E). This dynamic analysis uses seismic time history motion at the reactor vessel support elevation (Elevation 102 ft.). The second step of the fuel analysis involves running a detailed fuel assembly model using the WEGAP code. This detailed model (Figure 3.7-27F) conservatively represents an entire row of full-length fuel assemblies (15 total).

The fuel assembly model consists of a series of beam elements with torsional springs located at the various fuel assembly grid elevations to simulate the fuel assembly dynamic characteristics. The values of the mechanical constants such as the rigidity modulus and the torsional stiffness were selected to accurately represent the experimentally determined fuel assembly modal stiffness and natural frequencies.

The time history motion for the upper and lower core plates and core barrel are simultaneously applied to the simulated fuel assembly model as illustrated in Figure 3.7-27F. These input motions were obtained from the time history analysis of the reactor vessel and internals finite element model.

The maximum grid impact forces and the fuel assembly maximum deflection are determined with the reactor core model.

Because of the basic fuel assembly design configuration, the assembly impacting is restricted to the grid locations. The seismic and LOCA loads at each grid were combined using the SRSS method to obtain the design maximum loads. These loads are compared with the allowable grid load, which is determined based on the test data using 95 percent confidence level on the true mean criteria. Note that with the acceptance of the DCPP leak-before-break analysis by the NRC, dynamic LOCA loads resulting from pipe rupture events in the main reactor coolant loop piping no longer have to be considered in the design basis structural analyses and included in the loading combinations (see Section 3.6.2.1.1.1). Only the much smaller LOCA loads from RCS branch line breaks have to be considered.

3.7.3.15.3 Control Rod Drive Mechanism Evaluation

The replacement CRDMs were evaluated using a combination of linear and nonlinear finite element models which included the CRDM housings, RVCH head adapters, and the integrated head assembly (described in Section 3.7.3.15.4). The following models and analysis methods were employed for the specified earthquakes:

The DE and DDE horizontal direction seismic loads, acting on the CRDMs, were determined using finite element models and nonlinear seismic time history methods. The finite element models included nonlinear spring elements to represent the gaps and contact between the seismic plates (near the tops of the CRDM rod travel housings) and to represent the contact between the outer seismic plates and the seismic stop plates. A linear spring was also included to represent the combined tie-rod and integrated head assembly (IHA) stiffness connection to the wall. The IHA was included in the models as a vertical beam connecting the RVCH to the tie-rod spring. An IHA horizontal beam was included to connect the IHA vertical beam to the seismic stop plates.

Time history input motions for the CRDM DE and DDE horizontal earthquake analyses were developed at the RVCH elevation and at the elevation corresponding to the tie-rod/IHA connection to the wall. The time history inputs and CRDM models were used with the ANSYS structural analysis program to determine seismic forces acting on the CRDMs along their lengths due to DE and DDE horizontal direction earthquakes.

The HE horizontal direction seismic forces and the vertical direction seismic forces for DE, DDE, and HE were determined using linear elastic response spectra methods. The vertical direction seismic analyses used response spectra at the RVCH elevation, since the CRDMs are supported vertically only at the RVCH elevation. The HE horizontal direction seismic analysis used response spectra at the RVCH elevation and the elevation of the seismic plates. To account for gaps between the seismic plates, a static analysis was also performed using a forced horizontal direction displacement at the CRDM seismic plate elevation that corresponds to the maximum cumulative gap.

Analyses show the CRDM fundamental frequency range is 5 to 8 Hz and the dominant frequencies of response are below 10 Hz. Therefore, a conservative value of 10 Hz was used in the analyses.

3.7.3.15.4 Integrated Head Assembly (IHA) Evaluation

For seismic loading, a uniform linear elastic response spectrum analysis of the IHA was performed using the spectra at the 140 foot elevation of the cavity walls and the spectra at the CG of the RRVCH to obtain enveloped spectra. These enveloped spectra are applied at both the tie rod attachments to the cavity walls as well as at the IHA ring beam attachments at the RRVCH. Tension only capability of the tie rods was modeled. Modal responses were combined by the square root of the sum of the squares (SRSS) method consistent with Section 3.7.2.1.3. These analyses were supplemented with the time history modal superposition method for selected connections. The time history modal responses are combined by algebraic sum consistent with Section 3.7.2.1.2.

Critical damping values used for the IHA components are provided in 3.7.1.4, “Critical Damping Values” and were approved in Reference 32. The combinations of the spatial components of earthquake motion are in accordance with Section 3.7.2.16. The seismic loadings were used as inputs to the IHA structural analysis presented in Section 5.2.1.14.1(3).

3.7.4 SEISMIC INSTRUMENTATION PROGRAM

3.7.4.1 Safety Guide 12, March 1971, “Instrumentation for Earthquakes”

The seismic monitoring system instrumentation consists of strong motion triaxial accelerometers that sense and record ground motions, and the amplified seismic response of various locations in the plant structures. The licensing basis for the seismic monitoring system instrumentation is in accordance with Safety Guide 12, March 1971 (Reference 39). Enhancements to the seismic instrumentation have been made to improve the system effectiveness. The enhancements include additional accelerometers and rapid processing of the ground motion data. The enhancements exceed the intent of Safety Guide 12, March 1971, and are not considered part of the licensing basis. The seismic trip system is discussed in Section 7.2.

3.7.4.2 Location and Description of Instrumentation

Seismic instrumentation is provided in accordance with Safety Guide 12, March 1971. All instruments are rigidly mounted so their records can be related to movement of the structures and ground motion. All are accessible for periodic servicing and for obtaining readings.

3.7.4.2.1 Strong Motion Triaxial Accelerometers

Strong motion triaxial accelerometers provide time-histories of acceleration for each of three orthogonal directions. These histories are recorded in the accelerometer housings. The instruments start recording upon actuation of a seismic trigger which has an adjustable threshold. Seven strong motion triaxial accelerometers are provided in accordance with Safety Guide 12, March 1971. Additional accelerometers provide ground motion data beyond the regulatory guidance and are not part of the licensing commitment.

3.7.4.3 Control Room Operator Notification

Operation of the strong motion triaxial accelerometers (ESTA01 or ESTA28) will activate an annunciator in the control room and provide indications on the earthquake force monitor (EFM) in the Rack Seismic Instrument (RSI) panel. The EFM will display the acceleration levels for all areas of both the Unit 1 containment base sensor (ESTA01) and the free field sensor (ESTA28). For the Emergency Plan event classification, it also provides a status of level exceedance for any axis on both sensors within a few minutes. The setpoint thresholds are set in accordance with Emergency Plan Action Levels.

3.7.4.4 Comparison of Measured and Predicted Responses

In the event of an earthquake that produces significant ground motions, all seismic instruments are read and the readings compared to the corresponding design values. This comparison, together with information provided by other plant instrumentation and an inspection of safety-related systems, forms the basis for a judgment on severity, level, and the effects of the earthquake.

3.7.5 SEISMIC DESIGN CONTROL

3.7.5.1 Equipment Purchased Directly by PG&E

The position of PG&E's engineering staff in the corporate structure is shown in Figures 17.1-1 and 17.1-2. The procedures for specifying technical and PG&E quality assurance requirements in purchase orders and specifications are included in Sections 17.4, 17.5, and 17.8.

The seismic design requirements developed from the structure seismic system analysis are included in the purchase order or specification for PG&E Design Class I equipment. The purchase order or specification requires that the manufacturer submit seismic qualification data of the equipment to be furnished, for review by the responsible PG&E engineer. The procurement is approved only when all seismic design criteria are met.

3.7.5.2 Equipment Supplied by Westinghouse

The following procedure is implemented for PG&E Design Class I mechanical equipment that falls within one of the many categories analyzed as described in Section 3.7.2 and shown to be rigid (fundamental frequency > 20Hz for DE and DDE and > 33 Hz for HE).

(1) Equivalent static acceleration factors for the horizontal and vertical directions must be checked against those in the Design Criteria

Memoranda (DCM). Westinghouse must certify the adequacy of the equipment to meet the seismic requirements as described in Section 3.7.2 for DE, DDE, and HE.

(2) Westinghouse must check to ensure that the given equivalent static acceleration factors are less than or equivalent to those given in the equipment analysis.

(3) Westinghouse must perform the necessary reanalysis to the procedures and criteria presented herein for those cases, where required, due to revised DE, DDE, and HE seismic response spectra.

All other PG&E Design Class I equipment must be analyzed or tested as described in Sections 3.7.2 and 3.10.

Design control measures and design documentation for all PG&E Design Class I SSCs are in accordance with formalized quality assurance procedures. These procedures are presented in Chapter 17, Quality Assurance.

3.7.6 SEISMIC EVALUATION TO DEMONSTRATE COMPLIANCE WITH THE HOSGRI EARTHQUAKE REQUIREMENTS UTILIZING A DEDICATED SHUTDOWN FLOWPATH

The evaluation of DCPP for the effects of the HE, as described in the "Seismic Evaluation for Postulated 7.5 M Hosgri Earthquake, Units 1 and 2, Diablo Canyon Site" (Reference 15) was performed in the following phases:

(a) Phase I of the evaluation addressed the seismic qualification of a minimum subset of essential equipment needed to maintain the plant in hot standby, and then go to cold shutdown, assuming offsite power was not available for the loads associated with the HE. This subset of equipment, described in Section 5.1 of Reference 15, is defined as the dedicated shutdown flowpath.

(b) Phase II of the evaluation addressed the seismic qualification of all remaining PG&E Design Class I equipment, with the exception of radwaste handling systems, for the loads associated with the HE.

As a result of the completion of Phase II of the evaluation, the dedicated post-Hosgri safe shutdown flow path, previously identified in this subsection, is no longer required. Post-Hosgri safe shutdown is now accomplished per existing plant procedures using the appropriate PG&E Design Class I equipment (Reference 15).

3.7.7 Safety Evaluation

3.7.7.1 General Design Criterion 2, 1967 - Performance Standards

The DCPP seismic design ensures that PG&E Design Class I SSCs can withstand the effects of loads associated with the Design, Double Design, and Hosgri earthquakes and maintain their design basis functions.

Subject matter related to meeting GDC 2, 1967 falls under the following general categories:

Seismic Input

- Design Response Spectra (Described in Section 3.7.1.2)
- Design Response Spectra Derivation (Described in Section 3.7.1.3)
- Critical Damping Values (Described in Sections 3.7.7.3 & 3.7.7.4)
- Bases for Site Dependent Analysis (Described in Section 3.7.1.5)
- Soil-Supported Category I Structures (Described in Section 3.7.1.6)
- Soil-Structure Interaction (Described in Section 3.7.1.7)

Seismic System Analysis

The seismic system analyses address the following considerations:

- Seismic Analysis Methods (Described in Section 3.7.2.1)
- Natural Frequencies and Response Loads (Described in Section 3.7.2.3)
- Procedures Used to Lump Masses (Described in Section 3.7.2.4)
- Rocking and Translation Response Summary (Described in Section 3.7.2.5)
- Methods Used to Couple Soil with Seismic-System Structures (Described in Section 3.7.2.6)
- Development of Floor Response Spectra (Described in Section 3.7.2.7)
- Differential Seismic Movement of Interconnected Components (Described in Section 3.7.2.8)
- Effects of Variations on Floor Response Spectra (Described in Section 3.7.2.9)
- Use of Constant Vertical Load Factors (Described in Section 3.7.2.10)
- Methods Used to Account for Torsional Effects (Described in Section 3.7.2.11)
- Comparison of Responses (Described in Section 3.7.2.12)
- Methods to Determine PG&E Design Class I Structure Overturning Moments (Described in Section 3.7.2.14)
- Analysis Procedure for Damping (Described in Section 3.7.2.15)
- Combination of Components of Earthquake Motion for Structures (Described in Section 3.7.2.16)

Seismic Subsystem Analysis

The seismic subsystem analyses address the following considerations:

- Determination of number of earthquake cycles (Described in Section 3.7.3.1)
- Basis for selection of forcing frequencies (Described in Section 3.7.3.2)
- Procedure for combining modal responses (Described in Section 3.7.3.3)
- Root mean square basis (Described in Section 3.7.3.4)
- Design criteria and analytical procedures for piping (Described in Section 3.7.3.5)
- Basis for computing combined response (Described in Section 3.7.3.6)
- Amplified seismic responses (Described in Section 3.7.3.7)
- Use of simplified dynamic analysis (Described in Section 3.7.3.8)
- Modal period variation (Described in Section 3.7.3.9)
- Torsional effects of eccentric masses (Described in Section 3.7.3.10)
- Piping outside Containment Structure (Described in Section 3.7.3.11)
- Interaction of other piping with PG&E Design Class I Piping (Described in Section 3.7.3.12)
- Field location of supports and restraints (Described in Section 3.7.3.14)
- Seismic analyses for fuel elements, control rod Assemblies and control rod drives (Described in Section 3.7.3.15)

Seismic Instrumentation Program

The seismic instrumentation program, addresses the following:

- Safety Guide 12, March 10, 1971, “Instrumentation for Earthquakes” (Described in Section 3.7.4.1)
- Location and description of instrumentation (Described in Section 3.7.4.2)
- Control Room operator notification (Described in Section 3.7.4.3)
- Comparison of Measured and Predicted Responses (Described in Section 3.7.4.4)

Seismic Design Control

- Equipment Purchased Directly by PG&E (Described in Section 3.7.5.1)
- Equipment Supplied by Westinghouse (Described in Section 3.7.5.2)

3.7.7.2 Seismic Design Safety Function Requirement

(1) Design Earthquake and Double Design Earthquake Damping Values

Section 3.7.1.4 describes the damping values for the seismic evaluation of PG&E Design Class I SSCs for the DE and DDE.

3.7.7.3 Safety Guide 12, March 10, 1971 – Instrumentation for Earthquakes

Section 3.7.4 describes the use of Safety Guide 12, March 10, 1971, for the design of DCPP seismic monitoring instrumentation system.

3.7.7.4 Regulatory Guide 1.61, October 1973 - Damping Values for Seismic Design of Nuclear Power Plants

Section 3.7.1.4 describes the use of NRC Regulatory Guide 1.61, October 1973, damping values for the seismic evaluation of PG&E Design Class I SSC for the Hosgri earthquake.

3.7.7.5 Regulatory Guide 1.61, Revision 1 - Damping Values for Seismic Design of Nuclear Power Plants

Section 3.7.1.4 describes the use of NRC Regulatory Guide 1.61, revision 1, damping values for the seismic qualification of the IHA and control rod drive mechanism pressure housings for the design earthquake, the double design earthquake, and the Hosgri earthquake.

3.7.7.6 NUREG-0660 (Item II.C.3), May 1980 - NRC Action Plan Developed as a Result of the TMI-2 Accident

Section 3.7.3.13 describes the implementation of NUREG-0660, Item II.C.3 - Review of Safety Classifications and Qualifications, through the SISI Program. This program ensures that structures, systems, and components required for safe shutdown of the plant, as well as certain accident mitigating systems, will not be impaired from performing their safety function as a result of seismically induced interactions.

3.7.7.7 License Condition 2.C(7) of DCPP Facility Operating License DPR-80 Revision 44 (LTSP), Elements (4)

DCPP Unit 1 Facility Operating License No. DPR-80, License Condition Item 2.C.(7), stated, in part:

> "PG&E shall develop and implement a program to reevaluate the seismic design bases used for the Diablo Canyon Power Plant."

PG&E's reevaluation effort in response to the license condition was titled the "Long Term Seismic Program" (LTSP). PG&E prepared and submitted to the NRC the "Final Report of the Diablo Canyon Long Term Seismic Program" in July 1988. Between 1988 and 1991, the NRC performed an extensive review of the Final Report, and PG&E prepared and submitted written responses to formal NRC questions. In February 1991, PG&E issued the "Addendum to the 1988 Final Report of the Diablo Canyon Long Term Seismic Program". In June 1991, the NRC issued Supplement Number 34 to the Diablo Canyon SSER in which the NRC concluded that PG&E had satisfied License Condition 2.C.(7) of Facility Operating License DPR-80. In the SSER the NRC requested certain confirmatory analyses from PG&E, and PG&E subsequently submitted the requested analyses. The NRC's final acceptance of the LTSP is documented in a letter to PG&E dated April 17, 1992.

3.7.7.8 ASME Code Case N-411

Section 3.7.1.4 describes the use of ASME Code Case N-411 damping values for piping analyses.

3.7.8 REFERENCES

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3.8 DESIGN OF DESIGN CLASS I STRUCTURES

Figure 1.2-2 shows the location of all structures for DCPP Units 1 and 2. The design classification of plant structures is given in the DCPP Q-List (see Reference 8 of Section 3.2). In the Q-List, the following Design Class I structures are shown:

(1) Containment structure

(2) Auxiliary building

See Section 3.1 for discussion of the design of DCPP structures in conjunction with AEC General Design Criteria. The design of the containment structure is discussed in Section 3.8.1; the auxiliary building design is discussed in Section 3.8.2; and the Class I outdoor storage tanks, including the condensate storage, refueling water storage, and fire water tanks, are discussed in Section 3.8.3. The foundations and concrete supports of Class I structures are discussed in Section 3.8.4. Section 3.8.5 discusses the Design Class II turbine building and intake structure, both of which contain Design Class I equipment and components.

Note that the analytical results summarized in the following sections for the major plant structures are representative of the evaluations performed for the operating license review (Reference 31), but may not reflect minor changes associated with subsequent plant modifications.

3.8.1 CONTAINMENT STRUCTURE

3.8.1.1 Description of the Containment

The reactor containment for each unit is a steel-lined, reinforced concrete building of cylindrical shape with a dome roof that completely encloses the reactor and RCS. It ensures that essentially no leakage of radioactive materials to the environment would result even if gross failure of the RCS were to occur simultaneously with an earthquake of intensity twice the maximum postulated.

The containment structures for Units 1 and 2 are essentially identical, as mirror images. The following discussion applies to either unit:

The concrete outline and equipment locations are shown in Chapter 1. The exterior shell consists of a 142-foot-high cylinder, topped with a hemispherical dome. The minimum thickness of the concrete walls is 3.6 feet, and the minimum thickness of the concrete roof is 2.5 feet. Both have a nominal inside diameter of 140 feet and a nominal inside height of 212 feet. The concrete floor pad is 153 feet in diameter with a minimum thickness of 14.5 feet, with the reactor cavity near the center. The inside of the dome, cylinder, and base slab is lined with welded steel plate, which forms a leaktight membrane. The nominal thickness of the steel liner is 3/8-inch on the wall and dome and the nominal thickness of the steel liner on the base slab is 1/4-inch.

The containment is designed and will be maintained for a maximum internal pressure of 47 psig and a temperature of 271°F, coincident with a Double Design Earthquake.

The internal concrete structure approximates a 106-foot-diameter, 51-foot-high cylinder, with a slab on top. However, there are multiple openings and walls, such as the reactor support and the stainless steel lined refueling canal, which complicate the shape. The walls and top slab are generally 3 feet thick. This structure provides support for the reactor and components of the RCS, provides radiation shielding, and provides protection for the liner from postulated missiles originating from the RCS.

A polar crane is mounted on top of the internal concrete cylinder wall. The support of the polar crane, its connection to the concrete, and provisions to resist seismic forces are shown in Figure 3.8-23 and described in Section 9.1.4. Seismic analysis for the polar crane is discussed in Section 3.7.

The piping and electrical connections between equipment inside the containment structure and other parts of the plant are made through specially designed, leaktight penetrations. In addition to the piping and electrical penetrations, other penetrations are the 18-foot 6-inch diameter equipment hatch, the 9-foot 7-inch diameter personnel hatch, the 5-foot 6-inch diameter personnel emergency hatch, and the fuel transfer tube.

The 6-foot 7-inch by 13-foot ventilation duct is attached to the outside of the structure, extending from an elevation 25 feet above the base slab to the top of the dome. The duct is fabricated from steel plate with stiffeners.

A system of lightning rods is installed on the dome to protect against lightning damage.

The following paragraphs describe the various parts of the structure:

3.8.1.1.1 Exterior Shell

(1) *Reinforcing Steel*

The reinforcing steel arrangement is designed to provide continuous reinforcement for tensile and shear membrane forces in the cylinder and dome. The reinforcing in the cylinder wall consists of horizontal hoop bars, and inclined bars, oriented 60° from the horizontal. In Figure 3.8-1, layers (4) and (6) are the Number 18 hoop bars, spaced at 8-1/2 inches center-to-center vertically, and layers (3) and (5) are the inclined Number 18 bars spaced at 8-1/2 inches center-to-center, all spacing measured normal to the bars.

The dome reinforcing is accomplished by extending the inclined bars past the springline and over the dome. After crossing the dome, the same bar once again becomes an inclined bar in the cylinder. A layer (3) bar becomes a layer (5) bar after crossing the dome, as shown in

Figure 3.8-2. No inclined bars are terminated at the springline or in the dome.

The dome steel layout is based on the division of a sphere into 20 equilateral spherical triangles, as shown in Figure 3.8-3. At the springline, two sides of the triangles make an angle of 30° with the vertical. Thus, an inclined cylinder bar is parallel to the sides of the triangles at the springline. The inclined cylinder bars are extended into the dome so that they are always parallel to one side of a spherical triangle.

Figure 3.8-4 shows the five types of bars in the dome. When these five types are superimposed, there are three layers of reinforcing steel at every point above the pentagon ABCDE in Figure 3.8-3. Below pentagon ABCDE, the inclined bars make up two layers at every point, and bars similar to the cylinder hoop bars are used to provide reinforcing in the third direction.

Layers (1) and (2) (Figure 3.8-1) are inclined at 30° to the vertical and extend from the base slab to elevation 172 feet. These bars, spaced at 17 inches center-to-center, provide additional capacity to resist earthquake forces. Above elevation 170 feet, Number 4 bars are spaced at 12 inches center-to-center horizontally and vertically.

(2) *Splices*

All Number 18 bars are spliced by Cadwelding using "T-Series" sleeves, designed to develop the full tensile strength of the bar. As a general rule, splices are staggered a minimum of 3 feet.

For all penetrations except the equipment and personnel hatches, the Number 18 reinforcing bars are bent around openings. For the equipment and personnel hatch openings, a 2.5-inch-thick hexagonal collar, widened to 4 inches thick at the edges, is provided to transfer the reinforcing bar forces around the opening as shown in Figures 3.8-12 and 3.8-13. The reinforcing bars are Cadwelded to special studs threaded into the 4-inch edge of the hexagonal collar.

3.8.1.1.2 Liner

All liner seams are full penetration butt-welded, and are covered with steel channels welded to the inside of the structure. These "leak chase" channels provide a sensitive and accurate means of detecting leakage. They are arranged in zones so that one zone at a time may be pressurized to test the integrity of the liner plate welds.

The liner in the dome and cylinder wall is anchored by welded studs that extend into the concrete wall past the innermost layers of reinforcing steel. Three types of studs are used: a 3/8-inch diameter with an 8-1/2-inch shaft and a plain 4-inch "L" shaped arm, a

3/8-inch diameter with an 8-1/2-inch shaft and a threaded end, and a 1/2-inch diameter with an 11-inch shaft and a threaded end. All threaded studs are provided with an anchorage at the threaded end, and provide resistance to pullout that is equal to or greater than the 3/8-inch stud with a 4-inch arm. The studs are spaced a maximum of 19.6 inches on center (plus a placement tolerance of 1/2 inch) in a pattern that is compatible with the reinforcing steel, as shown in Figure 3.8-5.

For all penetrations in the exterior shell, a thickened plate is welded into the liner.

3.8.1.1.3 Penetrations

In general, a penetration consists of a sleeve embedded in the concrete wall and welded to the containment structure liner. The pipe, electrical conductor cartridge, duct, or access hatch passes through the embedded sleeve and one or both ends of the resulting annulus are closed off by welded end plates, bolted flanges, or flued heads. Typical electrical and piping penetrations are shown in Figures 3.8-6 through 3.8-10, and the fuel transfer tube penetration is shown in Figure 3.8-11. The penetrations are designed to maintain the same high degree of leaktight integrity afforded by the containment structure itself.

3.8.1.1.3.1 Electrical Penetrations

Electrical penetrations are either canister types or feed-through modules that allow electrical conductors to pass through the containment boundary. Penetrations are qualified for a single seal pressure boundary. The canister and feed-through modules are connected to the header plate, which is welded to the containment penetration sleeve. All penetrations are provided with a connection to allow periodic leak testing. The weld connecting the sleeve to the liner plate is provided with a leak chase channel for leak testing.

3.8.1.1.3.2 Piping Penetrations

Piping penetrations are provided for all piping passing through the containment boundary. Typical piping penetrations are shown in Figures 3.8-6 and 3.8-7. Several small pipes may pass through a single embedded sleeve to minimize the number of penetrations required. Welded end plates or flued heads are used to provide end closure. The welded joints are covered with a leak chase channel to allow periodic testing. The weld connecting the sleeve to the liner plate also has a leak chase channel.

Pipes carrying hot fluids through penetrations are designed to maintain the temperature of the concrete adjacent to the sleeve below 200°F under normal operating conditions.

Pipes and penetrations are anchored, as required, to resist the forces and movements incident at the penetration under normal and accident conditions, and to limit the loads imposed on the containment structure liner. Piping loads are transferred to the penetration sleeve and thence to anchors in the concrete wall rather than to the containment structure liner.

3.8.1.1.3.3 Equipment and Personnel Access Hatches

The equipment hatch is furnished with a double-gasketed flange and bolted dished door. Equipment up to a diameter of approximately 18 feet can be transferred into and out of the containment structure through this hatch. The hatch barrel is embedded in the containment structure wall and welded to the liner. Provision is made for pressurizing the space between the double gaskets of the door flanges and the weld seam leak chase channels at the sleeve-to-liner joint.

The two personnel hatches are double door, mechanically-latched, welded steel assemblies.

A quick-acting type equalizing valve connects each personnel hatch with the interior of the containment vessel for the purpose of equalizing pressure in the two systems when entering or leaving. The personnel hatch doors are interlocked to prevent simultaneous opening. Remote indicating lights and annunciators situated in the control room indicate the door operational status. Provision is made to permit bypassing the door interlocking system to allow doors to be left open during a plant cold shutdown. Each door hinge is capable of independent three-dimensional adjustment to assist proper seating. A lighting and communication system operating from an external supply is provided in the lock interior. Emergency access, to either the inner door from the containment interior, or to the outer door from outside, is possible by the use of special door unlatching tools. All doors on the personnel hatches are double gasketed and provided with fittings to allow pressurization of the space between the double gaskets.

3.8.1.1.3.4 Special Penetrations

(1) *Fuel Transfer Tube Penetration*

A fuel transfer tube penetration is provided for fuel movement between the refueling canal in the containment structure and the spent fuel pool. The penetration consists of a 20-inch-diameter stainless steel pipe installed inside a 24-inch-diameter pipe sleeve as shown in Figure 3.8-11. The inner pipe acts as the transfer tube and is fitted with a quick-opening hatch in the refueling canal and a standard gate valve in the spent fuel pool. This arrangement prevents leakage through the transfer tube in the event of an accident. The outer pipe is welded to the containment liner and provision is made, by use of a special seal ring to permit pressure testing all welds essential to the integrity of the penetration. Bellows expansion joints are provided on the pipes to compensate for any differential movement between the two pipes or other structures.

(2) *Containment Supply and Exhaust Purge Ducts*

The ventilation system purge duct is equipped with two quick-acting tight-sealing valves (one inside and one outside the containment) to be used for isolation purposes. These valves are normally closed during reactor operation. They are manually opened for containment purging but are automatically closed upon a signal of high containment pressure or high containment radiation level. The space between the valves can be pressurized to check the integrity of the penetration. In addition, the shaft seals of the purge valves are equipped with double seals with provision for testing the space between.

(3) *Spare Penetrations*

Capped spare penetrations are provided. The welds between the sleeve and the liner and between the sleeve and the cap are covered with leak chase channels.

All spaces that are equipped for pressurization of penetrations and penetration sleeves are included in the same system of pressurization zones as the liner seam leak chase channels.

Several spare penetrations are also provided with capped, blind flanged or valved and capped end connections. These are 10 CFR 50 Appendix J Type B penetrations and are leak rate tested in accordance with Appendix J, Option B, as modified by approved exemptions.

(4) *Mini-Equipment Hatches (Penetrations 58 and 60)*

The mini-equipment hatch penetrations are provided to facilitate the passage of electrical cables and compressed air/water hoses into containment during refueling outages to support maintenance activities. Each of the two penetrations are comprised of flange connections on both sides of containment. The in-containment flanges are equipped with double O-Rings, which form a double containment isolation boundary. The in-containment blind flanges are provided with pressure test connections to permit pressure testing between the O-Rings. During plant outages, a temporary configuration is used to provide a containment pressure seal while the penetration blind flange is removed from service. Both the O-Rings and temporary blind flange assemblies prevent leakage through the penetrations in the event of an accident.

3.8.1.1.4 Base Slab and Shell-Base Slab Connection

The seams on the base slab and reactor cavity liner are full penetration butt-welded and are covered with leak chase channels. The leak chase channels are arranged in zones in the same manner as those on the exterior shell liner.

There are two penetrations through the base slab for recirculation lines. These are similar to penetrations used in the exterior shell. Weld seams between the liner and the penetration sleeve and between the penetration sleeve and internal, are covered with leak chase channels. The volume in the end of the penetration internal has a fitting for pressurization. These leak chase channels and the volume in the end of the penetration internal are connected in the zones of pressurization used for liner leak chase channels.

The detail of the shell-base slab connection is shown in Figure 3.8-14. The vertical wide flange steel beams provide a gradual transition of load carrying elements between the base slab and the cylinder, and resist the radial bending moments and shears. The beams are keyed and grouted in a groove at the base slab and extend approximately 20 feet up the wall. They do not participate in resisting either uplift due to pressure, or shear and tension forces due to earthquake.

The 3-foot 8-inch thick cylinder wall is designed to offer minimum bending resistance at the junction with the base slab. To achieve this the wall is divided into three layers, with the contact surface between the layers designed as a slip surface. The 12-inch inner layer, next to the liner plate, provides stiffness to the liner plate. The L-shaped stud anchors, welded to the liner plate, and layers (1) and (2) of the wall reinforcing bars are in this layer. The middle layer is the wide flange steel beams. The voids between the beam webs are filled with concrete. The outer layer is 20 inches thick. Layers (3) through (6) of the wall reinforcing bars are in this layer. The slip surface between layers is provided by covering both flanges of the steel beams with two sheets of Johns-Manville #60 asbestos sheet packing. This packing is graphite coated on one side, and the two sheets are placed with the graphite-coated sides in contact. PG&E has successfully used this means of providing sliding supports on penstock piers for several years. The inert nature of the material, and the fact that it is completely isolated from the atmosphere by a minimum of 20 inches of concrete, combine to ensure that it will be fully effective throughout the lifetime of the plant.

The detail at the bottom of each of these three layers is shown in Figure 3.8-14. The innermost and outermost layers have a 1-inch neoprene pad to allow slight rotation without crushing of the concrete. The center layers, consisting of the beams, have a 5-inch-deep pocket in which the beams are placed and grouted.

The diagonal wall reinforcing extends to the bottom of the base slab for anchorage, as shown in Figure 3.8-15. The base slab bars are bent up at 45° and passed through the diagonal bars. The ends of the base slab bars are provided with a mechanical anchorage consisting of a Cadweld sleeve and a steel plate.

The shell liner is anchored to the base slab by Number 14 rebar welded to the bottom course liner plate, which is 3/4-inch-thick. These rebars are embedded 8-1/2 feet in the base slab concrete.

3.8.1.1.5 Internal Structure

The internal structure that is shown in Figures 3.8-16 through 3.8-22 consists of the following parts:

(1) The lower operating floor at elevation 91 feet is a 2-foot-thick concrete slab placed over the containment structure base slab liner.

(2) The circular crane wall is a 3-foot-thick, 106-foot-OD reinforced concrete wall, concentric with the exterior shell, and extending vertically from the containment structure base slab liner at elevation 89 feet to the main operating floor at elevation 140 feet. The runway for the 200-ton polar gantry crane is located on top of the circular crane wall. This wall is anchored to the containment structure base slab by Number 18 reinforcing bars. This anchorage is developed through the containment structure base slab liner by means of Cadweld sleeves welded to each side of the liner at the same locations. The polar crane is shown in Figure 3.8-23.

(3) The reactor shield wall is a 34-foot-OD, 17-foot-ID reinforced concrete wall. This wall is anchored to the containment structure base slab in the same manner as the circular crane wall.

(4) The fuel transfer canal is a stainless steel lined cavity that can be filled with water during refueling. The vertical walls of the fuel transfer canal are 4 feet thick.

(5) The main operating floor at elevation 140 feet is a 3-foot-thick concrete slab supported by the circular crane wall and the fuel transfer canal walls. This slab is thickened locally up to 7 feet near openings.

(6) Main steam line restraint towers are reinforced concrete buttresses extending from the main operating floor at elevations 140 to 184 feet.

(7) Annulus platforms are structural steel platforms at elevations 117 and 140 feet, located between the circular crane wall and the exterior shell. Steel framing is also provided at elevations 106 feet 8 inches and 101 feet 5 inches for support of piping. Figures 3.8-21 and 3.8-22 provide the structural steel framing details for these platforms.

3.8.1.1.6 Polar Crane

The polar crane structural steel frame consists of:

(1) Two main box girders 120 feet long, 10 feet deep and 4 feet wide, spaced 24 feet apart

(2) Four gantry legs, 52 feet long and made of tapered box sections supporting the girders

(3) Two sill beams, 28 feet long with box sections to connect the gantry legs

(4) Four tie beams connecting main girders and gantry legs

The arrangement as described above is shown in Figure 3.8-23.

The sill beams are supported by two wheel assemblies each and are restrained by guide struts. These struts allow the wheels to uplift during a seismic event but guide the wheels back on to the rail.

3.8.1.2 Applicable Codes, Standards, and Specifications

3.8.1.2.1 Codes and Standards

The following codes and standards were used, insofar as they are applicable, in the design and/or construction of the containment structure:

(1) ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-63)

(2) Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-65)

(3) Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-65)

(4) Inspection of the Cadweld Rebar Splice (Erico Products, Inc., RB-5M 768)

(5) Recommended Practices for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction, American Welding Society, AWS D 12.1-61

(6) AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, February 12, 1969

(7) Construction of the containment structure liner conforms to the applicable parts of Part UW, "Requirements for Unfired Pressure Vessels Fabricated by Welding," Section VIII, ASME Boiler and Pressure Vessel Code, 1968 Edition, including Addenda through Summer 1968

(8) Those parts of penetration insert plates, penetration sleeves, airlocks, and access hatches, which form part of the pressure boundary, conform to Class B requirements of Section III, ASME Boiler and Pressure Vessel Code, 1968 Edition, including Addenda through Summer 1968

(9) Code for Welding in Building Construction, AWS D 1.0-69. Work performed prior to December 12, 1969 is in accordance with the earlier edition, AWS D 1.0-66. For inspection of non-ASME structural welds or new non-ASME work performed after January 1, 1988, the guidelines of Nuclear Construction Issues Group (Visual Weld Acceptance Criteria, Vol. 1-3, EPRI Report No. NP-5380, September 1987) (Reference 27) may be used except for those cases where:

  (a) Fatigue is a governing design condition

  (b) The weld allowables are permitted to be higher than those allowed by AWS D1.1 (such as the full penetration welds evaluation for the HE)

  (c) The weld is part of work performed in the ASME Section XI Inservice Inspection Program.

(10) Stud welding is in accordance with the Supplement to American Welding Society Specifications AWS D 1.0-66 and AWS D 2.0-66 on Requirements for Stud Welding

(11) Materials, and the quality control tests for materials conform to ASTM standards

(12) Pressure tests of the containment structure, leak chase channels, double penetration volumes, volumes between double seals, and volumes between double isolation valves are in accordance with the requirements of ANSI N45.4-1972, Leakage Rate Testing of Containment Structures for Nuclear Reactors, dated March 16, 1972

(13) SG 12, Instrumentation for Earthquake, dated March 10, 1971

(14) SG 18, Structural Acceptance Test for Concrete Primary Reactor containments, dated October 27, 1971

(15) ASME Section III, Division 2, 1980

(16) ASME Section III, Division 1, Subsection NE, 1974

(17) American Petroleum Institute (API) Code 650, Welded Steel Tanks for Oil Storage

(18) United States of America Standards Institute (USASI) - N6.2 Safety Standard for the Design, Fabrication and Maintenance of Steel Containment Structures, for Stationary Nuclear Power Reactors

3.8.1.2.2 Regulatory Guides

The following guidance documents were issued after construction at the DCPP was partially completed:

(1) SG 10, Mechanical (Cadweld) Splices in Reinforcing Bars of Concrete Containments, dated March 10, 1971

(2) RG 1.15, Testing of Reinforcing Bars for Category I Concrete Structures, dated December 28, 1972

(3) SG 19, Nondestructive Examination of Primary Containment Liner Welds, dated August 11, 1972

(4) RG 1.55, Concrete Placement in Category I Structures, dated June 1973

Because the corresponding programs for the DCPP were conservatively formulated, the inspection provided essentially equals, and in many cases exceeds, that provided by the regulatory position in the guides. Detailed comparisons of the program used for the DCPP with the regulatory position of RG 1.15, SG 10, and SG 19 are presented in Tables 3.8-1 through 3.8-3, respectively. The quality assurance program for the DCPP meets the requirements of RG 1.55. In regard to RG 1.55, the references used for guidance are those listed in Appendix A of the RG, as they existed at the time of the Preliminary Safety Analysis Report (PSAR).

3.8.1.2.3 ACI-ASME for Containments

The technical requirements of the code (Reference 3) were derived from the Building Code Requirements for Reinforced Concrete (ACI 318-71), from Section III, Division 1, of the ASME Boiler and Pressure Vessel Code, and from other codes and standards commonly applied to containment structure design, fabrication, and examination. The requirements for the DCPP containment structures are based on those same codes and standards, except that in many cases an earlier edition was applied to DCPP.

Tables 3.8-1, 3.8-2, and 3.8-3 compare the DCPP programs for reinforcing steel, Cadweld splices, and nondestructive examination of the liner with the regulatory position in RG 1.15, SG 10, and SG 19, respectively.

The general requirements of the code require third party inspection for all containment structure fabrication and construction. For DCPP, third party inspection was provided for fabrication and installation of all containment structure penetrations in accordance with the Class B requirements of Section III, ASME Boiler and Pressure Vessel Code.

3.8.1.3 Loads and Loading Combinations

3.8.1.3.1 Design Loads

The following loads were considered in the design of the containment structure:

3.8.1.3.1.1 Dead Loads

Dead loads consist of the weight of concrete, reinforcing steel, steel liner, structural steel, and permanent equipment loads. Equipment loads are supplied by the manufacturers.

3.8.1.3.1.2 Live Loads

Live loads consist of temporary equipment loads and a uniform load to account for the miscellaneous temporary loadings that may be placed on the structure.

3.8.1.3.1.3 Internal Pressure Due to Loss-of-Coolant Accident

The design peak internal pressure used for design purposes is 47 psig, which is greater than any of the peak pressures calculated in the detailed analysis reported in Chapter 6.

For design purposes, a maximum pressure differential of 15 psi due to the hypothetical LOCA was assumed to exist between the volume within the circular crane wall and the surrounding containment structure volume. The pressurizer enclosure maximum pressure differential was assumed to be 4 psi due to the vent openings provided. These values are greater than the values calculated in the detailed analysis reported in Chapter 6.

3.8.1.3.1.4 Loads Due to Thermal Expansion

These are loads resulting from the internal temperatures associated with normal operation and the hypothetical LOCA. The maximum internal atmospheric temperature during normal operation is 120°F. The temperature transients associated with the LOCA pressure and temperature are shown in Appendix 6.2C of this FSAR Update. The analyses of Appendix 6.2C correspond to a design load factor of 1.0. To determine the temperature transients for load factors equal to 1.25 and 1.5, steam tables are used.

3.8.1.3.1.5 Loads Due to Postulated Pipe Ruptures and Missile Impact

Design of the internal structure includes calculation of the effects of forces from postulated pipe ruptures transmitted through pipe restraints and equipment supports, jet forces for postulated pipe ruptures, and forces resulting from postulated missile impact. The forces from postulated pipe ruptures are calculated as described in Section 3.6. The forces from postulated missile impact are calculated as described in Section 3.5.

3.8.1.3.1.6 Earthquake Loads

Earthquake loads are based on a time-history modal superposition analysis of the containment structure and surrounding rock mass, as appropriate, as described in Section 3.7.2.

3.8.1.3.1.7 Wind Loads

Wind loads are determined in accordance with the criteria presented in Section 3.3.

3.8.1.3.1.8 Test Pressure

Internal pressure is applied to test the structural integrity of the containment vessel up to 115 percent of the design pressure. For this structure, the test pressure is 54 psig.

3.8.1.3.1.9 Negative Pressure

Negative pressure consists of loading from an internal negative pressure of 3.5 psig. This negative pressure has taken into account the Technical Specification limit on lower bound containment pressure and on inadvertent containment spray actuations, which would result in a 70°F temperature decrease.

3.8.1.3.1.10 Crane Operating Loads

Crane-operating loads include:

(1) Live load impact = (LI) = 0.2L

(2) Lateral operating load = (LAT) = 0.1 (L+TD)

(3) Longitudinal operating load = (LONG) = 0.1 (L+TD)

where:

L = Crane rated live load
TD = Trolley dead weight

3.8.1.3.2 Loading Combinations

The following loading combinations are used in design of the containment structure elements.

3.8.1.3.2.1 Operating Conditions

(1) Exterior Structure and Base Slab

Dead load, thermal load, DE, and negative pressure are considered as follows:

$$C = D + T_O + DE + NP \qquad (3.8\text{-}1)$$

where:

C	=	required capacity of section based on the methods described in Section 3.8.1.5.1
D	=	dead load of structure and equipment loads
T_O	=	thermal loads during normal operating conditions
DE	=	loads resulting from the DE
NP	=	load due to negative pressure

(2) Internal Structure

For concrete structures, dead load, live load, thermal load, and DE load are considered as follows:

$$C = D + L + T_O + DE \qquad (3.8\text{-}2)$$

For annulus steel structures, the load combinations are:

$$C = D + T_O + TH + FL \qquad (3.8\text{-}3)$$

$$C = D + T_O + TH + FV + RVOT + DE \qquad (3.8\text{-}4)$$

where:

FL	=	friction loads applied in the direction of thermal movements
FV	=	fast valve closure load
L	=	live load
RVOT	=	relief valve opening thrust load
TH	=	restrained thermal loads of the supported piping

(3) Polar Crane

C = D + TD + L + LI

C = D + TD + L + LAT

C = D + TD + L + LONG

C = D + TD + DE

3.8.1.3.2.2 Accident Conditions

(1) Exterior Shell and Base Slab

$$U = 1.0D \pm 0.05D + 1.5P_A + 1.0T'' \quad (3.8\text{-}5)$$

$$U = 1.0D \pm 0.05D + 1.25P_A + 1.0T' + 1.25DE \quad (3.8\text{-}6)$$

$$U = 1.0D \pm 0.05D + 1.0P_A + 1.0T + 1.0DDE \quad (3.8\text{-}7)$$

$$U = 1.0D \pm 0.05D + 1.0P_A + 1.0T + 1.0HE \quad (3.8\text{-}8)$$

where:

U	=	required load capacity of section based on the methods described in Section 3.8.1.5.2
P_A	=	load due to accident pressure
T	=	load due to maximum temperature associated with $1.0P_A$
T'	=	load due to maximum temperature associated with $1.25P_A$
T"	=	load due to maximum temperature associated with $1.5P_A$
DDE	=	loads resulting from the DDE
HE	=	loads resulting from the HE

(2) Internal Structure

For concrete structures, dead load, live load, earthquake load, compartment pressurization, pipe reactions associated with a postulated pipe rupture, jet forces, and missile loads are considered wherever occurring as follows:

$$U = D + L + DDE + CP + R + J + M \quad (3.8\text{-}9)$$

$$U = D + L + HE + CP + R + J + M \quad (3.8\text{-}10)$$

For annulus steel structures, the load combinations are:

U = D + DDE + THA + FV + RVOT (3.8-11)

U = D + HE (3.8-12)

where:

CP	=	compartment pressurization associated with a pipe break
R	=	pipe reactions associated with a postulated pipe rupture
J	=	jet impingement load
M	=	missile impact load
THA	=	restrained thermal expansion loads of the supported piping

(3) Polar Crane

U = D + TD + DDE + T_O

U = D + TD + L + HE

where:

T_O	=	thermal load induced by the temperature differential between the crane structure and supporting concrete structure, during operating condition
U	=	capacity of the section as determined from an increase of allowable stresses by a factor of 1.7

3.8.1.4 Design and Analysis Procedures

3.8.1.4.1 Analysis of Containment Cylinder and Dome

For the loading conditions described in Section 3.8.1.3.2, the exterior wall is subjected to membrane forces and moments. These forces and moments are shown in Figures 3.8-27 through 3.8-34, and have been calculated based on the overall elastic behavior of containment exterior wall and dome in accordance with the conventional close form solution. An exception is that at the juncture of cylinder wall and base slab, the meridional moment and shear forces are computed as described in Section 3.8.1.3.1.4.

The stresses in the reinforcing steel and concrete subject to the above membrane forces and moments are computed by assuming that the concrete cracks under tension. This involved resolving compatibility and equilibrium equations by an iterative method. The stress analysis is performed with two sets of assumptions: (a) the effect of the liner plate is neglected, and (b) the liner plate is included as a stress-carrying element. Since the thicknesses of the cylinder and dome are small in comparison with the radii of curvature, they are analyzed as a thin walled shell structure.

(1) *Internal pressure and dead load*

The calculated membrane forces due to axisymmetric loads, such as internal pressure and dead load, are shown in Figure 3.8-27.

(2) *Earthquake*

Membrane forces are from the finite element, time-history modal superposition analysis described in Section 3.7.2. The plots of these membrane forces in the cylinder and dome due to the DE, DDE, and HE are shown in Figures 3.8-28 and 3.8-29, respectively. Shear forces in the dome due to the vertical input were calculated. Vertical input produces only radial shear, but no membrane shear. Radial shears were negligible when compared to membrane shears.

(3) *Wind*

Membrane forces from wind are shown in Figure 3.8-34. These are less than the membrane forces due to an earthquake.

(4) *Temperature*

Temperature loads are considered thermal gradients in the reinforced section, including the liner plate. The analysis procedure for thermal load is described in Section 3.8.1.4.4.

The combined membrane forces for the four accident loading conditions are shown in Figures 3.8-30, 3.8-31, 3.8-32, and 3.8-33.

3.8.1.4.2 Liner Anchors

The liner anchors are designed so that they have sufficient strength and flexibility to withstand any combination of liner stress and deformation that can be reasonably assumed to occur under accident loading conditions. The liner plate system is evaluated by developing allowable loads for attached threaded studs to support the mechanical or piping system. The load transfer mechanism from the external mechanical loads through the liner plate to the concrete stud system is developed to ensure the transfer of all loads into the concrete shell with all elements remaining elastic, while maintaining the leaktight boundary.

The concrete anchors are capable of accommodating the displacement of the liner plate under the operating and accident loading conditions.

3.8.1.4.3 Equipment and Personnel Hatch Openings

Membrane forces are transferred around the equipment hatch and personnel hatch openings by means of hexagonal-shaped steel collars to which the reinforcing steel is attached.

The analysis of the equipment hatch and personnel hatch openings takes into account the following:

(1) Direct membrane forces in the shell

(2) Force concentrations in the shell

(3) Bending effect in the shell

The area of the containment shell adjacent to the opening is extended beyond the opening far enough to make the effects of the opening negligible. This area is represented by a finite element mesh consisting of three parallel layers of plate elements which are interconnected by transverse beam elements representing the transverse normal and shear stiffness of concrete wall. The outer surface represents the hexagonal plate and outside reinforcement, the inner surface represents the sleeve, the liner plate and the inner reinforcement, and the intermediate layer represents the additional reinforcement around the opening. The hexagonal plate, sleeve, and liner plate are modeled by isotropic plate elements, and the reinforcement by orthotropic elements, in which the principal directions coincide with the directions of the reinforcement. The stiffness of concrete in the membrane directions is not included, because under tension the concrete is assumed to be cracked. The finite element model is shown in Figure 3.8-35. The analysis is performed by using BSAP/CE 800 computer program.

For axisymmetric loading, the vertical boundaries are restrained in the hoop direction. For the application of tangential shear force, the lower horizontal boundaries are restrained in the vertical and tangential directions. The vertical movement of the lower horizontal boundary is always restrained.

The internal pressure is applied to a large area of the shell wall adjacent to the opening and on the hatch. Since the hatch is not a part of the model, this pressure is transferred to the nodal point around the opening as nodal loads. The pressure on the containment shell and the nodal loads around the opening are applied in the radial direction. By examining the equilibrium of the sector of the wall isolated for modeling, having boundary conditions as described above, hoop membrane forces are induced to the model at the boundary conditions in which the following equation of equilibrium is met:

$$N\Theta = pR \tag{3.8-13}$$

where:

$N\Theta$ = hoop force
p = internal pressure
R = radius of cylinder

The isostress plots are shown in Figures 3.8-40 through 3.8-42. These stresses are the results of the load combination shown by Equation 3.8-5, which controls the stress evaluation.

3.8.1.4.4 Juncture of Cylinder and Base Slab

At the base of the cylinder, radial expansion from internal pressure is considered compatible with the stiffness of the base slab. In this region, the cylinder undergoes a transition from a very small radial displacement at the base slab to full membrane displacement a short distance up from the base slab. This displacement results in longitudinal curvature in the cylinder.

A system consisting of structural steel wide flange beams, embedded in the bottom 20 feet of the concrete cylinder wall and keyed into the base slab, as shown in Figure 3.8-14, provides radial shear strength. These structural steel beams are located continuously around the circumference of the cylinder and provide bending and shear strength adequate to ensure the integrity of the wall in the transition region.

3.8.1.4.5 Base Slab

The containment base slab is evaluated by performing static analysis of the model shown in Figure 3.8-43. The model is divided into discrete segments represented by beam elements in the two horizontal directions, which simulate the two way action of the base slab. The interconnecting nodal points are supported by horizontal and vertical springs which represent the properties of the underlying foundation rock. The horizontal and vertical stiffness of the containment shell, crane wall, and the reactor cavity are also represented by beams in the respective directions. The base slab model is analyzed for load combinations pertaining to the containment exterior structure as described in Sections 3.8.1.3.2.1 and 3.8.1.3.2.2.

The seismic loads resulting from the containment and the interior structures are applied at the appropriate nodal points. When these loads cause tension at the supports, the springs are released and an iterative analysis is performed until equilibrium is achieved. The results of the analyses are shown in Table 3.8-4, along with the corresponding allowable values.

3.8.1.4.6 Internal Structure

3.8.1.4.6.1 Concrete Structure

The principal structural features and design methods of the containment internal concrete structure are as follows:

The operating deck at elevation 140 feet is supported by the 3-foot-thick, 106-foot-OD circular crane wall, 4-foot-thick fuel transfer canal walls, and structural steel columns placed on the periphery next to the containment wall. The slab within the circular crane wall is, in general, 3 feet thick. Because of irregular shape, it is represented by approximate models with negative moments based on clamped edges and positive moments based on hinged edges. Because of large openings, it was necessary to thicken parts of the slab to 7 feet, and these parts are treated as beams spanning between the circular crane wall and fuel transfer canal walls. Outside the circular crane wall the operating deck consists of a 1-foot 6-inch-thick concrete slab supported on the circular crane wall and on steel beams on the periphery; steel grating is placed over steel beams. Lateral forces are transmitted to the circular crane wall through diaphragm action of concrete slabs. The circular crane wall provides support for the operating floor at elevation 140 feet and also acts as a primary system transmitting lateral loads into the base.

3.8.1.4.6.2 Annulus Structure

The principal structural features and design methods of the annulus structure are:

The structure consists of four main framing levels at elevations 140, 117, 106, and 101 feet. This framing system is located between the circular crane wall and the containment exterior shell. The operating deck at elevation 140 feet consists mainly of a 1-foot 6-inch-thick concrete slab supported on the circular crane wall and on steel beams by columns on the periphery near the exterior containment shell. Portions of the floor consist of steel grating supported on steel beams.

The framing system is anchored to the crane wall which provides all lateral support; lateral forces are transmitted to the circular crane wall through diaphragm action of the concrete slab at elevation 140 feet, and by structural framing system at other floor elevations. Vertical support is provided by the crane wall around the inner perimeter and by the concrete base slab at elevation 90.5 feet around the outer perimeter of the annulus. The annulus framing system is not attached to the exterior shell of the containment.

The annulus structure is modeled into a three-dimensional computer model and is analyzed by the BSAP computer program for Unit 1 and the GT STRUDL and SAP2000 computer programs for Unit 2, using a method of equivalent static loads to represent seismic forces. Stresses in internal structure of containment building, including selected structural steel elements in the annulus framing system, are listed in Table 3.8-5.

3.8.1.4.7 Polar Crane

The polar crane is analyzed using a conventional frame analysis technique. The seismic analysis is described under Section 3.7.

3.8.1.4.8 Computer Programs

The main computer programs used for static and dynamic analyses of containment structure are listed in Table 3.8-6. The table also describes the general function of the programs and their respective verification measures.

3.8.1.5 Structural Acceptance Criteria

The structural acceptance criteria for the containment structure exterior shell and internal structure are as follows:

3.8.1.5.1 Operating Conditions

For operating conditions, the containment structure is designed for the allowable stresses of the applicable code such as ACI 318-63, AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, and ASME Boiler and Pressure Vessel Code, except that the increase in allowable stress or decrease in load factor usually allowed for load combinations involving earthquake or wind forces is not used.

3.8.1.5.2 Accident Conditions

For accident conditions, the containment structure is designed for overall elastic behavior under all load combinations, except at the juncture of containment exterior wall and base slab where inelastic analysis was performed by taking into consideration cracking of concrete under tension.

For structural elements designed by strength method, the yield stress of the material is reduced by ϕ factors, which are determined as follows:

Exterior shell reinforcing, structural steel embedded in exterior shell, and liner plate	ϕ= 0.95
Other structural steel	ϕ= 0.90[a]
Reinforced concrete in base slab and internal structure	ϕ factor in accordance with ACI 318-63

[a] See footnote in discussion of loading combinations in Section 3.8.2.

The yield strength values of material under a non-Hosgri event are governed by the following codes:

Reinforced concrete	ACI 318-63
Structural steel	AISC
Hexagonal collars of equipment and personnel hatches	ASME Sect. III, Div. 2, and ASME Sect. III, Div. 1
Penetration sleeves	ASME Sect. III, Div. 1
Liner plate	Table CC-3720-1[b] ASME Sect. III, Div. 2

The yield strength of steel and the ultimate strength of concrete for the HE are taken as the average values of properly substantiated test results. However, in no case are the yield values used in strength computations of structural steel greater than 70 percent of the corresponding average ultimate strength values determined by the tests. The ϕ factors described above are still applicable in the HE. The average strength values for concrete are shown in Table 3.8-6A. The minimum and average yield and ultimate strength values for reinforcing and structural steel are shown in Table 3.8-6B.

For structural steel and concrete elements designed by normal working strength method, the allowable stresses determined by AISC and ACI 318-63 codes, respectively, are increased by 1.6 for the DDE and 1.7 for the HE, except for shear in structural steel which is determined by the Von Mises criterion as outlined in the commentary of AISC.

3.8.1.5.3 Factors of Safety

The factors of safety for the exterior shell and internal structure of the containment structure are at least as great as indicated by the load factors given in Section 3.8.1.3.2. The calculated stresses for the exterior shell are given in Figures 3.8-37 through 3.8-39, and the calculated stresses for the internal structure are given in Table 3.8-5.

Separations between the containment structure and the auxiliary building are adequate to ensure these structures will not impact each other when subject to design load combinations. Calculated displacements, separations, and factors of safety against impact are shown in Table 3.8-5B.

The relative seismic displacements between the containment structure exterior shell and the internal structure have been calculated as the sum of the maximum seismic displacement of each structure. Except for a few localized areas, the minimum cold gap

[b] Table CC-3720-1 is referred to establish acceptable design strain levels for the liner plate. The construction of the liner plate is pursuant to the specifications of Section 3.8.1.2.1(7).

between the internal structure and the exterior shell is 2 inches. The factors of safety against contact for all areas, including the localized areas, are greater than 2.33 for the governing seismic event, the DDE, after thermal effects on the gap are considered. The calculated factors of safety for the HE are greater than those for the DDE.

3.8.1.6 Materials, Quality Control, and Special Construction Techniques

During the first 16 months of construction, a PG&E civil engineer was assigned to the construction site on a full-time basis. This engineer was familiar with, and had participated in, the design of the containment structure. For the period he was on site, he was part of the Quality Assurance Department (described in Chapter 17) and his responsibilities included performing audits on the various construction quality assurance programs. This engineer was qualified as ANSI Level II for radiographic, magnetic particle, ultrasonic, and dye penetrant methods of nondestructive testing. In addition, other engineers from PG&E who were involved in the design of the containment structure maintained daily contact with the site by telephone calls, and made periodic visits to the site during construction.

Inspectors from PG&E's engineering staff performed regularly scheduled shop inspections on materials and components for the containment structure.

PG&E's construction staff provided a complete staff of resident engineers, field engineers, quality control engineers, and inspectors for supervision and inspection of construction operations at the site. Their responsibilities for quality control of the containment structure were as follows:

(1) To inspect materials delivered to the jobsite and examine supplier's certified test reports of physical and chemical properties

(2) To inspect handling and placing of concrete, reinforcing bars, embedded items, and forms

(3) To maintain an adequate force of qualified supervisory personnel at all times

(4) To maintain qualified personnel, as a part of its field engineering force, to perform a thorough inspection of each significant construction operation

(5) To supervise and be fully responsible for the quality of work performed by contractors

(6) To maintain records of inspections that were performed

Many of PG&E's construction personnel at the site attended a formal course of instruction in radiographic, magnetic particle, ultrasonic, and dye penetrant methods of nondestructive testing. PG&E technicians staffed the onsite materials laboratory where tests on cement, aggregate, concrete, and reinforcing steel were performed.

3.8.1.6.1 Concrete

Concrete is a dense, durable mixture of sound aggregate, cement, water, and such admixtures as may be found advantageous. The concrete design strengths used in the containment structure are:

Exterior Shell	3,000 psi
Base Slab	5,000 psi
Internal Structures	5,000 psi

The concrete compressive strength and the modulus of elasticity values used in the analysis for load combinations, including the HE, are given in Table 3.8-6A.

Concrete construction meets, as a minimum, the requirements of ACI 318-63, Building Code Requirements for Reinforced Concrete.

(1) *Cement*

Cement is clean, fresh, Type II, low alkali, moderate heat, Portland cement conforming to the specifications of ASTM C 150, except that the PG&E specification is more stringent in requiring that the compressive strengths for any mill-run or bin be not less than 1,700 psi at 3 days, 2,700 psi at 7 days, and 4,000 psi at 28 days, and that the loss on ignition be less than 2 percent. In addition, the following Optional Chemical Requirements of ASTM C 150 are required by PG&E specification:

(a) Total alkalis of the cement, calculated as the percent of Na_2O + 0.658 times the percent of K_2O, is limited to 0.60 percent.

(b) The sum of tricalcium silicate and tricalcium aluminate is limited to 58 percent. During manufacture, samples of cement were taken once each shift, or at the rate of one sample for every 2,000 barrels. After the quality history was established, in accordance with Section 5 of the Federal Test Method Standard Number 158a, testing was performed at the reduced testing rate specified in that standard. A report of the tests made on each sample was sent to PG&E engineering research staff. In addition, each shipment of cement was accompanied by a mill certificate, and a report of the average of all the individual tests was sent with the initial delivery from each new lot or grind.

Cement shipped to the batch plant was not placed in a plant bin unless it had been accepted by PG&E. In addition to the tests the cement manufacturer performed, PG&E made the following tests on each new lot to ensure conformance with ASTM C 150:

- ASTM C 109: Compressive Strength of Hydraulic Cement Mortars (using 2-inch cube specimens)
- ASTM C 114: Chemical Analysis of Hydraulic Cement
- ASTM C 151: Autoclave Expansion of Portland Cement
- ASTM C 191: Time of Setting of Hydraulic Cement by Vicat Needle
- ASTM C 204: Fineness of Portland Cement by Air Permeability Apparatus

The tests prescribed in ASTM C 114 were also performed periodically during storage to check for any effect on cement characteristics. These tests supplemented visual inspection during storage.

(2) *Aggregates*

Aggregates consist of inert materials that are clean, hard, durable, free from organic matter, not coated with clay or dirt, and conforming to ASTM Designation C 33, Standard Specification for Concrete Aggregates. In addition to the requirements of ASTM C 33, the PG&E specification requires that:

(a) Sodium Sulfate Test for Soundness (ASTM C 88). For fine aggregate, the portion retained on a Number 50 screen, be limited to a weighted average loss of no more than 8 percent after 5 cycles. For coarse aggregate, the weighted average loss after 5 cycles be no more than 10 percent

(b) Sand Equivalent Test (California Division of Highways Test Method Number California 217). Sand equivalent value be at least 75

(c) The fineness modulus be within the limits of 2.6 to 2.9

(d) Los Angeles Rattler Test (ASTM C 131) for coarse aggregate. Loss by weight using Grading A, be a maximum of 10 percent by weight at 100 revolutions and 40 percent by weight at 500 revolutions

(e) Cleanness Value (California Division of Highways Test Method Number California 227-B) for coarse aggregate. Cleanness value be at least 75

(f) Specific Gravity (ASTM C 127) for coarse aggregate. Specific gravity on a saturated surface dry basis be at least 2.60

(g) The chloride content of aggregate be no more than 440 ppm

The following tests were performed by the aggregate supplier at the frequency indicated:

Test	ASTM Destination	Frequency
Screen Analysis and Fineness Modulus	C 136	B
Clay Lumps and Friable Particles	C 142	D
Minus 200 Mesh	C 117	D
Organic Impurities	C 40	D
Soft Particles	C 235	D
Lightweight Particles	C 123	D
Specific Gravity	C 127 & 128	C
Absorption	C 127 & 128	C
Unit Weight	C 29	C
Los Angeles Abrasion (coarse)	C 131	E
Soundness	C 88	E
Effect of Organic Impurities on Fine Aggregate	C 87	F
Petrographic	C 295	F
Sand Equivalent Test	California Test Method 217	A
Cleanness Value	California Test Method 227-B	C

Frequency:

(a) Once each 100 tons, but not more than 10, nor less than one per day of production

(b) Once each 2,000 tons, but not less than one test per week during production

(c) Every 10,000 tons, or once every 10 days of production

(d) Every 20,000 tons, or once every 20 days of production

(e) Once for initial source approval, then once per 30,000 tons

(f) One per deposit

All tests except the Soundness Test (ASTM C 88) and Soft Particles (ASTM C 235) were also performed by PG&E on a periodic basis. Samples were taken at the place where the aggregate entered the batch bin.

(3) *Admixtures*

Admixtures conformed to the following ASTM standards:

(a)	Pozzolan	ASTM C 618	
(b)	Air Entraining Agent	ASTM C 260	
(c)	Water Reducing Agent	ASTM C 494	1.1.1, Type A

A certificate of compliance accompanied each load of admixture delivered to the construction site.

(4) *Water*

Water is clean and free from deleterious amounts of silt, oil, acids, alkali, salts, and organic substances. Chlorides, calculated as Cl, are limited to 1,000 ppm, and sulfates, calculated as SO_4, are limited to 1,000 ppm.

(5) *Concrete Mixing, Placing, and Testing*

The contractor was required to submit concrete mix designs meeting PG&E specification requirements. The mixes were designed in accordance with Method 2, Section 308, of ACI 301. PG&E's material testing laboratory made sample batches of the proposed mixes and tested them according to:

(a) ASTM C 192, Making and Curing Concrete Test Specimens in the Laboratory

(b) ASTM C 39, Compressive Strength of Molded Concrete Cylinders

(c) ASTM C 143, Slump of Portland Cement Concrete by the Pressure Method

For each design mix, 7-day and 28-day compressive strength tests were made on 6 x 12 inch cylindrical samples in the laboratory.

The contractor was required to submit lift drawings, which showed the location of all construction joints and embedded items, for approval by PG&E. The lift drawings were approved prior to concrete placement. At construction joints in all structural concrete, the surface of the hardened concrete was roughened to expose the coarse aggregate by either bush hammering, wet sandblasting, or cutting with an air-water jet. Prior to placing the next lift of concrete, the surface of the hardened, cleaned concrete was wetted and given a 1/2-inch coat of bonding mortar on all horizontal joints. The bonding mortar had the same sand-cement ratio as the concrete mix, and had a water-cement ratio such as to make a thick slurry but, at most, no greater than that for the concrete. Vertical joints in walls were provided with shear keys. Vertical joints were staggered by at least 6 inches.

The concrete was batched and mixed in an automatic batching and mixing plant located at the construction site. Approved concrete mixes were punched on cards, and the appropriate card was inserted into the control console to initiate batching. The console automatically printed out the quantities of each material in the batch, and the time, date, batch number, and mix identification for each batch. Prior to plant startup, all weighing equipment was certified. This equipment was periodically checked to ensure continuing accuracy.

A full-time PG&E inspector checked the batching and mixing operation. The maximum temperature of concrete at placement was as follows:

(1) 55°F, base slab

(2) 70°F, internal structure and exterior shell

The concrete was placed within 45 minutes after introduction of water to the mix.

Concrete placement was inspected by PG&E inspectors. The concrete was either maintained in a moist condition for 7 days by approved methods, or coated with an approved curing compound.

Concrete was sampled at the frequency required by ACI 301-66. Sampling concrete and making, curing, and testing specimens was in accordance with:

(1)	ACTM C 172	Sampling Fresh Concrete
(2)	ASTM C 31	Making and Curing Concrete Compressive and Flexural Strength Test Specimens in the Field
(3)	ASTM C 39	Compressive Strength of Molded Concrete Cylinders
(4)	ASTM C 143	Slump of Portland Cement Concrete
(5)	ASTM C 231	Air Content of Freshly Mixed Concrete by the Pressure Method

All taking and testing of concrete samples was done by qualified PG&E personnel.

Compressive strength tests were evaluated in accordance with ACI 214. PG&E specifications required that 95 percent of all cylinders tested meet or exceed the specified strength for 5000 psi concrete, and 90 percent meet or exceed the specified strength for 3000 psi concrete. The correlation between field specimens and design strengths was evaluated continuously during construction.

The average strengths and coefficients of variations of concrete tested were:

Mix	Design, psi	Cement[(a)], sacks/yd	Average Strength, psi	Coefficient of Variation	Number of Tests
Unit 1					
7AP	5000	7.5	6500	4.3%	11
8	5000	7.5	6400	6.5%	134
8A	5000	7.0	6220	8.3%	18
8AP	5000	6.6	6120	6.4%	43
9BP	3000	6.0	3800	7.0%	87
Unit 2					
8A	5000	7.0	6680	6.7%	40
8AP	5000	6.6	6200	7.1%	101

These coefficients of variation represent "excellent control" as defined in Table 2 of ACI 214-65.

Concrete in Unit 1 and 2 containments is Class AP for base slab and interior concrete and Class BP for cylinder and dome. Mixes designated 7AP, 8, 8A, and 8AP are Class AP. Mix 9BP is Class BP.

(a) Cement and pozzolan

3.8.1.6.2 Reinforcing Steel

Reinforcing steel is deformed billet-steel bar conforming to ASTM Designation A 615. All reinforcing bars in the containment structure are Grade 60, except for the following, which are Grade 40:

(1) Liner anchorages in the base slab

(2) Anchorages on the structural steel beams embedded at the base of the containment structure wall

Table 3.8-1 compares the program for testing of reinforcing bars at the DCPP to the requirements of RG 1.15, which was issued after construction at DCPP was partially complete. Table 3.8-1 also indicates those areas where the PG&E specification is more stringent than ASTM A 615.

Heat number identification was maintained on reinforcing steel from the start of manufacture through placement in the structure.

Physical and chemical test results were sent to the construction site with the first load of steel from each heat. Test values were checked by PG&E inspectors or quality control engineers.

Detailing was in accordance with ACI Standard 315-65, Manual of Standard Practice for Detailing Reinforced Concrete Structures. Bars to be bent were cold bent around pins of the following minimum diameters:

(1) Stirrups and ties - four times the bar diameter

(2) Number 8 bars or smaller - six times the bar diameter

(3) Numbers 9, 10, and 11 - eight times the bar diameter

(4) Numbers 14 and 18 - ten times the bar diameter

Fabrication tolerances were as follows:

(1) Cut length:

Number 14 and 18 bars	0 inch, -3/8 inch
All other bars	±1 inch

(2) Depth of truss bars:

Number 18 bars	±2 inch
All other bars	0 inch, -1/2 inch

(3)	Stirrups, ties, and spirals:	±1/2 inch
(4)	All other bends:	
	Number 14 and 18 bars	±1/2 inch
	All other bars	±1 inch

Placement tolerances were as follows:

(1)	Concrete cover to formed surfaces:	
	Number 14 and 18 bars	–1/2 inch, +2 inch
	All other bars	±1/2 inch
(2)	Longitudinal location of bends:	
	Number 14 and 18 bars	±2 inch
	All other bars	±1 inch
(3)	Depth of bars in slabs:	
	8 inches or less in thickness	±1/4 inch
	Over 8 inches in thickness	±1/2 inch
(4)	Lateral location in the plane of reinforcing:	±2 inch

Occasionally, reinforcing steel bars had to be moved to avoid interferences. In this situation, a bar could be moved, within the plane of the reinforcing layer or curtain, up to one-half the specified spacing. If this was not sufficient, the resulting arrangement was submitted to PG&E for approval. Also, if the bar had to be moved out of the reinforcing layer or curtain to avoid an interference by more than one bar diameter or the above tolerances, whichever was greater, the resulting arrangement was submitted to PG&E for approval.

Tack welding to reinforcing bars was not permitted.

Reinforcing steel placement was inspected by contractor quality control inspectors and by PG&E inspectors.

The average and minimum properties of the tested Number 18 bars in the containment structure were as follows (these values are also shown in Table 3.8-6B):

(1)	Yield	- minimum	61,750 psi
		- average	66,854 psi

(2)	Tensile	- minimum	93,750 psi
		- average	105,992 psi
(3)	Elongation	- minimum	7.0%
		- average	9.4%

3.8.1.6.3 Splices

(1) Cadweld Splices

Cadweld splices were used at all locations for primary reinforcing in the exterior shell and base slab. Cadweld splices were used in a few locations in the internal structure.

Quality control procedures for Cadweld splices are described in Table 3.8-2 that compares the program used at the DCPP to that required by SG 10. SG 10 was issued after construction at DCPP was partially complete.

The average and minimum strengths of tested Cadweld tensile samples were:

(a)	Minimum tensile strength	85,000 psi
(b)	Average tensile strength	97,725 psi
(c)	Number of tests	641
(d)	Number of Cadwelds placed	19,068

(2) Butt-welded Splices

Butt-welded splices were used in a few locations where there was insufficient room to properly mount the Cadweld crucible. The quality control measures applied are the same as those described in Section 3.8.2.6.3 for butt-welded splices.

(3) Lap Splices

Lap splices are in accordance with ACI 318-63.

3.8.1.6.4 Liner, Penetration Sleeves, and Penetration Internals

The containment structure liner is carbon steel, conforming to ASTM A 516, Carbon Steel Plates for Pressure Vessels for Moderate and Lower Temperature Service, Grade 70. This steel has a minimum yield strength of 38,000 psi, a minimum tensile strength of 70,000 psi, and a minimum elongation of 17 percent in an 8-gage length at failure. Charpy V-notch impact tests were performed at +20°F, in accordance with ASTM A 370.

Penetration sleeves conform to one of the following three material specifications:

(1) ASTM A 106, Seamless Carbon Steel Pipe for High Temperature Service, Grade B, with the additional requirement that Charpy V-notch impact tests be performed at 0°F

(2) ASTM A 333, Seamless and Welded Steel Pipe for Low Temperature Service, Grade 1, except that Charpy V-notch impact tests are performed at 0°F

(3) ASTM A 516, Carbon Steel Plates for Pressure Vessels for Moderate and Lower Temperature Service, Grade 70, to ASTM A 300, except that Charpy V-notch impact tests were performed at 0°F

For all three material specifications, the Charpy impact tests were in accordance with the requirements of Paragraph N-330 of ASME Section III, 1968 edition.

Penetration internals conform to the following material specifications:

(1) Equipment and personnel hatches are ASME SA 516, Grade 70, to SA 300 with Charpy impact values at 0°F, in accordance with paragraph N-330 of ASME Section III, 1968 edition

(2) Carbon steel flued heads are ASME SA 105, Grade II, with Charpy impact tests at 0°F, in accordance with paragraph NB-2300 of Section III, ASME B&PV Code, 1971 edition. Ultrasonic and magnetic particle inspections are performed in accordance with Paragraphs NB 2542 and NB 2545, respectively

(3) Stainless steel flued heads are ASME SA 182, Grade F 304. Ultrasonic and liquid penetrant inspections are performed in accordance with Paragraphs NB 2542 and NB 2546, respectively

Welded studs attached to the liner meet the requirements of ASTM A 108, Grade 1015-1018.

The Charpy impact test temperatures stated in the paragraphs above were selected to be at least 30°F below the lowest service temperature in accordance with the ASME B&PV Code, Section III, 1968 Edition for Class B (containment) vessels. For future repair, replacement, or alteration of ferritic containment pressure boundary material, the notch toughness test requirements of Section III, NE-2300 will be used in lieu of the original requirements. Charpy impact tests will be performed at or below the lowest service temperature and material 5/8 inch or less in thickness will be exempt from notch toughness testing. Further information on notch toughness testing of containment materials appears in Section 3.1.8.14.

Mill test reports certifying the physical and chemical properties of the liner plate delivered to the jobsite were required from the steel supplier. The average and minimum properties of liner plate are as follows:

The Charpy impact test temperature for the hexagonal collars was selected to be at least 30°F below the lowest service temperature in accordance with the ASME B&PV Code, Section III, 1968 Edition for Class B (containment) vessels. For future repair, replacement, or alteration of the hexagonal collars, the notch toughness test requirements of Section III, NE-2300 will be used in lieu of the original requirements. Charpy impact tests will be performed at or below the lowest service temperature and material 5/8 inch or less in thickness will be exempt from notch toughness testing. Further information on notch toughness testing of containment testing materials appears in Section 3.1.8.14.

Reactor Pit and Floor Plates		Unit 1	Unit 2
Yield Strength	- minimum	43,800 psi	39,800 psi
	- average	51,400 psi	55,100 psi
Tensile Strength	- minimum	71,000 psi	74,000 psi
	- average	76,500 psi	78,900 psi
Elongation	- minimum	19%	17%
	- average	25%	24%
Total number of heats		16	11
Total number of slabs		58	62
Total number of tests		58	62

Cylinder and Dome		Unit 1	Unit 2
Yield Strength	- minimum	41,900 psi	38,100 psi
	- average	48,800 psi	46,100 psi
Tensile Strength	- minimum	70,200 psi	70,100 psi
	- average	74,900 psi	73,681 psi
Elongation	- minimum	19%	18%
	- average	26.5%	25.4%

Cylinder and Dome	Unit 1	Unit 2
Total number of heats	23	22
Total number of slabs	251	255
Total number of tests	251	255

Fabrication of the containment structure liner conforms to the applicable parts of Part UW, Requirements for Unfired Pressure Vessels Fabricated by Welding, Section VIII, ASME Boiler and Pressure Vessel Code.

All of the welds were visually examined by contractor quality control inspectors. All field welds were also visually examined by PG&E inspectors.

Table 3.8-3 compares the program for nondestructive testing of containment structure liner welds, including penetration sleeves and inserts, used on the DCPP to that required by SG 19, which was issued after construction at DCPP was partially complete.

Erection tolerances for the liner were as follows:

The liner of the completed structure shall be substantially round. At points not more than 4 inches above the base, the radius of the 3/4-inch liner shall be 69 feet 11-13/16 inches plus or minus 1/2-inch. The maximum diameter of the 3/8-inch liner shall not exceed 140 feet 4 inches and the minimum diameter shall not be less than 139 feet 8 inches.

The liner shall be erected true and plumb. At any point the out-of-plumb shall not exceed 1/240 of the height of the point above the base. For any plate (10 feet ± in height), the out-of-plumpness shall not exceed 1/120.

Flat spots or local out-of-roundness shall not exceed 2 inches in 15 feet of arc.

The base liner shall not deviate from a plane surface between anchorages by more than 1/240.

Stud welding was in accordance with the Supplement to AWS Specification D1.0-66. The tolerance of the location of each stud was ± 1/2-inch. At the beginning of each work day, each welder attached at least two test studs that were then tested by bending the stud approximately 45 percent toward the plate to demonstrate the integrity of the stud-to-plate weld. If failure occurred in the weld, the welding procedure or technique was corrected and two successive studs successfully welded and tested before further studs were attached to the liner plate. These test studs were allowed to remain in place but are not considered as a part of the regular stud pattern required by the design. A 100 percent visual inspection of liner and stud anchors was made prior to pouring concrete.

3.8.1.6.5 Structural Steel

Hexagonal collars at the equipment hatch and personnel hatch meet the requirements of ASTM A 516, Grade 70, and ASTM A 300, except that Charpy V-notch impact tests were performed at 20°F.

The following quality control procedures were followed in the fabrication of the hexagonal steel collars at the equipment hatch and personnel hatch openings:

(1) The 4-inch-thick plate for the edge pieces was ultrasonically examined in accordance with ASTM A 435, except that scanning covered 100 percent of the surface.

(2) Fabrication conformed to the applicable parts of Part UW, Requirements for Unfired Pressure Vessels Fabricated by Welding, of Section VIII of the ASME Boiler and Pressure Vessel Code. All welds are full penetration butt welds and were 100 percent radiographed in accordance with Paragraph UW-51.

(3) The reinforcement plates were heat treated after fabrication in accordance with Paragraph UCS-56, Requirements for Postweld Heat Treatment, of Section VIII of the ASME Boiler and Pressure Vessel Code.

3.8.1.7 Testing and Inservice Surveillance Requirements

After each containment structure was complete, with liner, concrete, and all electrical and piping penetrations, equipment hatch and personnel locks in place, tests were performed as discussed in the following sections.

3.8.1.7.1 Structural Integrity Test

The structural integrity test was performed by pressurizing the containment structure with air up to 115 percent of design pressure, or 54 psig. During this test, structural deflections were measured, crack patterns in the concrete were measured and photographed, and strains in the liner and reinforcing steel measured electrically and recorded. The deflections, crack patterns, and strains were compared to the theoretical predictions to verify the structural integrity of the containment structure. The structural integrity test of each containment structure meets the requirements of RG 1.18, Structural Acceptance Test for Concrete Primary Reactor Containments.

The Unit 1 containment structure is a prototype concrete primary reactor containment, as defined in RG 1.18.

For the structural integrity test, the pressure was increased in increments to the maximum of 54 psig. Measurements were made at 0, 15, 25, 35, 47, and 54 psig during pressurization and again during depressurization. At each pressure level, the deflection and strain gage readings were made after a 1-hour wait to allow adjustment of strains. The crack patterns were recorded both before and immediately after the test and at the maximum pressure level achieved during the test.

The instrumentation for Unit 1 was as follows:

The radial and vertical growth was measured by means of calibrated targets attached to the exterior shell and sighted by means of high magnification theodolites. Radial deflections were measured at three points on each of six equally spaced meridians: at the springline, at mid-height of the cylinder, and at the top of the base slab. Vertical deflections were measured at the springline and at the top of the dome.

The radial and tangential deflections of the containment structure wall were measured at twelve locations adjacent to the equipment hatch, which is the largest opening.

The pattern of cracks that exceed 0.01-inch in width were mapped or photographed near the base-wall intersection, at mid-height of the wall, at the springline of the dome, and around the equipment hatch. At each location, an area of at least 40 square feet was mapped or photographed.

Strain measurements were made at the following locations, in accordance with the requirements for prototype containment structures:

(1) In the wall at the top of the base mat

(2) In the wall at the equipment hatch, with one gauge located approximately 0.5 times the wall thickness from the edge of the opening

(3) In the wall at the level of the springline

(4) In the wall where pure membrane stress is anticipated, i.e., where there are no discontinuities

Inasmuch as the concrete is assumed cracked, and the strength of the concrete is neglected, strain measurements were made on the reinforcing steel and liner, rather than in the concrete. At the equipment hatch, additional strain measurements were made on the structural steel hexagonal collar. In the wall at the top of the base slab, additional strain measurements were made on the structural steel wide flange beams.

The method used for attaching strain gauges to Number 18 reinforcing bars is shown in Figure 3.8-44.

In evaluating the results of the structural integrity test, the deflection measurements were considered the most reliable result.

Strains, deflections, and crack patterns were compared to the theoretical predictions. The acceptance criterion was that actual readings had to be within 20 percent of the predicted values. This criterion was based on an evaluation of:

(1) Residual stress due to concrete shrinkage

(2) Measurement errors

(3) Temperature variations

(4) As-built deviations of the containment shell from a circular shape

(5) Actual results of other structural integrity tests (primarily at the R. E. Ginna plant)

The deflection measurement and crack mapping program for Unit 2 was identical to that for Unit 1.

3.8.1.7.2 Overall Integrated Leakage Rate Tests

During the depressurization phase of the structural integrity test, the sequence was stopped at 47 psig to conduct an overall integrated leakage rate test at design pressure.

During the overall integrated leakage rate tests, the double penetration and weld seam leak chase channel zones were open to the atmosphere inside the containment structure.

All leakage rate tests are conducted and evaluated in accordance with Appendix J of 10 CFR 50, Option B, as modified by approved exemptions.

3.8.1.7.3 Sensitive Leakage Rate Tests

A sensitive leakage rate test can be performed at some future date with only the volume of the weld seam leak chase channels and double penetrations included in the test. A sensitive leakage rate test would be performed with penetrations and leak chase channels at not less than the peak calculated containment internal pressure (Pa), and with the containment structure at atmospheric pressure.

3.8.1.7.4 Inservice Surveillance Requirements

Periodic leakage rate testing is performed in accordance with the requirements of Appendix J of 10 CFR 50. Inservice surveillance to ensure continued containment integrity is discussed in Section 6.2.1.4. Instrumentation employed to monitor containment status is described in Section 6.2.1.5.

3.8.2 OTHER DESIGN CLASS I STRUCTURES (AUXILIARY BUILDING)

3.8.2.1 Description of the Auxiliary Building

The auxiliary building is located between the Unit 1 and Unit 2 containment structures. It contains the control room that includes consoles and a fuel handling area for each unit. In addition, the auxiliary building contains equipment for the chemical and volume control systems, the safety injection systems, the residual heat removal systems, the component cooling water systems, the liquid radwaste systems, the gaseous radwaste system, and others.

The main floor levels in the auxiliary building are at elevations 85, 100, 115, and 140 feet. Elevations 60 and 73 feet are below ground level, which is at elevation 85 feet, except for the east side of the building where ground level is at elevation 115 feet. Floor plans at elevations 100, 115, and 140 feet are shown in Figures 3.8-60, 61, and 62.

The foundation of the auxiliary building is divided between 3 elevations. The structure is supported at elevations 85 feet (areas GE, GW, and L) 100 feet (area J), and elevation 60 feet (areas H and K).

Figure 1.2-2, Plant Layout, shows relative locations of the plant buildings. The general arrangement of equipment in the auxiliary building, including the fuel handling areas, is shown in Figures 1.2-4 through 1.2-11, Figures 1.2-21 through 1.2-26, and Figures 1.2-29 and 1.2-30. Generally, one-half of the auxiliary building is a mirror image of the other, with each half of the structure containing equipment for one unit. The control room is located at elevation 140 feet. The two fuel handling areas that contain the spent fuel pools, the fuel handling cranes, fuel racks, and related equipment are located on each side of the east end of the auxiliary building with the top of the spent fuel pools at elevation 140 feet.

The auxiliary building is a reinforced concrete shear wall structure, except for the fuel handling area crane support structure which is a structural steel moment resisting and braced frame structure supported on elevation 140 feet and extending up to elevation 188 feet. The shear walls and slabs of the auxiliary building are generally 2 feet thick. The walls of the spent fuel pools are 6 feet thick except for local areas around the fuel transfer tubes. The foundation slabs under the spent fuel pools have a minimum thickness of 5 feet. The spent fuel pool sides and bottoms are lined with stainless steel, 1/4-inch-thick on the bottoms and 1/8-inch nominal thickness on the sides. Representative concrete outlines, reinforcing steel arrangements, and structural steel details for the auxiliary building are shown in Figures 3.8-45 through 3.8-59.

The 125-ton overhead crane in the fuel handling area, shown in Figure 3.8-59, is equipped with restraints that prevent derailing from motions associated with an earthquake.

The only connections between the auxiliary building and other structures are the fuel transfer tube and miscellaneous piping. The fuel transfer tube is fitted with expansion bellows that allow relative movement between the auxiliary building, the containment structure exterior shell, and the internal structure of the containment structure. The design of the expansion bellows considers the maximum axial and lateral relative deflection. Piping systems are analyzed for the maximum relative displacements of the auxiliary building and other structures, and the piping anchor points in the structures are designed to withstand the resulting forces.

3.8.2.2 Applicable Codes, Standards, and Specifications

The following codes and standards are used in the design, construction, inspection, and testing of the auxiliary building:

(1) ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-63), except that design loading combinations are as described in Section 3.8.2.3.2

(2) Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-65)

(3) Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-65)

(4) Inspection of the Cadweld Rebar Splice (Erico Products, Inc., RB-5M768)

(5) Recommended Practices for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction, American Welding Society AWS D12.1-61

(6) AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, February 12, 1969

(7) Code for Welding in Building Construction, AWS D1.0-69. Work performed prior to December 12, 1969 is in accordance with the earlier edition, AWS D1.0-66. For inspection of non-ASME structural welds or new non-ASME work performed after January 1, 1988, the guidelines of Nuclear Construction Issues Group (Visual Weld Acceptance Criteria, Vol. 1-3, EPRI Report No. NP-5380, September 1987) may be used except for those cases where:

    (a) Fatigue is a governing design condition.

    (b) The weld allowables are permitted to be higher than those allowed by AWS D1.1 (such as the full penetration welds evaluation for HE).

(c) The weld is part of work performed in the ASME Section XI Inservice Inspection Program.

(8) Stud welding is in accordance with the Supplement to American Welding Society Specifications AWS D1.0-66 and AWS D2.0-66 on Requirements for Stud Welding

(9) Materials and the quality control tests for materials conform to ASTM standards

RG 1.15, Testing of Reinforcing Bars for Category I Concrete Structures (dated December 28, 1972), and RG 1.55, Concrete Placement in Category I Structures (dated June 1973), were issued after construction of the DCPP was nearly complete. A comparison of the program used for the DCPP with the regulatory position of RG 1.15 is presented in Table 3.8-1. The quality assurance program for the DCPP meets the requirements of RG 1.55. In regard to RG 1.55, the references used for guidance are those listed in Appendix A of the RG, as they existed at the time of the PSAR.

3.8.2.3 Loads and Loading Combinations

3.8.2.3.1 Design Loads

The following loads are considered in the design of the auxiliary building.

3.8.2.3.1.1 Dead Loads

Dead loads consist of the weight of the structure, and permanent equipment loads.

3.8.2.3.1.2 Live Loads

Live loads consist of temporary equipment loads and a uniform load to account for the miscellaneous temporary loadings that may be placed on the structure.

3.8.2.3.1.3 Earthquake Loads

Earthquake loads are based on a time-history modal superposition analysis. This analysis is described in Section 3.7.2.

3.8.2.3.1.4 Wind Loads

Wind loads are determined in accordance with the criteria presented in Section 3.3. However, considering the UBC and ASCE Paper 3269 pressures, the forces due to wind are much less than those due to earthquake; consequently, seismic loads, rather than wind, are entered into the load combination equations.

3.8.2.3.1.5 Thermal Loads

Thermal loads are loads induced by local increases in temperature. Thermal loads result from normal operating conditions and from postulated accident conditions.

3.8.2.3.1.6 Pipe Reaction Loads

Pipe reactions that result from hydraulic forces, thermal expansion, and seismic events, are transferred to the structure through pipe supports. Pipe reaction loads result from normal operating conditions and postulated accident conditions.

3.8.2.3.1.7 Jet and Missile Loads

Jet and missile loads are localized forces on structures in the immediate vicinity of a postulated pipe break. Jet forces result from the impingement of high energy fluid on an object. Missile forces result when a part possessing kinetic energy strikes an object.

Missile forces are calculated by the methods described in Section 3.5. Jet forces and pipe reactions from a postulated broken pipe are calculated as described in Section 3.6.

3.8.2.3.1.8 Pressure Loads

Pressure loads are forces generated by a postulated pipe break. Pressures from a postulated broken pipe are calculated as described in Section 3.6.

3.8.2.3.2 Loading Combinations

3.8.2.3.2.1 Normal Conditions

Dead load, live load, loads from the DE, thermal loads, and pipe reactions are considered in all possible combinations. Inasmuch as working stress design is used for normal operating loads, the factored load approach is not used. For each structural member, the combination of these loads that produces the maximum stress is used for design. Stated in equation form:

$$C = D + L + DE + T_o + R_o \qquad (3.8\text{-}14)$$

where:

C	=	required capacity of member based on the methods described in Section 3.8.2.5.1
D	=	dead load of structure and equipment loads
L	=	live load
DE	=	loads resulting from the DE
T_o	=	thermal loads during normal operating conditions
R_o	=	pipe reactions during normal operating conditions

3.8.2.3.2.2 Abnormal Conditions

Dead load, live load, earthquake loads, and loads associated with accidental pipe rupture are considered in the following combinations; for each structural member, the combination that produces the maximum stress is used for design:

Concrete Structural Elements

$$U = D + L + T_A + R_A + 1.5\ P_A \qquad (3.8\text{-}15)$$

$$U = D + L + T_A + R_A + 1.25\ P_A + 1.0\ (Y_j + Y_m + Y_r) + 1.25\ DE \qquad (3.8\text{-}16)$$

$$U = D + L + T_A + R_A + 1.0\ P_A + 1.0\ (Y_j + Y_m + Y_r) + DDE \qquad (3.8\text{-}17)$$

$$U = D + L + T_A + R_A + 1.0\ P_A + 1.0\ (Y_j + Y_m + Y_r) + HE \qquad (3.8\text{-}18)$$

where:

T_A = thermal loads on structure generated by a postulated pipe break, including T_O

R_A = pipe reactions on structure from unbroken pipe generated by postulated pipe break conditions, including R_O

PA = pressure load within or across a compartment and/or building generated by a postulated pipe break, and including an appropriate dynamic factor (DLF) to account for the dynamic nature of the load

Y_j = jet load on structure generated by a postulated pipe break, including an appropriate DLF

Y_m = missile impact load on a structure generated by, or during, a postulated pipe break, such as a whipping pipe, including an appropriate DLF

Y_r = reaction on structure from broken pipe generated by a postulated pipe break, including an appropriate DLF

U = ultimate strength required to resist design loads based on the methods described in ACI 318-63. See Section 3.8.2.5.2 for equation 3.8-16 through 3.8-18

DDE = loads resulting from the DDE

HE = loads resulting from an HE

Steel Structural Elements

Where elastic working stress design methods are used[(a)(b)]:

$$1.6S^{(a)} = D + L + T_A^{(b)} + R_A + P_A \quad (3.8\text{-}19)$$

$$1.6S^{(a)} = D + L + T_A^{(b)} + R_A + P_A + 1.0(Y_j + Y_m + Y_r) + DE \quad (3.8\text{-}20)$$

$$1.6S^{(a)} = D + L + T_A^{(b)} + R_A + P_A + 1.0(Y_j + Y_m + Y_r) + DDE \quad (3.8\text{-}21)$$

$$1.7S = D + L + T_A^{(b)} + R_A + 1.0P_A + 1.0(Y_j + Y_m + Y_r) + HE \quad (3.8\text{-}22)$$

Where plastic design methods are used:

$$0.90Y^{(a)} = D + L + T_A^{(b)} + R_A + 1.5P_A \quad (3.8\text{-}23)$$

$$0.90Y^{(a)} = D + L + T_A^{(b)} + R_A + 1.25P_A + 1.0(Y_a + Y_m + Y_r) + 1.25DE \quad (3.8\text{-}24)$$

$$0.90Y^{(a)} = D + L + T_A^{(b)} + R_A + 1.0P_A + 1.0(Y_j + Y_m + Y_r) + DDE \quad (3.8\text{-}25)$$

$$1.0Y = D + L + T_A^{(b)} + R_A + 1.0P_A + 1.0(Y_j + Y_m + Y_r) + HE \quad (3.8\text{-}26)$$

where:

S	=	required section strength based on elastic design methods and the allowable stresses[(c)] defined in Part 1 of the AISC "Specifications for the Fabrication and Erection of Structural Steel for Buildings," February 12, 1969
Y	=	required section strength based on plastic design methods[(c)] described in Part 2 of AISC Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings, February 12, 1969

(a) For existing structures, the 1.6 factor applied to the required section strength, S, and the 0.90 reduction factor applied to the required section strength, Y, are increased to 1.7 and 1, respectively. In such situations, however, it is verified that deflections will not result in the loss of function of any safety-related system.

(b) Thermal loads are neglected when it can be shown that they are secondary and self-limiting in nature and where the material is ductile.

(c) See Section 3.8.2.5 regarding material properties used in conjunction with load combinations including HE.

For both concrete and steel structural elements, both cases of L having its full value present during the postulated pipe rupture, or being completely absent, are checked.

3.8.2.4 Design and Analysis Procedures

Structural analysis of the auxiliary building is performed by the traditional methods of engineering analysis for structural steel and reinforced concrete structures. These methods are based on the principles of equilibrium, compatibility of deformations, and predictions of material strength by the methods of the AISC Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings (AISC Code), and the ACI Standard Building Code Requirements for Reinforced Concrete (ACI Code).

The use of these codes is discussed in Section 3.8.2.5. The following sections discuss the specific design methods used for the structural steel and concrete parts of the auxiliary building.

3.8.2.4.1 Structural Steel

The fuel handling area crane support structure is a 370 x 60 x 50-foot high steel framed structure clad with metal siding and covered by metal decking and built-up roofing. The structure is supported on concrete walls at elevation 140 feet on the eastern side of the auxiliary building. Lateral forces are resisted by steel cross-braced frames in the north-south direction, moment resisting frames in the east-west direction, and by the roof, which is a trussed and cross-braced diaphragm covered with metal decking.

Acceleration profiles and forces from the analyses described in Section 3.7.2.1.7.1 are applied to a detailed static model of the entire fuel handling area crane support structure to obtain forces for the stress evaluation of the structural members and connections and to obtain structural displacements. Various crane and lifted load positions are considered. The stress evaluation is carried out for the load combinations described in Section 3.8.2.3. Stresses are evaluated against the criteria in Section 3.8.2.5. The calculated stress ratios for the most critical members are given in Table 3.8-7.

3.8.2.4.2 Concrete

The vertical and the lateral load-resisting system of the auxiliary building consists of reinforced concrete columns and walls tied together with reinforced concrete slabs. The evaluation of these elements is carried out for the loading combinations given in Section 3.8.2.3, according to the criteria in Section 3.8.2.5.

The seismic forces and moments are based on the response of the auxiliary building seismic models described in Section 3.7. A detailed analytical model of the auxiliary building is then developed to distribute forces and moments to the various walls, diaphragms, and columns.

Slabs

Slabs are evaluated separately for out-of-plane loads and in-plane loads.

For out-of-plane loads, shear stresses and moments are calculated assuming one-way or two-way slab action as appropriate. Out-of-plane capacity-to-demand ratios for selected slabs are shown in Tables 3.8-8 through 3.8-10. The slab in-plane capacities are investigated at critical sections. The selection of these sections is based on the location of numerous openings across the entire section of the diaphragm and the magnitudes of shear, moment, and axial forces on the entire section. The in-plane capacity-to-demand ratios for the concrete slabs are shown in Tables 3.8-11 through 3.8-13.

Walls

The critical wall elements are selected based on the magnitude of demand loads and presence of openings. The forces and moments for the governing load combination in those elements are compared to their respective capacities, and the capacity-to-demand ratios are shown in Tables 3.8-14 through 3.8-16.

Concrete Columns

The concrete columns are evaluated for axial and flexural loads. Flexural loads on columns due to the vertical loads are determined by considering frame action with the slabs. Flexural loads include the effect of minimum eccentricity specified by ACI 318-63. Column moments due to interstory drift are found to be negligible. Capacity-to-demand ratios for selected columns are shown in Table 3.8-17.

3.8.2.4.3 Load Dissipation to the Foundation

The adequacy of the structural system, at and below elevation 85 feet, to dissipate lateral loads to the rock foundation is evaluated for the load combinations given in Section 3.8.2.3.2. The results, given in Tables 3.8-18 through 3.8-23, and illustrated for HE loads in Figures 3.8-63 and 3.8-64, indicate the adequacy of the system to dissipate the loads.

3.8.2.4.4 Computer Programs

Computer programs used in the structural analysis, and the verification measures used, are listed in Table 3.8-6.

3.8.2.5 Structural Acceptance Criteria

For DE and DDE load combinations, the nominal design strength for concrete and specified yield strength for reinforcing and structural steel are considered. For load combinations including HE, however, the actual material properties are used. See

Sections 3.8.2.6.1, 3.8.2.6.2, and 3.8.2.6.4 for material properties associated with concrete, reinforcing steel, and structural steel, respectively. See Section 3.8.2.6.5 for allowable stresses for bolted connections. Ductility, when applied for HE load combinations, is in accordance with Table 3.8-24.

3.8.2.5.1 Normal Loads

For normal loads, the auxiliary building is designed for the allowable working stresses of ACI 318-63, as supplemented by Section 3.8.2.5.3, and Part 1 of the AISC Code, except that the increase in allowable stress usually allowed for load combinations involving earthquake forces is not used.

3.8.2.5.2 Abnormal Loads

For abnormal loads, the auxiliary building is designed for overall elastic behavior. For concrete elements, the strength design method of ACI 318-63 applies, as supplemented by Section 3.8.2.5.3. For the evaluation of steel elements using elastic design methods, the allowable stresses are defined in Part 1 of AISC Code. For the evaluation of steel elements using "plastic design," Part 2 of the same AISC specifications applies.

The capacity for the various concrete structural elements is based on the yield strength of the material, reduced by a factor, f, which provides for the possibility that small, adverse variations in material strengths, workmanship, dimensions, and control, while individually within required tolerances and the limits of good practice, occasionally may be additive. The f-factors used are in accordance with ACI 318-63.

For load combinations involving Y_j, Y_m, and Y_r, local stresses due to these concentrated loads may exceed the allowable provided there is no loss of function. See Reference 6, Enclosure Number 3, Document (B), for more detailed information concerning the acceptance criteria for these load combinations.

3.8.2.5.3 In-Plane Loads on Concrete Elements

The design of slab diaphragms and shear walls for in-plane forces is not explicitly covered by ACI 318-63. Section 104 of ACI 318-63 allows criteria based on test data to be used for the design of elements not covered by its provisions. Consequently, the document entitled "Recommended Evaluation Criteria for Diablo Canyon Nuclear Power Plant Auxiliary Building Walls and Diaphragms" (Reference 7) is developed to provide criteria for evaluation of auxiliary building shear walls and floor diaphragms for in-plane seismic forces, including the simultaneous effects of out-of-plane forces.

Accordingly, the structural elements are evaluated as follows:

(1) The columns are evaluated by the provisions of ACI 318-63 for all loading conditions.

(2) The slabs and walls are evaluated for out-of-plane loads according to ACI 318-63, and for in-plane loads according to Reference 7.

3.8.2.5.4 Factors of Safety

The calculated capacity-to-demand ratio for selected structural elements of the auxiliary building are given in Tables 3.8-7 through 3.8-17. In all cases, these ratios are greater than the minimum allowable value of 1. Therefore, all structural elements satisfy the criteria.

The gap between the auxiliary building and the containment structure, as well as the factor of safety against the structure impacting during a seismic event, is discussed in Section 3.8.1.5.3. Separations between the auxiliary building and the turbine building are adequate to ensure these structures will not impact each other when subject to design load combinations. Calculated displacements, separations, and factors of safety against impact are shown in Table 3.8-23A.

3.8.2.6 Materials, Quality Control, and Special Construction Techniques

Sections 3.8.1.6.1 and 3.8.1.6.2 for the containment structure also apply to the auxiliary building, except as superseded by information in the following paragraphs.

3.8.2.6.1 Concrete

Concrete strengths are shown below.

Walls and slabs below elevation 85 feet; slabs 4 feet and thicker at elevation 85 feet; slab at elevation 85 feet bounded by column lines 16.8 - 19.2 - L - H:

Design f'c = 3000 psi (DE and DDE combinations)

All other concrete:

Design f'c = 5000 psi (DE and DDE combinations)

Average 28-day strengths are used with HE load combinations. The average strengths of representative mixes are as follows:

Design Strength	Average 28-day Strength	Number of Tests
3000 psi	3920 psi	167
5000 psi	5650 psi	368

3.8.2.6.2 Reinforcing Steel

Reinforcing steel is ASTM A 615, Grade 40, except in some locations where Grade 60 is used. ASTM minimum values are used with DE and DDE load combinations. Average test values are used with HE load combinations. The average and minimum properties of representative bar sizes, are as follows:

	#8		#11	
	Grade 40	Grade 60	Grade 40	Grade 60
Design Yield Strength, psi	40,000	60,000	40,000	60,000
Average Yield Strength, psi	49,655	66,189	48,302	68,582
Minimum Yield Strength, psi	41,200	60,250	42,950	61,710
Average Tensile Strength, psi	82,236	102,403	81,074	105,822
Minimum Tensile Strength, psi	74,392	96,500	72,940	94,390
Average Elongation, %	18.9	13.94	14.82	14.41
Minimum Elongation, %	13.0	11.0	8.5	9.4
Total Number of Heats	67	18	91	56

3.8.2.6.3 Splices

The majority of splices in the auxiliary building are lap splices, made in accordance with ACI 318-63. Cadweld splices are used in some locations in the auxiliary building. The quality control procedures described for Cadweld splices in the containment structure also apply to Cadweld splices in the auxiliary building.

Butt-welded splices are used where a section of wall has to be temporarily left open for access, and in certain other locations. Butt-welded splices are made in accordance with ACI 318-63, and the American Welding Society's Recommended Practices for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction, using the "short-arc" process or low hydrogen stick electrodes by the shielded arc process. Both processes have minimum preheat and interpass temperatures of 400°F. Completed welds are wrapped with a protective blanket of insulating material to avoid rapid cooling.

Procedure qualification and welder qualification are as follows:

(1) A welding procedure qualification test is made for each position and for each grade and size of bar. The test consists of two tension tests and one nick break test. Bars may not be rolled during welding.

(2) Welder qualification tests are made for each position, type of electrode, grade and size of bar, and joint design. Qualification for one size of bar is considered qualification for all smaller sizes. Each test consists of one tension and one nick break test. Bars may not be rolled during welding.

(3) Tension specimens are tested to failure and must comply with the minimum tensile requirements for the grade of reinforcing steel.

(4) The nick break specimen is broken and visually examined for soundness. The specimen must exhibit the following: the sum of the longest dimension of all inclusions visible in any one joint must not exceed ½-inch; no inclusion may be closer to the weld surface than a distance equal to the largest dimension of the inclusion; there must be no incomplete fusion or lack of penetration or cracks in the weld or base metal.

Testing percentages applicable to butt-welded splices for each welder, position, and grade of bar are as follows:

(1) Two out of the first ten splices

(2) Six out of the next 90 splices

(3) Four out of second and subsequent 100 splice units

Qualification for one size of bar is considered as qualification for all smaller sizes.

3.8.2.6.4 Structural Steel

Structural steel is ASTM A 36 and ASTM A 441; ASTM minimum values are used with DE and DDE load combinations, and average test values are used with HE load combinations. Minimum and average values are as follows:

	ASTM A36	ASTM A441
Design Yield (ksi)	36	42

Testing of structural steel installed through 1977 gave the following:

	ASTM A36	ASTM A441
Average Test Yield (ksi)	43.95	51.62
Average Test Ultimate (ksi)	68.04	75.91

Charpy impact tests were performed on all structural steel at the following temperature:

Framing for pipe rupture restraints	40°F
Structural steel	20°F

3.8.2.6.5 Structural Bolts

Structural bolts are ASTM A307, A325, and A490, allowable stresses per Table 1.5.2.1 of the AISC "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings," February 12, 1969, are used with the DE, DDE, and HE load combinations. However, it is acceptable to increase the allowable stresses for the Hosgri load combinations, based on the results of properly substantiated testing (References 34 through 38).

3.8.3 OUTDOOR WATER STORAGE TANKS

3.8.3.1 Description of the Outdoor Water Storage Tanks

The Design Class I outdoor storage tanks, located adjacent to east of auxiliary building, are steel studded tanks with concrete shielding, as shown in Figure 3.8-65, sheets 1 and 2.

There are two refueling water storage tanks and two condensate water storage tanks, one to service each unit of the plant. The firewater and transfer tank, which serves both Units 1 and 2, is made up of two concentric cylindrical steel tanks connected by a common dome roof. The inner cylindrical tank is the firewater tank and the outer tank is the transfer tank. The structural configuration of the condensate tanks is similar to that of the refueling water storage tanks.

The Design Class I tanks are supported on concrete fill down to bed rock and are anchored to bed rock with rock anchors as shown in Figure 3.8-65, Sheet 2.

3.8.3.2 Applicable Codes, Standards, and Specifications

The following codes and standards are used in the design, construction, inspection, and testing of the outdoor water storage tanks.

(1) ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-63, and ACI 318-71)

(2) Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-65)

(3) Recommended Practices for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction, American Welding Society AWS D12.1-75

(4) AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, 6th and 7th Editions

(5) Code for Welding in Building Construction, AWS D1.1-77, Rev. 2. Work performed prior to (December 12, 1969) is in accordance with the earlier edition, AWS D1.0-66. For inspection of non-ASME structural welds or new non-ASME work performed after January 1, 1988, the guidelines of Nuclear Construction Issues Group (Visual Weld Acceptance Criteria,

Vol. 1-3, EPRI Report No. NP-5380, September 1987) may be used except for those cases where:

(a) Fatigue is a governing design condition.

(b) The weld allowables are permitted to be higher than those allowed by AWS D1.1 (such as full penetration weld evaluation for the HE).

(c) The weld is part of work performed in the ASME Section XI Inservice Inspection Program.

(6) Stud loading is in accordance with the Supplement to American Welding Society Specifications AWS D1.0-66 and AWS D2.0-66 on Requirements for Stud Welding

(7) Materials and quality control tests for materials conform to ASTM standards

(8) ASME Section VIII, Division 2, 1974

(9) AWWA D100, American Waterworks Association, Standard to Steel Tanks, Standpipes Reservoirs and Elevated Tanks for Water Storage

3.8.3.3 Loads and Loading Combinations

3.8.3.3.1 Normal Conditions

$$C = D + HS + 1.0\ DE + 1.0\ R_O \qquad (3.8\text{-}27)$$

where:

C = required load capacity of section as described in Section 3.8.3.5.1
D = dead load of tank
HS = hydrostatic load
DE = loads resulting from DE
R_O = pipe reactions during normal operating conditions, including dead, thermal, and DE loads.

3.8.3.3.2 Abnormal Conditions

$$U = D + HS + 1.0\ DDE + 1.0\ R_A \qquad (3.8\text{-}28)$$

$$U = D + HS + 1.0\ HE + 1.0\ R_A \qquad (3.8\text{-}29)$$

where:

D and HS are defined in Section 3.8.3.3.1, and

U	=	strength required to resist abnormal loads as described in Section 3.8.2.5.2
DDE	=	loads resulting from the DDE
HE	=	loads resulting from the HE
R_A	=	pipe reactions during abnormal conditions, including dead, thermal, and DDE or HE loads

3.8.3.4 Design and Analysis Procedures

Condensate storage tanks, the refueling water storage tanks, the primary water storage tanks, and the fire water and transfer water tanks were originally designed to meet the criteria based on the DE, DDE, and HE. The code used in designing the tanks was AWWA D100, 1967, with stress allowables restricted to those permitted by ASME Section VIII, Division 2. Revised security criteria called for additional resistance to bullet penetration and explosives and required all the above steel tanks, except the primary water storage tanks, to be modified with minimum reinforced concrete protection as determined by Argonne National Laboratory. The Design Class I tanks with concrete protection were reevaluated for DE, DDE, and HE using finite-element computer models as described in Section 3.7.

The stresses in the stiffeners and the steel liner plate around the vault openings were determined by hand calculation and compared with the stress allowables. The forces, moments, and stresses in the refueling water storage tank resulting from dead load, hydrostatic pressure, hydrodynamic loads, and seismic loads are listed in Table 3.8-25. These values are within the allowable limits as described in Section 3.8.3.5.

3.8.3.5 Structural Acceptance Criteria

3.8.3.5.1 Normal Loads

For normal loads, the outdoor water storage tanks are designed for the allowable working stresses of the ACI 318-63 for concrete, Part 1 of the AISC Specification, 6th Edition for structural steel components, and ASME Section VIII, Division 2, 1974, for steel liner plates.

3.8.3.5.2 Abnormal Loads

For abnormal loads, the outdoor water storage tanks are designed for overall elastic behavior. For concrete elements the strength design method of ACI 318-63 applies for DDE loads and of ACI 318-71 for HE loads. For the evaluation of structural steel elements using elastic design methods, for the loading condition including DDE loads, 1.6 times AISC 6th Edition, Part 1 allowables are used; whereas, for the HE load combination, Part 2 of AISC 7th Edition, the Plastic Design method applies.

The steel liner plates are evaluated by using stress intensities as defined in ASME Section VIII, Division 2, 1974, and applying a factor of 0.9 to the minimum specified yield strength for DDE, and 1.0 to the yield strength based on test results for HE load conditions. For local stress intensities around nozzles in the vault opening area for DDE loads, a factor of 1.0 to the minimum specified yield strength applies, whereas for HE loads, a factor of 2.4 to the ASME Section VIII, Division 2, 1974, allowable values applies.

3.8.3.5.3 Factors of Safety

The calculated capacity-to-demand ratios for critical structural elements of the outdoor water storage tanks are greater than the minimum allowable value of 1. Therefore, all structural elements satisfy the criteria.

3.8.3.6 Materials, Quality Control, and Special Construction Techniques

The quality control measures discussed in Sections 3.8.1.6.1 and 3.8.1.6.2 for the containment structure also apply to Design Class I tanks.

3.8.3.6.1 Materials Tank Walls and Roof Dome

Condensate and Inner and Outer Tank Walls of Firewater and Transfer Tank	-	Carbon steel liner plates and stiffeners conform to ASTM A-516 GR 60. The Condensate Storage Tanks are coated with an epoxy coating and the Fire Water and Transfer Tank is coated with Vinyl Paint. Carbon steel studs conform to ASTM A108 GR 1015- 1018.
Refueling Water Tank	-	Stainless steel liner plates and stiffeners conform to ASTM A-240 Type 304L. Stainless steel studs conform to ASTM A276 type 304 annealed.

All Tanks, Except Inner Tank of Firewater and Transfer Tank:

- Concrete strength (minimum specified) f_C = 4 ksi

- Rebar conforms to ASTM A615, GR 60, fy = 60 ksi (minimum specified)

3.8.4 FOUNDATIONS AND CONCRETE SUPPORTS

The foundation structures for the containment and the auxiliary building are included in Sections 3.8.1 and 3.8.2, respectively. The foundations of the Class I outdoor water storage tanks are described in this section.

3.8.4.1 Foundations for Design Class I Tanks

The following Design Class I concrete-protected steel tanks are located adjacent to the east side of the auxiliary building on reinforced concrete foundation slabs:

(1) Condensate water storage tank (one for each unit)

(2) Refueling water storage tank (one for each unit)

(3) Fire water and transfer tank (common to both units)

3.8.4.1.1 Description of the Foundation Slabs

Each of the condensate water storage tanks and refueling water storage tanks has a separate, circular foundation slab. The fire water tank and the transfer tank, which serves both units, are concentric tanks on a common circular foundation slab. Each of the foundation slabs is shown in Figure 3.8-65 and consists of a 1-foot-thick reinforced concrete slab with an integral edge beam tied to a reinforced concrete wall. Each of the tanks, except for the fire water tank, is anchored to its foundation slab with ASTM A 193, Grade B7 anchor bolts. The bolt diameters are 1-1/4 inches for the condensate water storage tanks and the transfer tank, and 1-3/8 inches for the refueling water storage tanks. The wall of the fire water tank is welded to an insert plate in the foundation.

The tank foundation slabs are resting on concrete fill anchored to bedrock with rock anchors. The reinforced concrete protective walls of the steel tanks are anchored to bedrock as shown in Figure 3.8-65, Sheet 2.

3.8.4.1.2 Applicable Codes, Standards, and Specifications

The foundation slabs for the Design Class I outdoor storage tanks listed are designed and constructed in accordance with the ACI 318-63.

3.8.4.1.3 Loads and Loading Combinations

The foundation slabs for the Design Class I outdoor storage tanks are designed for dead loads, hydrostatic load, and seismic load:

$$C = DL + HS + EQ + R \qquad (3.8\text{-}30)$$

where:

C = total load on foundation
DL = dead load of tank
HS = hydrostatic load

EQ	=	maximum seismic loads including inertial, impulsive, and convective loads
R	=	pipe reaction load including dead, thermal, and seismic loads

3.8.4.1.4 Design and Analysis Procedures

The foundation slabs with concrete fill are designed to prevent overturning and sliding of the tank and limit bearing pressure to 80 ksf. The rock anchors attaching the tank to the foundation are designed so that the maximum uplift force is within the allowable capacity provided in Figure 3.8-65, Sheet 2.

3.8.4.1.5 Structural Acceptance Criteria

Stresses in the reinforced concrete foundation slabs are limited to the allowable values in ACI 318-63.

3.8.4.1.6 Materials, Quality Control, and Special Construction Techniques

The quality control measures discussed in Sections 3.8.1.6.1 and 3.8.1.6.2 for the containment structure also apply to the Design Class I tank foundations.

Material strengths for the Design Class I tank foundation slabs are as follows:

(1) Concrete strength of foundation slab and concrete fill is 3000 psi.

(2) Reinforcing steel in foundation slab is ASTM A 615, Grade 40.

(3) Rock anchors conform to VSL 28, strand #ER5-28, with double corrosion protection and are fully grouted with concrete strength of 4000 psi.

3.8.4.2 Concrete and Structural Steel Supports

Concrete and structural steel supports for RCS components are described and evaluated in Section 5.5.14. Loading combinations for these supports are discussed in Section 5.2.

3.8.5 DESIGN CLASS II STRUCTURES CONTAINING DESIGN CLASS I EQUIPMENT

The turbine building and the intake structure are Design Class II structures that contain Design Class I equipment. The turbine building contains the component cooling heat exchangers, emergency diesel generators, 4.16-kV vital switchgear, control room pressurization system, and other Class I systems. The intake structure contains the auxiliary saltwater (ASW) pumps and associated equipment.

To ensure that the Design Class I equipment would not be affected by failure of the Design Class II structures, both the turbine building and the intake structure are evaluated for the HE, using responses from the dynamic analyses discussed in Section 3.7.

The capability of the intake structure to protect the ASW system during design flood events is evaluated to ensure this capability, as described in Sections 2.4 and 3.4.

The OTSC is located in the turbine building buttresses and is designed to meet seismic loading criteria.

3.8.5.1 Turbine Building

3.8.5.1.1 Description

The turbine building was originally designed as a Design Class II structure using static equivalent seismic loads. Subsequently, the building was dynamically analyzed and designed to assure that it would not collapse and impair the function of Class I equipment during a DDE. Later, during the Hosgri evaluation, the building was reevaluated again and upgraded to withstand the Hosgri seismic loads. As a result of the Hosgri evaluation, buttresses and concrete walls were added to the turbine building, and internal modifications, such as reinforcing main columns, strengthening floor diaphragms, and roof and wall bracing, were made.

The turbine pedestal was originally designed as a Design Class II structure, using static equivalent seismic loads. During the Hosgri evaluation, the pedestal was reevaluated and upgraded to withstand the Hosgri seismic loads. As a result of the Hosgri evaluation, six piers were posttensioned and the pedestal-to-building separations were increased along the east and west sides of the pedestal.

The turbine building is located adjacent to the west side of the auxiliary building as shown in Figure 1.2-2, Plant Layout. The general layout of equipment in the turbine building, including the turbine generators, is shown in Figures 1.2-13 through 1.2-20, Figures 1.2-24 through 1.2-27, and Figures 1.2-30 through 1.2-32. Generally, the Unit 1 and Unit 2 portions of the turbine building are opposite hand and similar to the other, with each portion of the structure containing equipment for one unit. Exceptions are the presence of a machine shop and material storage area common to both units in the Unit 1 portion, and the OTSC in the Unit 2 portion of the structure.

Main floor levels in the turbine building are at elevations 85, 104, 119, and 140 feet. The foundation of the building is at elevation 85 feet. Representative plans at the main floor levels roof truss lower chord level, and a typical section are shown in Figures 3.8-66 through 3.8-71.

The turbine building is a reinforced concrete shear wall structure except for the superstructure, which is a structural steel moment resisting and braced frame structure extending from elevation 140 feet to elevation 217 feet. Shear walls generally range from 16 to 29 inches thick. Floors are 10- to 12-inch-thick reinforced concrete slabs or 1/2-inch-thick steel plate, supported on steel framing and steel columns. The reinforced concrete foundation mat is generally 3 feet thick except under the turbine pedestal, where the thickness is 10 feet. Reinforced concrete turbine pedestals, one for each unit, are located in the building; six piers of each pedestal are posttensioned. The pedestals are structurally isolated from the building floors and extend from the common foundation slab, elevation 85 feet, to elevation 140 feet. Two 135-ton overhead cranes are located in the building.

3.8.5.1.2 Applicable Codes, Standards, and Specifications

The following codes and standards are used in the HE evaluation, and in the design, construction, inspection, and testing of HE modifications to the turbine building:

(1) ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-71) except that, for the HE evaluation and design, the 1973 Supplement to ACI 318 is used and design load combinations are as described in Section 3.8.5.1.3

(2) Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-74)

(3) Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-65)

(4) Recommended Lateral Force Requirements, 1974 Seismology Committee Structural Engineers Association of California (SEAOC)

(5) Recommended Practices for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction, American Welding Society (AWS D12.1-75)

(6) AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, February 12, 1969, is generally used for steel structures. AISC Specification, dated November 1, 1978, is also used for evaluating selected connections.

(7) Code for Welding in Building Construction (AWS D1.1 Rev. 2-77). For inspection of non-ASME structural welds or new non-ASME work performed after January 1, 1988, the guidelines of Nuclear Construction Issues Group (Visual Weld Acceptance Criteria, Vol. 1-3, EPRI Report No. NP-5380, September 1987) may be used except for those cases where:

    (a) Fatigue is a governing design condition

(b) The weld allowables are permitted to be higher than those allowed by AWS D1.1 (such as full penetration weld evaluation for the HE)

(c) The weld is part of work performed in the ASME Section XI Inservice Inspection Program

(8) Materials and the quality control tests for materials conform to ASTM standards

3.8.5.1.3 Loads and Loading Combinations

3.8.5.1.3.1 Design Loads

The following loads are considered in the HE evaluation of the turbine building.

3.8.5.1.3.1.1 Dead Loads

Dead loads consist of the weight of the structure, permanent attachments and permanent equipment.

3.8.5.1.3.1.2 Live Loads

Live loads consist of any actual live loads acting on the element considered.

3.8.5.1.3.1.3 Seismic Loads

Seismic loads are based on a response spectrum modal superposition analysis. This analysis is described in Section 3.7.2.

3.8.5.1.3.2 Loading Combination

Concrete Structural Elements

$$U = D + L + HE \qquad (3.8\text{-}31)$$

where:

U	=	Strength determined in accordance with the methods described in ACI 318-71 and 1973 Supplement, except strength of shear walls is based on method described in Section 3(c) of the 1974 SEAOC. (See also Section 3.8.5.1.5.)
D	=	dead load
L	=	live load
HE	=	loads resulting from an HE

Steel Structural Elements

Where plastic design methods are used:

$$Y = D + L + HE \qquad (3.8\text{-}32a)$$

Where elastic working stress design methods are used:

$$1.7S = D + L + HE \qquad (3.8\text{-}32b)$$

where:

S = required section strength based on elastic design methods and the allowable stresses defined in Part 1 of the AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, February 12, 1969

Y = required section strength based on plastic design methods described in Part 2 of AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, February 12, 1969 (See also Section 3.8.5.1.5.)

3.8.5.1.4 Design and Analysis Procedures

Structural analysis of the turbine building is based on the traditional methods of engineering analysis for structural steel and reinforced concrete structures. These methods are based on the principles of equilibrium, compatibility of deformations, and predictions of material strength by the methods of the AISC Code and the ACI Code. The use of these codes is discussed in Section 3.8.5.1.5.

The lateral force resisting system of the turbine building above elevation 140 feet consists of moment resisting bents, formed by steel roof trusses and steel plate columns, steel cross-brace frames at exterior walls, and a steel bracing system in the plane of the roof truss lower chords. At and below elevation 140 feet, lateral resistance is provided by concrete and steel plate floors acting as diaphragms; by concrete shear walls; by concrete buttresses along the east and west sides of the building; and by steel cross-braced frames above elevation 104 feet at the north and south ends of the building. Vertical forces are transmitted to the foundation by the steel plate columns, concrete walls, and interior steel columns which support the steel floor framing system.

HE forces on the lateral force resisting system of the turbine building and the turbine pedestal are based on the response of the turbine building and pedestal seismic models described in Section 3.7. In some cases detailed analytical models are developed to calculate building member forces. Structural evaluation of members is performed for the load combinations described in Section 3.8.5.1.3. The evaluation considers bridge crane location and lifted load as described in Section 3.7. Members are evaluated against the acceptance criteria in Section 3.8.5.1.5. Results of the evaluation are shown in Tables 3.8-26 and 3.8-27.

Computer programs used in the structural analysis and the verification measures are listed in Table 3.8-6.

3.8.5.1.5 Structural Acceptance Criteria

For HE load combinations actual material properties are used. Lateral force resisting elements are allowed inelastic deformation subject to the ductility limits shown in Table 3.8-24. The strength of concrete elements is determined in accordance with the methods of ACI 318-71 and the 1973 Supplement. Strength of concrete shear walls is determined in accordance with Section 3(c) of 1974 SEAOC. Strength of steel elements is determined in accordance with the AISC Code, February 12, 1969.

Calculated forces and capacities for selected structural elements of the turbine building and turbine pedestals are compared in Tables 3.8-26 and 3.8-27. Generally, the predicted forces in combination with earthquake effect do not exceed member strengths. A limited number of members are found to exhibit inelastic behavior which does not exceed the allowable ductility limits of Table 3.8-24. Therefore, all structural elements satisfy the criteria.

Separations between the turbine building's primary structure and the turbine pedestal are adequate to ensure these structures will not impact each other when subject to the HE load combination. The relative displacements between these structures are summarized in Table 3.8-27A.

3.8.5.1.6 Materials, Quality Control, and Special Construction Techniques

The turbine building, including the turbine pedestals, was originally constructed prior to 1978. Following the HE evaluation of the plant, modifications to the turbine building and the turbine pedestals were made during the period 1978 to 1979. Materials installed during these two periods are described in the following paragraphs.

3.8.5.1.6.1 Concrete

Design strengths of concrete are as follows:

	Age, days	Compressive Strength, psi
Original construction:		
East-west walls	28	5000
Elevation 140 feet floor	60	5000
All other concrete	28	3000

Hosgri Modifications:		
Concrete above elevation 85 feet	28	5000
All other concrete	28	3000
Modifications for sixth diesel generator addition, all associated concrete	28	4000

Average strengths are used with the HE load combination. The average strengths are as follows:

	Age, days (except as noted)	Compressive Strength, psi
Original Construction:		
Turbine pedestal	6 years[a]	6000
East-west walls	28	5500
Elevation 140 feet floor	60	6590
All other concrete	28	3870
Hosgri Modifications:		
Concrete above elevation 85 feet	28	5680
All other concrete	28	4260

3.8.5.1.6.2 Reinforcing Steel

Reinforcing steel is ASTM A615, Grade 40, except in some locations where grade 60 is used. Average test values are used with HE load combinations. Properties of the reinforcing steel are as follows:

	Grade 40	Grade 60
Design yield strength, psi	40,000	60,000
Original construction		
Average yield strength, psi	51,400	65,900
Average tensile strength, psi	80,600	101,400
Hosgri Modifications		
Average yield strength, psi	51,900	67,000
Average tensile strength, psi	81,300	106,500

[a] Turbine pedestal concrete strength is based on cylinder tests of 6-year-old stored specimens. The strength is verified by rebound hammer tests and by tests of concrete core samples.

3.8.5.1.6.3 Structural Steel

Structural steel is ASTM A36 except for reinforcing bars installed at flanges of some columns along column lines A and G where ASTM A572 Grade 50, is used. Properties of the structural steel are as follows:

	ASTM A36	ASTM A572, Gr 50
Design yield strength, psi	36,000	50,000
Original construction		
Average yield strength, psi	44,000	
Average tensile strength, psi	68,000	
Hosgri Modifications		
Elevation 140 and 119 feet floor plate		
Average yield strength, psi	40,800	
Average tensile strength, psi	69,300	
Other structural steel		
Average yield strength, psi	44,500	55,200
Average tensile strength, psi	68,200	87,400

3.8.5.2 Intake Structure

3.8.5.2.1 Description of Intake Structure

The seismic Design Class II intake structure is a reinforced concrete building constructed with 3,000 psi minimum-specified-strength concrete. The structure has plan dimensions of approximately 240 x 100 feet. The long dimension corresponds to the north-south direction, and is parallel to the seaward face of the structure. The intake structure is backfilled by rock on three sides, and has water on the fourth (western) side. The top deck of the structure has a maximum elevation of +17.5 feet. A concrete ventilation tower with steel coaxial ventilation pipe extends to an elevation of +49.4 feet. The structure is supported by a concrete mat foundation at elevation -31.5 feet. Figures 3.8-72 through 3.8-74 illustrate plans at elevations +17.5, -2.1, and -31.5 feet; Figures 3.8-75 through 3.8-77 illustrate representative sections through the structure.

The top level of the structure consists of an 18-inch-thick concrete slab, except for the roadway area where it is 24 inches thick. Openings, as shown in Figure 3.8-72, are provided to allow removal of pumps, screens, and gates. The pump deck floor at elevation -2.1 feet supports the four main circulating water pumps and the four seismic Design Class I ASW pumps. Design Class I ASW equipment is located in ventilated watertight compartments. The structure is symmetric about a vertical plane in the east-west direction through its centerline.

3.8.5.2.2 Applicable Codes, Standards, and Specifications

The following codes and standards were used in the Hosgri evaluation and the design, construction, inspection, and testing of the intake structure.

(1) ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-63, ACI 318-71, ACI 318-77)

(2) Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-65)

(3) Recommended Practices for Welding Reinforcing Steel, Metal Inserts and Connections in Reinforced Concrete Construction, American Welding Society AWS D12.1-75

(4) AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, February 12, 1969, and November 1, 1978

(5) Code for Welding in Building Construction, AWS D1.1-77, Rev. 2. For inspection of non-ASME structural welds or new non-ASME work performed after January 1, 1988, the guidelines of Nuclear Construction Issues Group (Visual Weld Acceptance Criteria, Vol. 1-3, EPRI Report No. NP-5380, September 1987) may be used except for those cases where:

    (a) Fatigue is a governing design condition

    (b) The weld allowables are permitted to be higher that those allowed by AWS D1.1 (such as the full penetration welds evaluation for the HE)

    (c) The weld is part of work performed in the ASME Section XI Inservice Inspection Program

(6) Recommended Lateral Force Requirements 1974 Seismology Committee, Structural Engineers Association of California (SEAOC)

(7) Materials and quality control tests for materials conform to ASTM standards

3.8.5.2.3 Loads and Loading Combination

Seismic Load Combination

$$U = D + L + HE \qquad (3.8\text{-}33)$$

where:

D	=	dead load of the structure and equipment loads
L	=	live load
HE	=	loads resulting from HE
U	=	strength required to resist design loads as described in Section 3.8.5.2.5

Wave Load Combination

$$U = D + L + W_f \qquad (3.8\text{-}34)$$

where:

U, D, and L are as defined in (1) above
W_f = wave force associated with breakwater degraded to MLLW

3.8.5.2.4 Design and Analysis Procedures

3.8.5.2.4.1 General

Structural analysis of the intake structure is performed by the traditional methods of engineering analysis for structural steel and reinforced concrete structures. These methods are based on the principles of equilibrium, compatibility of deformations, and predictions of material strength by the methods of the AISC Code, and the ACI Code. The use of these codes is discussed in Section 3.8.5.2.5.

3.8.5.2.4.2 Seismic Forces

A time-history dynamic analysis is performed with a computer program to determine the structure response spectra. A response spectrum dynamic modal superposition analysis is performed to determine structure response maxima. The analytical procedure using modal superposition methods is described in Section 3.7.2.

The demands resulting from the combination of north-south, east-west, and vertical components of the HE, in conjunction with dead loads, actual live loads, and soil pressures, are less than the yield capacity of the major portion of the structure, including the area housing the Class I ASW. The only exceptions are some of the flow straighteners (or piers) that exhibit stresses beyond code values. However, these piers demonstrate ductility properties that would preclude structural failure. The ductilities are within allowables as stated in Section 3.8.5.2.5. Table 3.8-28 presents the results of the analysis.

The intake structure is reviewed to verify that there is an adequate factor of safety against sliding and overturning, and the foundation pressure is within the allowable value of 50 ksf as described in Reference 13.

3.8.5.2.4.3 Wave Forces

As shown in Figures 3.8-78 and 3.8-79, a scaled, three-dimensional physical model of the cooling water intake basin, the intake structure, and a hypothetically damaged breakwater was constructed to examine the wave effects on the intake structure as described in References 8 and 9.

Based on the test results, the intake structure was modified to mitigate wave slam (high magnitude, high frequency) pressure behind the curtain wall and to withstand the measured pressures on the bottom of the slab.

The ASW pump compartments were modified to provide the required ventilation on the Design Class I equipment and to prevent flooding due to combined tsunami and storm wave runup conditions as described in Chapter 2. As discussed in Section 3.3.2.3.2.10, the ASW pump compartment modifications were reviewed for a tornado with missiles.

A risk analysis, as described in Reference 14, was performed to determine the frequency of vessel impact with the intake structure which houses the ASW pumps. In the analysis, the breakwater was assumed to be degraded to the MLLW level. The analysis considered only large vessels (greater than 250 tons displacement), since impact on the intake structure by smaller vessels was concluded, on the basis of a deterministic analysis, to be inconsequential to the safety-related function of the ASW pumps.

The results of risk analysis for frequency of impact indicate a frequency of 6.7×10^{-6} breakwater boundary crossings per year for storm-independent analysis and 1.9×10^{-5} breakwater boundary crossings per year for storm-dependent analysis. The probability of large vessels, therefore, crossing the degraded breakwater and impacting the intake structure is quite low.

3.8.5.2.5 Structural Acceptance Criteria

For the load combinations given in Section 3.8.5.2.3, the intake structure is designed so that it does not sustain damage that would adversely affect the function of the Class I ASW system and prevent it receiving an adequate supply of water.

For load combinations with seismic force, strength is based on SEAOC 1974 concrete shear walls; ACI 318-63 and ACI 318-71 for other concrete members; and AISC, Seventh Edition, Part II for steel members. Lateral force resisting elements are allowed inelastic deformation consistent with ductility factors indicated in Table 3.8-24. For these elements, the allowable stress limitations given in the codes above need not apply.

For load combinations with wave forces, strength is based on ACI 318-71 and AISC, Seventh Edition, Part II for all structural members except for ASW pump compartment modifications. For the latter, strength is based on ACI 318-77 and 1.6 times AISC Eighth Edition, Part I allowable values.

3.8.5.2.6 Materials, Quality Control, and Special Construction Techniques

The intake structure was originally constructed prior to 1981. As a result of January 1981 storm damage to the west-breakwater, hydraulic model studies of wave effects on the intake structure were conducted in 1982 (References 8 through 12), and the intake structure was modified to withstand these wave effects.

The material strengths for these periods are provided below:

Concrete

Prior to 1981	Minimum specified fc'	=	3000 psi @ 28 days
	Average test values fc'	=	3630 psi
1981 modifications	Minimum specified fc'	=	5000 psi @ 28 days

Reinforcing Steel

Prior to 1981	ASTM A615, Grade 40;	Minimum specified fy	= 40 ksi
		Average test values fy	= 49.6 ksi
1981 Modifications	ASTM A615, Grade 60;	Minimum specified fy	= 60 ksi

Structural Steel

Prior to and after1981	ASTM A36;	Minimum specified fy	= 36 ksi

3.8.6 PIPEWAY STRUCTURES

3.8.6.1 Description of Pipeway Structures

The pipeway structure for each unit is a steel frame structure attached to the outside of the containment shell, the auxiliary building, and the turbine building as shown in Figure 3.8-80. The pipeway structure in one unit is essentially a mirror image of the other. The primary function of the pipeway structure is to support main steam and feedwater piping. The pipeway structure has five major platforms located at elevations 109, 114, 118, 127, and 138 feet. Connections between the pipeway structure and the auxiliary and turbine buildings are provided with slotted holes oriented such that horizontal motions cannot be transmitted between the structures.

3.8.6.2 Applicable Codes, Standards, and Specifications

3.8.6.2.1 Codes

The following codes and standards are used, insofar as they are applicable, in the design and/or construction of the pipeway structure.

(1) AISC Specification for Design, Fabrication, and Erection of Structural Steel for Buildings, February 12, 1969

(2) ACI Standard Building Code Requirements for Reinforced Concrete (ACI-318-63)

(3) Standard code for welding in building construction (AWS D1.0-69). For inspection of non-ASME structural welds or new non-ASME work performed after January 1, 1988, the guidelines of Nuclear Construction Issues Group (Visual Weld Acceptance Criteria, Vol. 1-3, EPRI Report No. NP-5380, September 1987) may be used except for those cases where:

   a) Fatigue is a governing design condition.

   b) The weld allowables are permitted to be higher than those allowed by AWS D1.1 (such as the full penetration weld evaluation for the HE).

   c) The weld is part of work performed in the ASME Section XI Inservice Inspection Program.

(4) Materials, and the quality control tests for materials, conform primarily to ASTM and ASME standards. Additional materials and supplemental quality assurance requirements conform to ANSI standards

3.8.6.3 Loads and Loading Combinations

3.8.6.3.1 Design Loads

The following loads are considered in the design of the pipeway structure.

3.8.6.3.1.1 Dead Loads

Dead loads consist of the weight of the structure, piping, pipe rupture restraints, electrical raceways, and equipment.

3.8.6.3.1.2 Live Loads

Live loads are temporary loads that may be placed on the structure. These are considered small in relative magnitude and, therefore, are considered negligible.

3.8.6.3.1.3 Earthquake Loads

Earthquake loads are as described in Section 3.7.2.1.7.1.

3.8.6.3.1.4 Wind Loads

Wind loads are determined in accordance with the criteria presented in Section 3.3. However, the forces due to wind are much less than those due to earthquake; consequently, seismic loads, rather than wind, are entered into the load combination equations.

3.8.6.3.1.5 Thermal Loads

Thermal loads are those induced by the main steam and feedwater pipes through the support system. These loads are considered negligible.

3.8.6.3.2 Loading Combinations

The following loading combinations are used in the design of the pipeway structure.

3.8.6.3.2.1 Normal Conditions

Dead loads and design earthquake (DE) are considered as follows:

$$S = D + DE \qquad (3.8\text{-}35)$$

where:

S	=	required capacity of structural members based on the method described in Section 3.8.6.5.1
D	=	dead load
DE	=	loads resulting from the DE

3.8.6.3.2.2 Abnormal Conditions

Where elastic working stress design methods are used:

$$1.6S = D + DDE \qquad (3.8\text{-}36)$$

$$1.7S = D + DDE + Yr \qquad (3.8\text{-}37)$$

$$1.7S = D + 1.25DE + Yr \quad (3.8\text{-}38)$$

$$1.7S = D + HE \quad (3.8\text{-}39)$$

Where plastic design methods are used:

$$1.0Y = D + DDE + Yr \quad (3.8\text{-}40)$$

$$1.0Y = D + 1.25DE + Yr \quad (3.8\text{-}41)$$

where:

DDE	=	loads resulting from the DDE
HE	=	loads resulting from the HE
Yr	=	reaction on structure from a broken pipe, generated by a postulated pipe break, including an appropriate dynamic load factor (DLF)
Y	=	required section strength based on plastic design methods described in Part 2 of the AISC specification referenced in Section 3.8.6.2.1

3.8.6.4 Design and Analysis Procedures

3.8.6.4.1 Hosgri Event

Seismic forces from the response spectrum dynamic analysis described in Section 3.7.2.1.7.1 are used in the stress evaluation of the Unit 1 pipeway structure. The Unit 2 pipeway structure is evaluated using a detailed three-dimensional model and the static equivalent method of seismic analysis described in Section 3.7.2.1.7.1. The calculated stress ratios for the most critical members are given in Table 3.8-5A.

3.8.6.4.2 Design Earthquake and Double Design Earthquake

Member forces are calculated using corresponding Hosgri forces adjusted in proportion to ratios of DE or DDE to HE spectral accelerations. These forces are used in the stress evaluation of the Unit 1 and 2 pipeway structures. Stresses obtained by this method are confirmed by time-history dynamic analysis described in Section 3.7.2.1.7.1. Calculated stress ratios for the most critical members are given in Table 3.8-5A.

3.8.6.4.3 Computer Programs

Computer programs used in the structural analysis and the verification measures are listed in Table 3.8-6.

3.8.6.5 Structural Acceptance Criteria

For DE and DDE load combinations in the absence of pipe break loads (Yr), the minimum specified yield strength for structural steel is considered. For load combinations including HE or Yr, the actual material properties are used. In addition, the following conditions apply.

3.8.6.5.1 Normal Conditions

For normal conditions, the pipeway structure is designed to the allowable working stresses in Part 1 of the AISC Code, February 12, 1969; however, the increase in allowable stress usually allowed for load combinations involving earthquake forces is not used.

3.8.6.5.2 Abnormal Conditions

For abnormal conditions, the pipeway structure, in general, is designed for overall elastic behavior. For load combinations (3.8-37), (3.8-38), (3.8-40), and (3.8-41) of Section 3.8.6.3.2.2, the acceptance criteria described therein should be satisfied first without considering the effect of Yr. When considering the effect of Yr, local section strength capacities may be exceeded provided there is no loss of function of any safety-related system.

3.8.6.5.3 Factors of Safety

The calculated capacity-to-demand ratio for the most critical members of the pipeway structure are given in Table 3.8-5A. In all cases these ratios are greater than the minimum allowable value of 1.0. Therefore, all structural elements satisfy the criteria.

3.8.6.6 Materials, Quality Control, and Construction Techniques

Structural steel is ASTM A441 and ASTM A516 Grade 70. ASTM minimum specified values are used with DE and DDE load combinations in the absence of Yr. Average test values are used with load combinations that include HE or Yr. Minimum and average values are as follows:

	ASTM A441	ASTM A516
Minimum yield strength, psi	45,000	38,000
Average yield strength, psi	51,600	51,040

High strength bolts, nuts, and washers used for connections are predominantly ASTM A490. Some ASTM A325 bolts are also used when found acceptable. Impact tests for structural steel and high strength bolts were performed in accordance with ASTM Standard Method A370 at 0°F.

Welding electrodes conform to ASTM A233, E70 series low hydrogen.

Where indicated on drawings, nondestructive testing was performed as required utilizing ultrasonic or magnetic particle techniques.

Approved substitute for ASTM A441 structural steel is ASTM A572 Grade 42. For any new construction after May 2004, the structural steel used may be A572 Grade 42. The impact test is required for this new steel.

3.8.7 SAFETY-RELATED MASONRY WALLS

In accordance with Reference 28, safety-related masonry walls (see Section 3.8.7.1) have been reevaluated and modified as necessary using conservative design and analysis procedures, and structural acceptance criteria as specified in Section 3.8.7.5. Design and analysis methods and structural acceptance criteria are in accordance with Reference 29. NRC Staff acceptance of the wall reevaluation design and analysis methods is documented in Reference 30.

3.8.7.1 Description of Safety-Related Masonry Walls

Safety-related masonry walls are those walls which support safety-related piping or equipment, or whose failure could prevent a safety-related system from performing its intended safety function. Safety-related walls are located in the auxiliary and turbine buildings at locations identified in Figures 3.8-83, -84, and -85, and are evaluated in accordance with Reference 29. These walls are fire walls or nonbearing partitions serving various functions and are not required to resist tornado or missile loads and are not part of the buildings lateral force resisting system. Some of the walls support small piping, conduits, or instrumentation tubing. A few walls support concrete or metal deck ceilings.

All walls are single width, 8 or 12 inches thick, fully grouted, and reinforced in the horizontal and vertical directions with steel reinforcing bars. The bottoms of all walls are tied to the building structure.

In general, walls extend to the underside of the floor structure, where lateral support is provided by a structural supporting system. A separation joint filled with compressible material is provided at the top and side boundaries of all walls where they abut the building structure floors or columns. Walls are braced with steel members. The bottoms of some walls are connected to the floor with bolted steel angles. Some walls are strengthened by steel plates bolted to each face of the wall.

3.8.7.2 Applicable Codes, Standards, and Specifications

The following codes and standards are used in the design, construction, inspection, and testing of safety-related masonry walls:

(1) Building Code Requirements for Concrete Masonry Structures (ACI 531-79) and Commentary (ACI 531R-79)

(2) AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, February 12, 1969

(3) Materials and quality control tests for materials conform to the applicable ASTM standards

3.8.7.3 Loads and Loading Combinations

3.8.7.3.1 Design Loads

The following loads are considered in the evaluation of safety-related masonry walls:

3.8.7.3.1.1 Dead Loads

Dead loads consist of the weight of the wall and supported items.

3.8.7.3.1.2 Live Loads

Live loads consist of occupancy loads, if any, acting on the wall.

3.8.7.3.1.3 Earthquake Loads

Earthquake loads are those resulting from the HE. The loads are based on the response spectrum, single-degree-of-freedom method described in Section 3.7.2.1. The percentage of critical damping used is 7 percent.

3.8.7.3.1.4 Thermal Loads

Thermal loads are loads induced by local increases in temperature resulting from normal operating and from postulated accident conditions.

3.8.7.3.1.5 Pipe Reaction Loads

Pipe reactions result from hydraulic forces, thermal expansion, and seismic events. These loads are transferred to the structure through pipe supports and result from normal operating conditions and postulated accident conditions.

3.8.7.3.1.6 Pressure Loads

Pressure loads are forces generated by a postulated pipe break. Pressures from a postulated broken pipe are calculated as described in Section 3.6.

3.8.7.3.1.7 Loads and Loading Combinations

The following load combinations are used in evaluation of safety-related masonry walls:

U = D + L + To + Ro + HE

U = D + L + Ta + Ra + 1.5Pa

U = D + L + Ta + Ra + 1.0Pa + HE

where:

U	=	strength determined using acceptance criteria described in Section 3.8.7.5
D	=	dead load
L	=	live load
To	=	thermal loads during normal operating conditions
Ro	=	pipe reactions during normal operating conditions
HE	=	loads resulting from an HE
Ta	=	thermal loads generated by a postulated pipe break, including To
Pa	=	pressure load generated by a postulated pipe break
Ra	=	pipe reactions from unbroken pipes generated by postulated pipe break conditions, including Ro

3.8.7.4 Design and Analysis Procedures

Evaluation of safety-related masonry walls is performed using traditional methods of engineering analysis. Proper consideration is given to boundary conditions, cracking of sections, and the dynamic behavior of the walls. Both in-plane and out-of-plane loads and interstory drift effects are considered.

HE forces on the walls are based on applicable building floor response spectra generated by the analysis described in Section 3.7.2.1. In some cases, detailed models of the walls and supporting steel members are used to calculate forces and stresses in the walls.

HE forces on the walls including wall reactions are calculated by linear elastic analysis. Applied forces on the walls include the combined effects of vertical loads and horizontal out-of-plane wall deflections (P-Δ effect). Masonry wall stiffnesses are based on best estimate (median) properties, which are:

E_m = 750 f'm
F'_m = 1950 psi
f_r = $4(f'_m)^{0.5}$

The stiffness and strength of walls strengthened with steel plates at each face are based on wall panel tests. Drypack grout at the top of the walls is treated as unreinforced and is not relied upon to withstand earthquake loads. An additional evaluation of the walls is performed to address the variability of material properties, workmanship, and construction tolerances.

Computer programs used in the structural analysis and the verification measures are listed in Table 3.8-6.

3.8.7.5 Structural Acceptance Criteria

In the evaluation of safety related masonry walls for load combinations, including HE or pipe break loads, actual material properties may be used.

The moment capacity of a masonry wall is not less than the moment produced by the applied loads. Moment capacity of the masonry wall is determined using the strength design method, with a strength reduction factor, ϕ, equal to 1.0. Allowable masonry shear stress is 1.3 x 1.1 $(f'_m)^{0.5}$, where f'_m is as defined in Section 3.8.7.4. Allowable masonry bearing stress is 2.5 x 0.25 (f'_m). Analysis of the behavior of masonry walls strengthened with steel plates is substantiated by tests. Allowable forces on steel members are based on 1.6 times allowable stresses defined in the AISC Code, Part 1, or 0.9 times member strengths defined in the AISC Code, Part 2.

3.8.7.6 Materials, Quality Control, and Special Construction Techniques

3.8.7.6.1 Masonry Units

Masonry units are hollow, load-bearing, open-ended lightweight units of ASTM Designation C90, Grade A. The average compressive test strength of masonry units is 3400 psi on the net area.

3.8.7.6.2 Reinforcing Steel

In general reinforcing steel is ASTM A615, Grade 40. The average yield strength by test is 51,400 psi.

In limited areas, reinforcing steel is ASTM A615, Grade 60. The average yield strength by test is 64,200 psi.

3.8.7.6.3 Core Fill

Grout having a minimum specified compressive strength of 2000 psi is placed in all masonry unit cells. The average tested compressive strength is 3285 psi.

3.8.7.6.4 Mortar

Mortar is ASTM Designation C270, Type S.

3.8.7.6.5 Construction Inspection

Inspection procedures meet the intent of Section 4.5 of Building Code Requirements for Concrete Masonry Structures (ACI 531-79).

3.8.8 SPENT FUEL STORAGE RACKS

3.8.8.1 Description of the Spent Fuel Pool and Racks

The description of the SFP is provided in Sections 3.8.2.1 and 9.1.2.2. Each fuel pool has 16 high density fuel rack modules as shown in Figure 9.1-2. They are free-standing and consist of individual cells with an 8.85 by 8.85-inch square cross-section, each of which stores a single Westinghouse PWR fuel assembly. The number of cells varies from 34 to 110 per module. The cells are fabricated by welding two formed stainless steel channels, which are welded together by stainless steel gap channels to provide the required predetermined distance between the cells. Typically, each module is provided with four support legs, three of which are adjustable and one fixed. The adjustable support legs are used to achieve a leveled free-standing position on the pool floor. For ease of installation and to reduce potential interferences with liner seam welds, each rack support leg is supported on a bridge plate. Typically, each rack module is equipped with girdle bars located near the top, which are designed to accommodate seismically induced impact loads.

3.8.8.2 Applicable Codes, Standards, and Specifications

The following codes and standards are used in the design and construction of the racks.

(1) AISC Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, 1969 Edition

(2) ASME Boiler and Pressure Vessel Code, Section III and Subsection NF, 1983 Edition

(3) AEC "Spent Fuel Storage Facility Design Basis," SG 13, March 1971

(4) Westinghouse Fuel Assembly, Storage and Refueling Equipment Design Interface Specification No. F-8, Rev. 8

(5) NRC "OT Position for Review and Acceptance of Spent Fuel Storage and Handling Applications," April 14, 1978, and "Modifications to the OT Position," January 18, 1979, letters from B. K. Grimes to All Power Reactor Licensees

(6) Appendix D to Standard Review Plan, Section 3.8.4, "Technical Position on Spent Fuel Pool Racks," Revision 0, July 1981, NRC

3.8.8.3 Loads and Loading Combinations

The loads and loading combinations and the corresponding acceptance criteria are as follows (Reference 20):

	Loading Combination	Stress Limit
(a)	D $D + T_o$ $D + T_o + E$	Level A service limits
(b)	$D + T_a + E$ $D + T_o + P_f$	Level B service limits
(c)	$D + T_f + E'$ $D + F_d$	Level D service limits (The functional capability of the fuel racks should be demonstrated)

where:

D	=	Dead weight-induced stresses (including fuel assembly weight)
F_d	=	Force caused by the accidental drop of the heaviest load from the maximum possible height
P_f	=	Upward force on the racks caused by postulated stuck fuel assembly
E	=	DE
E'	=	HE
T_o	=	Differential temperature induced loads (normal or upset condition)
T_a	=	Differential temperature induced loads (abnormal design conditions)

3.8.8.4 Design and Analysis of Racks

3.8.8.4.1 Design Basis Rack Model

The racks are analyzed using a nonlinear dynamic model as shown in Figure 3.8-81. The model simulates the rack as a single stick supported on a rigid base with supports. Impact springs are provided at the girdle bar and the baseplate locations to account for rack-to-rack and/or rack-to-wall impact. The legs are represented by 4 impact springs to account for the impact as well as frictional sliding. The rattling of the fuel assemblies in cells is considered by the use of additional impact springs. The model includes 2 mass points comprising 8 degrees of freedom. Mass 1 is located in the rack module and has 6 degrees of freedom, (i.e., 3-dimensional space with 3 linear translations and 3 rotations). Mass 2 is located at the top of the fuel and moves with 2 translational degrees of freedom. The lower mass point of the fuel assembly is lumped with the rack module mass (Reference 20).

3.8.8.4.2 Design Basis Rack Analysis and Results

Due to the complexity of the nonlinear time-history analysis, the racks, in general, are analyzed using a single rack model (Figure 3.8-81) that uses conservative model parameters. The seismic input motions are provided in the form of three orthogonal time histories at the fuel pool liner location. A minimum value of 0.2 and a maximum value of 0.8 are used for the range of friction coefficients between the rack supports and the pool liner (Reference 23). The effects of fluid are considered in accordance with the method advanced by Fritz (Reference 24). The impact springs are set at values to produce conservative impact forces. Parametric studies have been performed to evaluate the effects of various design variables such as friction coefficients, size of rack, fuel loading (partially loaded, fully loaded, or empty) on rack, spacing between racks (corner rack vs. non-corner rack), fabrication tolerance, etc. The bounding loads are obtained from those parametric analyses, which are then used for the design of rack components.

The racks have been analyzed to store LOPAR, VANTAGE 5, and ZIRLO fuel assemblies. The impact loads between the cell wall and the fuel assemblies are less than those provided by Westinghouse.

3.8.8.4.3 Multi-Rack Confirmatory Model

The design basis analysis includes several conservative assumptions applied to a single rack model to obtain conservative impact loads. To confirm the adequacy of this methodology, additional multi-rack analyses have been performed (References 21 and 22).

Figure 3.8-82 shows the two-dimensional dynamic model used in this analysis. Each rack is represented by four degrees of freedom simulating fuel rattling, translation, and rocking of the racks. The parameters for the model are developed in a manner similar to those used for the design basis model, except more realistic assumptions are made to compute the fluid coupling coefficients and spring constants.

3.8.8.4.4 Multi-Rack Confirmatory Analysis and Results

The analyses are performed using the nonlinear time history method. The governing horizontal (east-west) and vertical ground motions are applied simultaneously to the model. Parametric studies have been performed to evaluate the effects of various friction coefficients, fuel loading on racks, sizes of racks, lateral gaps, and fabrication tolerances. The results of the analysis demonstrate that the use of conservative model parameters in the design basis analysis (Reference 20) yields conservative rack and fuel assembly impact loads.

3.8.8.5 Materials, Quality Control, and Special Construction Techniques

The materials for both Region 1 and Region 2 rack modules are:

Stainless steel sheet and plate	ASTM A-240-304L
Weld filler material	ASME SFA-5-9 Type 308L and 308LSI
Top part of support	ASTM 479-S21800
Bottom part of support	ASTM SA564-630

3.8.8.6 Design and Analysis of Pool Structure

The existing pool structure was evaluated for postulated interactions of the rack modules with the structure as a result of the seismic event. The effect of the change in fuel rack mass on global dynamic response of the pool structure was considered. The change in global mass was determined to be on the order of 1 percent to 2 percent, therefore, the change in dynamic response is insignificant.

The pool walls were evaluated for the out-of-plane effects due to hydrostatic, hydrodynamic, thermal, and seismic loads. They adequately meet the loading combinations and acceptance criteria in Sections 3.8.2.3.2 and 3.8.2.5, respectively. The walls were also checked for additional impact loads that may result from the rack-to-wall impact and they meet the acceptance criteria of Section 3.8.2.5.

The liner was evaluated for the maximum vertical impact load and maximum horizontal sliding load and was determined to be adequate for leak tightness. The concrete slab that supports the liner was evaluated for the floor impact load using the allowable values specified in ACI 349-80 (Reference 25). The liner in-plane loadings that result from the sliding of the rack and thermal effects were evaluated in accordance with the allowable strains and anchor displacements as specified in ASME Section III, Division II, 1983 (Reference 26).

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3.8.10 REFERENCE DRAWINGS

Figures representing controlled engineering drawings are incorporated by reference and are identified in Table 1.6-1. The contents of the drawings are controlled by DCPP procedures.

3.9 MECHANICAL SYSTEMS AND COMPONENTS

This section discusses the design of PG&E Design Class I mechanical systems and components for Diablo Canyon Power Plant (DCPP) Unit 1 and Unit 2.

3.9.1 Design Bases

3.9.1.1 General Design Criterion 1, 1967 – Quality Standards

The PG&E Design Class I mechanical systems and components that are essential to the prevention of accidents which could affect the public health and safety, or mitigation of their consequences, shall be identified and then designed, fabricated, and erected to quality standards that reflect the importance of the safety function to be performed.

3.9.1.2 General Design Criterion 2, 1967 – Performance Standards

The PG&E Design Class I mechanical systems and components are designed to withstand the effects of, or are protected against, natural phenomena such as earthquakes, flooding, tornadoes, winds, and other local site effects.

3.9.1.3 General Design Criterion 4, 1987 – Environmental and Dynamic Effects Design Bases

Consideration of the dynamic effects associated with postulated pipe ruptures of Reactor Coolant Loop (RCL) piping (i.e., hot, cold, and crossover leg) are excluded from the DCPP Unit 1 and Unit 2 design basis with the approval of leak-before-break (LBB) methodology by demonstrating that the probability of fluid system piping rupture is extremely low under conditions consistent with the design basis for the piping systems.

3.9.1.4 General Design Criterion 9, 1967 – Reactor Coolant Pressure Boundary

The DCPP Unit 1 and Unit 2 reactor coolant pressure boundary is designed to have an exceedingly low probability of gross rupture or significant leakage throughout its design lifetime.

3.9.1.5 General Design Criterion 33, 1967 – Reactor Coolant Pressure Boundary Capability

The DCPP Unit 1 and Unit 2 reactor coolant pressure boundary is capable of accommodating without rupture, and with only limited allowance for energy absorption through plastic deformation, the static and dynamic loads imposed on any boundary components as a result of any inadvertent and sudden release of energy to the coolant.

3.9.1.6 General Design Criterion 40, 1967 – Missile Protection

The PG&E Design Class I mechanical systems and components engineered safety feature (ESF) are designed to or are provided with protection from dynamic effects that might result from plant equipment failure.

3.9.1.7 10 CFR 50.55a – Codes and Standards

The PG&E Quality/Code Class I mechanical systems and components of the reactor coolant pressure boundary are designed, fabricated, erected, and tested in accordance with the requirements for Class A components of Section III of the ASME Boiler and Pressure Vessel Code.

3.9.1.8 10 CFR 50.55a(f) – Inservice Testing Requirements

The American Society of Mechanical Engineers (ASME) Code components (i.e. active pumps and valves) are tested to the requirements of 10 CFR 50.55a(f)(4) and a(f)(5) to the extent practical.

3.9.1.9 Safety Guide 20, December 1971 – Vibration Measurements on Reactor Internals

The DCPP Unit 1 and Unit 2 reactor internals were subjected to confirmatory vibration testing and subsequent visual inspection as part of the preoperational tests to provide added confirmation of the capability of the structural elements of the reactor internals to sustain flow-induced vibrations.

3.9.1.10 NUREG-0737 (Item II.D.1), November 1980 - Clarification of TMI Action Plan Requirements

Item II.D.1 – PG&E is required to demonstrate that the DCPP Unit 1 and Unit 2 pressurizer safety valves (SV), power operated relief valves (PORVs), PORV block valves, and all associated discharge piping will function adequately under conditions predicted for design basis transients and accidents.

3.9.2 Mechanical Systems and Components Description

3.9.2.1 Dynamic System Analysis and Testing

A mechanical design description of the internals and core components showing the differences and similarities between the two DCPP units is presented in Section 4.2.2. The dynamic analysis techniques and methods used to determine and confirm the dynamic response of the reactor internals are presented in the following section. Detailed information of the dynamic system analysis and testing is presented in the reports listed in References Section 3.9.4. Chapter 14 describes the plant initial tests and operation.

A description of the analyses used in the design of PG&E Design Class I mechanical equipment, such as pumps and heat exchangers, to withstand the DE, DDE, and HE-seismic loadings is provided in Section 3.7.

3.9.2.1.1 Vibration Operational Test Programs

This section describes the nature of flow-induced vibrations in the reactor coolant loops and the analyses performed to ensure such vibrations are at an acceptable level.

3.9.2.1.1.1 Main Piping System, Flow-Induced Vibration

Pressure pulses, from the reactor coolant pump impeller are prevented from resulting in flow-induced vibrations in the main piping systems of the RCL. The reactor coolant pump perturbing frequency is quite high when compared to the piping natural frequency. Frequency separation, therefore, ensures a very small probability of self-excited or sympathetic vibration.

3.9.2.1.1.2 Reactor Internals Flow-Induced Vibration

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

The dynamic behavior of reactor components has been studied using experimental data obtained from operating reactors along with results of model tests and static and dynamic tests. The following procedures have been performed in the study of thermal shield vibration:

(1) *During a test program performed with a 1/7th-scale model, the natural frequencies of the thermal shield in water and the maximum vibration amplitude were measured.*

(2) *Shaker test programs performed on a prototype thermal shield with the actual boundary conditions provided full-scale natural frequencies and mode shapes in air. These modes were established by measuring accelerations at the center, top (support elevation), and bottom of the shield. In Figure 3.9-3, the results obtained are plotted for n = 4 and correspond to a thermal shield with eight supports which are indicated on the same figure. The amplitudes of vibration are fitted with a curve y = A sin 4q.*

(3) *Maximum displacements were measured during the preoperational reactor test and were correlated with the information obtained in the 1/7th-scale model and shaker test.*

(4) *In Figure 3.9-4, the maximum amplitudes of vibration are plotted as measured on a thermal shield with six supports. The experimental points have been least-square fitted with a curve y = A sin 3q.*

In general, the study follows two parallel procedures. Frequencies and spring constants are derived analytically, and these values are confirmed from the results of the tests. Damping coefficients are established experimentally, and forcing functions are estimated from pressure fluctuations measured during operation and in models. In parallel, the responses of important reactor structures were measured during preoperational reactor tests and the frequencies and mode shapes of the structures obtained. Once all the dynamic parameters were obtained as explained above, the forcing functions could be estimated. Internals behavior during reactor operation is measured using mechanical devices and nuclear noise methods.

Some components, such as control rod guide tubes, fuel rods, and incore instrumentation tubes are subjected to cross flow and parallel flow with respect to the axis of the structure. For both cases, cross and parallel, the response is obtained after the forcing function and the damping of the system are determined.

3.9.2.1.1.3 Vibration Monitoring

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Since internals of four loop reactors are designed and manufactured to essentially the same procedures, processes, and similar drawings, the response of these structures is similar. Performance data from the instrumentation of actual reactors, as well as mechanical and flow scale models, are available (References 2, 4, and 5). The pre- and post-operational flow test examinations of the Indian Point II Plant internals, the four loop prototype plant, have been completed indicating that all the components performed as predicted. No evidence or sign of damage or incipient failure was found.

The testing programs consisted of measurements of the stresses, deflections, and responses of selected key points in the internals structures during hot functional and low power physics tests. The main purpose of this testing program was to ensure that no unexpected large amplitudes of vibration existed in the internals structure during operation. The tests were extended to provide data and results on what were assumed to be indicators of overall core support structure performance and to verify particular stress and deflection quantities.

3.9.2.1.1.4 Loose-Parts Monitoring

A loose-parts monitoring system is provided for early detection of possible loose parts in the Reactor Coolant System (RCS), in order to reduce the probability of loose parts causing damage to RCS components. This system is described in Section 4.4.5.

3.9.2.1.2 Dynamic Testing Procedure

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

During startup functional testing, piping, including supports and restraints, of certain systems was observed carefully by experienced startup personnel. At selected points of calculated maximum movement, visual inspection and/or measurements were taken and compared with those calculated to establish that stress limits are not exceeded.

If vibration was noted to cause piping or supports movements beyond those allowed, corrective action in the form of additional or redesigned supports, snubbers, etc. was taken and the system was retested to determine that vibrations have been reduced to an acceptable level. Stress analysis on the system was rerun if deemed necessary by the designer. The systems and transients included in this program are listed below:

(1) Reactor coolant pumps start

(2) Reactor coolant pumps trip

(3) Main steam turbine stop valves trip

(4) Steam dump to condenser valves open

(5) Main steam safety and relief valves lift and blowdown

(6) Pressurizer relief valves lift and blow down (Unit 1 only)

(7) Auxiliary feedwater pump turbine stop valve trip

(8) Charging pumps start and trip

(9) Safety injection pumps start and trip

(10) Residual heat removal (RHR) pumps start and trip

(11) Containment spray pumps start and trip

(12) Accumulators discharge to loops

(13) Pressurizer spray valves open and trip closed

(14) Pressurizer power relief valves open

Locations of observation points for piping movements for preoperational piping vibration tests for the above systems were determined from dynamic analysis performed on the *piping systems, with system stiffness and restraint locations taken into account. Observed deflections were compared with code allowable values.*

3.9.2.1.3 Dynamic System Analysis Methods for Reactor Equipment

To verify structural adequacy of reactor internal components and the reactor core for DCPP Unit 1 and Unit 2, nonlinear LOCA and seismic dynamic analyses are performed. These dynamic analyses are performed for both LOCA branch line break, as well as for seismic excitations of DDE and Hosgri (HE).

For faulted conditions, the response of reactor internals due to DDE or HE and LOCA conditions are additive by the SRSS (Square Root of the Sum of Squares) method. The methods and techniques for these dynamic analyses are described below.

3.9.2.1.3.1 LOCA (Loss-Of-Coolant-Accident) Analysis

Details of the RPV system finite element model which is used in these analyses are described. Results of these analyses consist of the nodal time history displacements and the interface impact loads on the reactor vessel, reactor internals and the core. The time history displacements of all major components such as reactor vessel head, vessel bottom and the vessel/barrel nozzles are also generated for their later use in the component stress analyses.

The RPV system finite element model consists of three concentric structural submodels connected by nonlinear impact elements and linear stiffness matrices. The first sub-model represents the reactor vessel shell and its associated components. The reactor vessel is restrained by four reactor vessel supports (situated beneath alternate nozzles) and the attached primary coolant piping.

The second sub-model represents the reactor core barrel, thermal shield, lower support plate, tie plates, and the secondary support components for Unit 1, whereas, for Unit 2 the second sub-model is identical to that of unit 1 except that it has core barrel with neutron pads instead of thermal shield.

These sub-models are physically located inside the first, and are connected to them by stiffness matrices at the internals support ledges. The core barrel to the reactor vessel shell impact is represented by nonlinear elements at the core barrel flange, upper support plate flange, core barrel outlet nozzles, and the lower radial restraints. In addition, vertical impact loads on the fuel assembly top and bottom nozzles.

The third and innermost sub-model represents the upper support plate assembly consisting of guide tubes, upper support columns, upper and lower core plates, and fuel. The third sub-model is connected to the first and second by stiffness matrices and nonlinear elements. The fluid-solid interactions in the LOCA analysis are accounted for through the hydraulic forcing functions generated by Multiflex Code (Reference 3).

The WECAN computer code, which is used to determine the response of the reactor vessel and its internals, is a general purpose finite element code. In the finite element approach, the structure is divided into a finite number of discrete members or elements. The inertia and stiffness matrices, as well as the force array, are first calculated for each element in the local coordinates. Employing appropriate transformations, the element global matrices and arrays are assembled into global structural matrices and arrays, and used for dynamic solution of the differential equation of motion for the structure.

The WECAN Code solves equations of motion using the nonlinear modal superposition theory. Initial computer runs such as dead weight analysis and the vibration (modal) analyses are made to set the initial vertical interface gaps and to calculate eigen values and eigenvectors. The modal analysis information is stored on magnetic tapes, and is used in a subsequent computer run which solves equations of motion. The first time step performs the static solution of equations to determine steady state solution under normal operating hydraulic forces. After the initial time step, WECAN calculates the dynamic solution of equations of motion, nodal displacements, and impact forces, which are stored on tape for post-processing.

Reactor internals response to both cold and hot leg pipe ruptures was analyzed. The LOCA hydraulic forcing functions used in the RPV system analyses were obtained for hypothetical breaks considered in the main loop line. However, with the acceptance of DCPP leak-before-break (LBB) by USNRC (Reference 14), the dynamic effects of breaks in the main reactor coolant loop no longer have to be considered in the design basis of analyses. Only the next most limiting breaks in auxiliary lines have to be considered.

Note that the preceding paragraphs describe the RPV and internals system dynamic analyses for which the WECAN computer code was used. Current analyses (such as the dynamic analyses performed in support of the replacement reactor vessel head project) utilize the ANSYS computer code. The methodology used to develop the ANSYS system models is consistent with the methodology used to develop historic WECAN models. The direct time integration method is used in ANSYS to solve the dynamic equations of motion for the system; whereas the nonlinear mode superposition method is used in WECAN to solve the dynamic equations of motion for the system.

The breaks considered for the replacement vessel head project included: (1) the accumulator line; (2) the pressurizer surge line; and (3) the residual heat removal line.

3.9.2.1.3.2 Reactor Internals Components Subjected to Horizontal Excitations

The analysis methodology is summarized below for components that are subject to horizontal excitations during LOCA conditions. The components include the core barrel, guide tubes, and upper support columns.

3.9.2.1.3.2.1 Core Barrel

For the hydraulic analysis of the pressure transients during hot leg blowdown, the maximum pressure drop across the barrel is a uniform radial compressive impulse. The barrel was analyzed for dynamic buckling using the following conservative assumptions:

(1) The effect of the fluid environment is neglected (water stiffening is not considered).

(2) The shell is treated as simply supported.

During cold leg blowdown, the upper barrel is subjected to a nonaxisymmetric expansion radial impulse that changes as the rarefaction wave propagates both around the barrel and down the outer flow annulus between vessel and barrel. The analysis of transverse barrel response to cold leg blowdown was performed as follows:

(1) The upper core barrel was treated as a simply supported cylindrical shell of constant thickness between the upper flange weldment and the lower core barrel weldment. No credit was taken for the supports at the barrel midspan offered by the outlet nozzles. This assumption leads to conservative deflection estimates of the upper core barrel.

(2) The upper core barrel was analyzed as a shell with four variable sections to model the support flange, upper core barrel, reduced girth weld section, and a portion of the lower core barrel.

(3) The barrel with the core and neutron shield panels(Unit 2 only) was analyzed as a beam elastically supported at the lower radial support, and the dynamic response obtained.

3.9.2.1.3.2.2 Guide Tubes

The dynamic loads on rod cluster control guide tubes are more severe for a LOCA caused by hot leg rupture than for an accident by cold leg over the rod cluster control guide tubes. Thus, the analysis was performed only for a hot leg blowdown.

The guide tubes in closest proximity to the ruptured outlet nozzle are the most severely loaded. The transverse guide tube forces during the hot leg blowdown decrease with increasing distance from the ruptured nozzle location.
A detailed structural analysis of the rod cluster control guide tubes was performed to establish the equivalent cross section properties and elastic end support conditions. An analytical model was verified both dynamically and statically by subjecting the control rod cluster tube to a concentrated force applied at the transition plate. In addition, the guide tube was loaded experimentally using a load distribution to conservatively approximate the hydraulic loading. The experimental results consist of a load deflection curve for the

rod cluster control guide tube, plus verification of the deflection criteria to ensure rod cluster control insertion.

3.9.2.1.3.2.3 Upper Support Column

Upper support columns located close to the broken nozzle during the hot leg break will be subjected to transverse loads due to cross flow. The loads applied to the columns were computed with a method similar to the one used for the guide tubes, i.e., by taking into consideration the increase in flow across the column during the accident. The columns were studied as beams with variable sections and the resulting stresses were obtained using the reduced section modulus at the slotted portions. The models used for static (or steady-state dynamic) analysis are:

- The upper support, deep beam, and upper core plate were modeled with flat shell elements, the support columns with three-dimensional beam elements, and the fuel assemblies and holddown springs with three-dimensional spring elements. Because of symmetry, a one-eighth slice of the upper package was modeled. The core plate is perforated and was modeled as a geometrically equivalent solid plate that has elastic constants modified according to the theory of perforated plates.

- Columns of two different lengths were modeled: the long columns connecting the plates and the short columns connecting the beam grid with the upper core plate.

- The lower support structure was modeled using a finite-element structural analysis computer program. The lower core plate and upper core support, as well as the lower part of the core barrel, was represented by flat triangular shell elements. Reduced plate strength, due to the perforations, was accounted for by using an equivalent elastic modulus and Poisson ratio in the calculations. This structure was loaded with various vertical forces, due to normal and abnormal operation, and the deflections and stresses are obtained for each case. The experimental values were converted according to basic scaling laws and applied to the prototype structure. The test values are larger, as expected, since they are obtained in the absence of the core plate and support columns structures, making the lower core support more flexible. Using the same model, this code was also used to compute stresses and deformation due to nonuniform temperature distributions. With temperature at the component surfaces and the gradient generated by the γ-heat generation as input for the system code, the deflected shape of the structure was obtained.

Stresses in components, such as the perforated upper and lower core plates, core support plate, and top support plate, were then computed using the stress intensification factor provided by the standard theory of perforated plates.

3.9.2.1.4 Correlation of Test and Analytical Results

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

The program used to establish the integrity of reactor internals has involved extensive design analysis, model testing, and post-hot functional inspection. Additionally, a full-size reactor has been instrumented to measure the dynamic behavior of a plant the size and type of DCPP. Measured values have been compared to predicted values.

The Indian Point II reactor has been established as the prototype for the DCPP Unit 1 internals verification program. The Trojan plant (Portland General Electric Company) provides additional internals verification for Unit 2. (Unit 1 has a thermal shield similar to Indian Point II; Unit 2 has neutron panels similar to Trojan.)

The only significant differences between the DCPP units' internals and Indian Point II are the modifications resulting from the use of a 17 x 17 fuel array in place of 15 x 15, and the replacement of the annular thermal shield with neutron shield panels. The change to neutron shield panels applies only to Unit 2. The change to 17 x 17 applies to both Unit 1 and Unit 2.

The only structural changes in the internals resulting from the design change from the 15 x 15 to the 17 x 17 fuel assembly are in the support columns and assembly guide tubes. The new 17 x 17 guide tubes are stronger and more rigid, hence they are less susceptible to flow-induced vibration problems. The fuel assembly itself is relatively unchanged in mass and spring rate, and thus no significant deviation is expected from the 15 x 15 fuel assembly vibration characteristics.

The remainder of the core structure design is identical to the prototypes that have been tested and proven to be well within design expectations and limits.

The Trojan plant is the lead plant featuring neutron panels and 17 x 17 fuel assemblies.

The Trojan plant internals were instrumented for strain measurements on the core barrel and on the guide tube subject to highest cross flow. The data obtained provided verification of Westinghouse analysis and scale model predictions of neutron panels and 17 x 17 internals behavior in a full-size plant.

The Four Loop Internals Assurance Program conducted on Indian Point II, supplemented by the Trojan data on neutron shield panels and 17 x 17 fuel assemblies, satisfy Safety Guide 20, December 1971 (Reference 12) with respect to adequate plant testing of internals similar to those employed at DCPP. The core support structures received, in addition, the normal pre- and post-hot functional testing examination for integrity in accordance with Paragraph D, "Regulations for Reactor Internals Similar to the Prototype Design," of Safety Guide 20, December 1971. This examination included the points shown in Figure 3.9-1 for Unit 1 and Figure 3.9-2 for Unit 2, and also:

(1) All major load bearing elements of the reactor internals relied on to retain the core structure in place

(2) The lateral, vertical, and torsional restraints provided within the vessel

(3) Those locking and bolting devices whose failure could adversely affect the structural integrity of the internals

(4) Those other locations on the reactor internal components that are similar to those which were examined on the prototype Indian Point II and Trojan designs

The interior of the reactor vessel was also examined for evidence of loose parts or foreign material. Specifically, the inside of the vessel was inspected before and after the hot functional tests, with all the internals removed, to verify that no loose parts or foreign materials were in evidence.

Lower Internals

A particularly close inspection was made on the following items or areas, using a 5x or 10x magnifying glass or penetrant test where applicable. The locations of these areas are shown in Figures 3.9-1 and 3.9-2 for Unit 1 and Unit 2, respectively:

(1) Upper barrel flange and girthweld

(2) Upper barrel to lower barrel girthweld

(3) Upper core plate aligning pin (examine for any shadow marks, burnishing, buffing, or scoring; check for the soundness of lockwelds)

(4) Irradiation specimen basket welds

(5) Baffle assembly locking devices (check for lockweld integrity)

(6) Lower barrel to core support girthweld

(7) For Unit 1, the flexible tie connections (flexures) at the lower end of the thermal shield

(8) For Unit 2, the neutron shield panel locking devices and dowel pin cover plate welds (examine the connections for evidence of change in tightness of lockweld integrity)

(9) Radial support key welds to barrel

(10) Insert locking devices (examine soundness of lockwelds)

(11) Core support columns and instrumentation guide tubes (check all the joints for tightness and soundness of the locking devices)

(12) Secondary core support assembly welds

(13) Lower radial support lugs and inserts (Examine for any shadow marks, burnishing, buffing, or scoring. Checking the integrity of the lockwelds: these members supply the radial and torsional constraint of the internals at the bottom relative to the reactor vessel while permitting axial growth between the two. One would expect to see, on the bearing surfaces of the key and keyway, burnishings, buffing, or shadowing marks that would indicate pressure loading and relative motion between the two parts. Some scoring of engaging surfaces is also possible and acceptable.)

(14) For Unit 1, mounting blocks thermal shield to core barrel (examine the connections for evidence of change in tightness or lockweld integrity)

(15) For Unit 1 and Unit 2, gaps at baffle joints (check for gaps between baffle and top former and at baffle-to-baffle joints)

Upper Internals

A particularly close inspection was made on the following items or areas, using a magnifying glass of 5x or 10x magnification where necessary:

The locations of these areas are shown in Figures 3.9-1 and 3.9-2 for Unit 1 and Unit 2, respectively.

(1) Thermocouple conduits, clamps, and couplings

(2) Guide tube, support column, and thermocouple column assembly locking devices

(3) Support column and conduit assembly clamp welds

(4) Upper core plate alignment inserts (Examine for any shadow marks, burnishing, buffing or scoring. Check for tightness and lock device integrity)

(5) Connections of the support columns, mixing devices, and orifice plates to the upper core plate (check for tightness and lock device integrity)

(6) Thermocouple conduit gusset and clamp welds

(7) Thermocouple end-plugs (check for tightness)

(8) Guide tube closure welds, tube-transition plate welds, and card welds

Acceptance standards are the same as required in the shop by the original design drawings and specifications.

During the hot functional test, the internals were subjected to a total operating time at greater than normal full flow conditions (four pumps operating) of at least 10 days. This provides a cyclic loading of approximately 10^7 cycles on the main structural elements of the internals. In addition, there was some operating time with only one, two, and three pumps operating. No signs of abnormal wear were found, no harmful vibrations were detected, and no apparent structural changes took place; therefore, the four loop core support structures are considered to be structurally adequate and sound for operation.

3.9.2.1.5 Analysis Methods Under LOCA Loadings

The analysis methods used to confirm the structural design adequacy of the RCS under LOCA loadings are described in Section 5.2.1. Since the breaks postulated for the original analyses are more severe than those that are now required to be considered, the original analyses are conservative.

3.9.2.1.6 Analytical Methods for ASME Code Class 1 Components

Plastic instability allowable limits given in ASME Section III are not used when dynamic analysis is performed, except as noted in Section 5.2.1.11. The analysis methods have the limits established by the ASME Section III for Normal, Upset, and Emergency conditions. For these cases, the limits are sufficiently low to ensure that the analysis is not invalidated. For ASME Code Class 1 components, the stress limits for faulted loading conditions are specified in Section 5.2. For ASME components other than Class I and components not covered by the ASME Code, the stress limits for faulted loading conditions are specified in Sections 3.9.2.2 and 3.9.2.3, respectively. These faulted condition limits are established in such a manner that there is an equivalence with the adopted elastic limits and consequently will not invalidate the elastic system analysis.

3.9.2.1.7 Design and Analysis Details for the Pressurizer Safety and Relief System

The method of analysis for safety valves and relief valves suitably accounts for the time-history of loads acting during and subsequent to valve opening (i.e., less than one second). The fluid-induced forcing functions are calculated for pertinent safety valve and relief valve discharge cases using one-dimensional equations for the conservation of mass, momentum, and energy.

The calculated forcing functions are applied at locations along the associated piping. Application of these forcing functions to the associated piping model constitutes the dynamic time-history analysis.

The dynamic response of the piping system is determined for the input forcing functions. Therefore, a dynamic amplification factor is inherently accounted for in the analyses.

Snubbers or strut-type restraints are used as required. The stresses resulting from the loads produced by the sudden opening of a relief or safety valve are combined with stresses due to other pertinent loads and are shown to be less than the allowable limits of the ANSI B31.1-1967 PG&E Quality/Code Class I and ANSI B31.7-1969 with 1970 Addenda for PG&E Quality/Code Class II and III pressurizer safety and relief system components. Also, the analyses show that the loads applied to the nozzles of the safety and relief valves do not exceed the maximum loads specified by the manufacturer.

The pressurizer safety and relief valve (PSARV) discharge piping systems provide overpressure protection for the RCS. The three spring-loaded safety valves, located on top of the pressurizer, are designed to prevent system pressure from exceeding design pressure by more than 10 percent. The three power-operated relief valves, also located on top of the pressurizer, are designed to prevent system pressure from exceeding the normal operating pressure by more than 100 psi. The valve outlet side is sloped to prevent the formation of water pockets. The safety valves have been converted from water-seated to steam-seated, and the water loop seal was eliminated by providing a continuous drain.

The pressurizer safety valves, manufactured by Crosby, are self-actuated, spring-loaded valves with backpressure compensation. The power-operated relief valves, manufactured by Masoneilan, are air-operated globe valves, capable of automatic operation via high pressure signal or remote manual operation. The safety valves and relief valves are located in the pressurizer cubicle and are supported by the attached piping. If the pressure exceeds the setpoints, the valves open. Even though the water loop seal has been eliminated, the analysis has not been revised to reflect any added margins because the valve discharge conditions without the water slug are less severe than those originally considered with the water slug. With a pressurizer safety valve water loop seal (now eliminated), the water slug from the loop seal discharges and the water slug, driven by high system pressure, generates transient thrust forces at each location where a change in flow direction occurs. The valve discharge conditions are considered in the analysis of the PSARV piping systems as follows: (a) the three safety valves remain closed, and (b) the three relief valves open simultaneously while the safety valves are closed. In addition to these two cases, which consider water seal discharge (water slug followed by steam), solid water from the pressurizer (cold overpressure) is also investigated.

For each pressurizer safety and relief piping system, an analytical hydraulic model is developed. The piping from the pressurizer nozzle to the relief tank nozzle is modeled as a series of single pipes. The pressurizer is modeled as a reservoir which contains steam at constant pressure (approximately 2500 psia for safety system and approximately 2350 psia for relief system) and at constant temperature of approximately 680°F. The pressurizer relief tank is modeled as a sink which contains steam and water mixture.

Fluid acceleration inside the pipe generates reaction forces on all segments of the line which are bounded at either end by an elbow or bend. Reaction forces resulting from fluid pressure and momentum variations are calculated. These forces are defined in terms of the fluid properties for the transient hydraulic analysis. Unbalanced forces are calculated for each straight segment of pipe from the pressurizer to the relief tank. The time histories of these forces are used for the subsequent structural analysis of the pressurizer safety and relief lines.

The structural model used in the seismic analysis of the safety and relief lines is modified for the valves thrust analysis to represent the safety and relief valve discharge. The time-history hydraulic forces are applied to the piping system lump mass points. The dynamic solution for the valve thrust is obtained by using a modified predictor-corrector-integration technique and normal mode theory.

The time-history solution is performed in subprogram FIXFM3. The input to this subprogram consists of the natural frequencies and normal modes, applied forces, and nonlinear elements. The natural frequencies and normal modes for the modified pressurizer safety and relief line dynamic model are determined with the WESTDYN program. The support loads are computed by multiplying the support stiffness matrix and the displacement vector at each support point. The time-history displacements of the FIXFM3 subprogram are used as input to the WESDYN2 subprogram to determine the internal forces, deflections, and stresses at each end of the piping elements.

The loading combinations and acceptance criteria considered in the analysis of the pressurizer safety and relief valve (PSARV) piping are given in Table 3.9-1. These load combinations are consistent with the final recommendations of the piping subcommittee of the EPRI PWR PSARV performance test program.

3.9.2.2 ASME Code Class 2 and 3 Components

This section discusses the design criteria for PG&E Quality/Code Class II and III components. The design of these components is based on the requirements of various codes and standards that were in effect when the items were purchased. These codes and standards have been widely used by the nuclear industry and were, to a large extent, incorporated or referenced in the 1971 edition of the ASME Boiler and Pressure Vessel Code, Section III. If the 1971 edition of the ASME Boiler and Pressure Vessel Code, Section III, had been available during the design of DCPP, all these PG&E Quality/Code Class II and III components would have been in accordance with the requirements for ASME Code Class 2 and 3 components.

The DCPP Q-List (refer to Reference 8 of Section 3.2) lists the codes and standards to which PG&E Quality/Code Class II and III components were designed. The quality group classifications for DCPP mechanical systems and components are described in Section 3.2.2.

3.9.2.2.1 Plant Conditions and Design Loading Combinations

Design pressure, temperature, and other loading conditions that provide the bases for design of mechanical systems or components are presented in the corresponding sections that describe the components and systems; refer to Chapters 6, 7, 9, and 11. Design codes, standards, and their applicability to systems and components are presented in DCPP Q-List and Section 3.2.2.

3.9.2.2.2 Design Loading Combinations

Design Criteria for Westinghouse supplied PG&E Quality/Code Class II and III components, are provided in Tables 3.9-2 through 3.9-7. Table 3.9-8 provides design information for selected tanks.

The loading combinations and acceptance criteria used for piping (except for PSARV piping) primary equipment, and primary equipment supports in the Westinghouse scope of analysis are provided in Table 5.2-5, 5.2-6, 5.2-7, and 5.2-8, and also in Table 3.9-2 through 3.9-7.

3.9.2.2.3 Design Stress Limits

Stress limits for Westinghouse supplied PG&E Quality/Code Class II and III components are provided in Table 3.9-2 through 3.9-7. Stress limits were selected to comply with the intent of ASME Code Section III and are sufficiently low to provide assurance that no gross deformation will occur in active components and that the active components will operate as required following the event. Active components are those whose operability is relied upon to perform a safety function such as safe shutdown of the reactor or mitigation of the consequences of a postulated pipe break in the reactor coolant pressure boundary. The limits established for passive (inactive) components are intended to ensure that violation of the pressure retaining boundary will not occur. Passive components are those whose operability (e.g., valve opening or closing, pump operation, or trip) are not relied upon to perform a safety function.

The designs of the condensate storage tanks, refueling water storage tanks, and the fire water and transfer tank are based on the AWWA D100, 1967 Code, with stress allowables restricted to those permitted by ASME Code Section VIII, Division 1. The design basis of these tanks is discussed in Section 3.8.3.

Load combinations and allowable stresses were taken from ANSI B31.1-1973 Summer Addenda. This code was used since the earlier codes did not address these load combinations and allowable stresses.

Piping stresses resulting from the seismic analyses are combined with deadload stresses, pressure stresses, and other stresses caused by other sustained loads, as suggested in ANSI B31.1-1973 Summer Addenda by the following equations:

$$\frac{PDo}{4t_n} + \frac{0.75iM_A}{Z} \leq 1.0S_h \tag{3.9-1}$$

$$\frac{PDo}{4t_n} + \frac{0.75iM_A}{Z} + \frac{0.75iM_B}{Z} \leq 1.2S_h \tag{3.9-2}$$

$$\frac{PDo}{4t_n} + \frac{0.75iM_A}{Z} + \frac{0.75iM_B'}{Z} \leq 1.8S_h \tag{3.9-3}$$

$$\frac{PDo}{4t_n} + \frac{0.75iM_A}{Z} + \frac{0.75iM_B''}{Z} \leq 2.4S_h \tag{3.9-4}$$

$$\frac{iMc}{Z} \leq S_A \tag{3.9-5}$$

where:

S_h	=	basic material allowable stress at operating temperature, psi
P	=	internal pressure, psig
Do	=	outside diameter of pipe, in
t_n	=	nominal wall thickness of pipe, in
Z	=	section modulus, in^3
i	=	stress intensification factor. The product of 0.75 x i shall never be taken as less than 1
M_A	=	resultant moment loading on cross section due to deadload and other sustained loads, in-lb
M_B	=	one-half of the resultant moment due to DE loads plus one-half of the full range of the resultant moment due to DE seismic anchor movements (SAM) if not included in Equations 3.9-5 or 3.9-6
M_B'	=	same as M_B except that moments from DDE are used instead of DE and anchor movements due to DDE are excluded
M_B''	=	same as M_B except the moments from HE are used instead of DE and anchor movements due to HE are excluded
M_C	=	the larger of (a) the full range of resultant moment due to seismic anchor movements (SAM), or (b) the range of resultant moment due to normal thermal expansion and anchor movements plus one-half of the full range of resultant moment due to SAM, in-lb
S_A	=	f (1.25 S_c + 0.25 S_h)
S_c	=	basic allowable stress at cold (ambient) temperature, psi
f	=	stress range reduction factor for cyclic loading = 1 (There are no

events causing more than 7000 loading cycles for DCPP)

If Equation 3.9-5 is not satisfied, then the following equation must be satisfied:

$$\frac{PDo}{4t_n} + \frac{0.75iM_A}{Z} + \frac{iM_C}{Z} \leq (S_h + S_A) \tag{3.9-6}$$

For hydrodynamic loadings the following stress equations must be satisfied:

$$\frac{PDo}{4t_n} + \frac{0.75iM_A}{Z} + \frac{0.75iM_D}{Z} \leq 1.2S_h \tag{3.9-7}$$

$$\frac{PDo}{4t_n} + \frac{0.75iM_A}{Z} + \frac{0.75i}{Z}(M_B{}^2 + M_D{}^2)^{1/2} \leq 1.8S_h \tag{3.9-8}$$

where:

M_D = Resultant moment due to hydrodynamic loads

All PG&E Design Class I pipe stresses were found to be within allowable limits specified in Equations 3.9-1 to 3.9-8.

Maximum allowable stresses for various loading combinations on hangers within B31.1 Code jurisdiction are as follows:

	DE	DDE	HE
Tension	0.417 Fy	0.9 Fy	Least of 1.2 Fy or 0.7 Fu
Shear	0.694 Fv	1.44 Fv	1.44 Fv
Compression	0.694 Fa	1.33 Fa	1.33 Fa
Bending	0.694 Fb	1.5 Fb	1.88 Fb
Bearing	0.625 Fy	Not Applicable	Not Applicable

where Fy, Fu, Fv, Fa, and Fb are from Section 1.5 of the AISC Steel Construction Manual, 7th Edition.

Supporting structures (supplemental steel) are in accordance with the 7th Edition of the AISC Steel Construction Manual. The stress limits and load combinations used by PG&E for equipment and equipment supports for their scope of analysis are:

DE Seismic Event

Component	Stress Limits (see notes a, b, c, d)

(Except cast iron) (active or inactive)	$Q_m \leq 1.1$ S $(Q_m$ or $Q_L) + Q_b \leq 1.65$ S
Inactive cast iron, pressure-retaining components	$Q_p \leq 0.1\ S_u$ $(Q_m$ or $Q_L) + Q_b \leq 1.5 \times 0.1\ S_u$
Inactive cast iron, nonpressure-retaining components	$(Q_m$ or $Q_L) + Q_b \leq 1.0 \times 0.2\ S_u$
<u>Support Element</u>	
Plate and shell (see note e)	$Q_m \leq 1.0$ S $Q_m + Q_b \leq 1.5$ S
Linear (see note f)	1974 ASME Code, Section III, Appendix XVII and Subsection NF
Bolts	1974 ASME Code, Section III, Appendix XVII and/or Code Case 1644, and/or AISC Manual, 7th Edition

DDE/HE Seismic Event

<u>Component</u>	<u>Stress Limits</u> (see notes a, b, c, f, h)
Inactive (Except cast iron)	$Q_m \leq 2.0$ S $(Q_m$ or $Q_L) + Q_b \leq 2.4$ S
Active (Except cast iron)	$Q_m \leq 1.2$ S $(Q_m$ or $Q_L) + Q_b \leq 1.8$ S
Inactive cast iron, pressure-retaining components	$Q_p \leq 0.1\ S_u$ $(Q_m$ or $Q_L) + Q_b \leq 2.4 \times 0.1\ S_u$
Inactive cast iron, nonpressure-retaining components	$(Q_m$ or $Q_L) + Q_b \leq 2.0 \times 0.2\ S_u$
<u>Support Elements</u>	
Plate and shell (see note e) (active components)	$Q_m \leq 1.2$ S $(Q_m + Q_b) \leq 1.8$ S

Plate and shell (inactive components)	$Q_m \leq 2.0$ S $(Q_m + Q_b) \leq 2.4$ S
Linear	1974 ASME Code, Section III, Appendix XVII, (see note f) Subsection NF and Appendix F (Stresses not to exceed S_y (see note g) for active components)
Bolts	1974 ASME Code, Section III, Appendix XVII and/or Code Case 1644 plus Appendix F and/or AISC Manual, 7th Edition

(a) Q_m = general membrane stress, ksi. This stress is equal to the average stress across the solid section under consideration. It excludes discontinuities and concentrations and is produced only by pressure and other mechanical loads.

(b) Q_L = local membrane stress, ksi. This stress is the same as Q_m except that it includes the effect of discontinuities.

(c) Q_b = bending stress, ksi. This stress is equal to the linear varying portion of the stress across the solid section under consideration, excludes discontinuities and concentration, and is produced only by mechanical loads.

(d) S = material allowable stress listed in either 1971 or 1974 ASME Code, Section III, or the code the component was purchased and manufactured under, allowable stress values. The allowable stress shall correspond to the highest metal temperature at the section under consideration during the condition under consideration.

(e) Plate and shell type supports: Plate and shell type component supports are supports such as vessel skirts and saddles that are fabricated from plate and shell elements and are normally subjected to a biaxial stress field.

(f) S_u = material minimum tensile strength listed in either the code the component was purchased and manufactured under, or ASME Code Section III.

(g) S_y = material allowable stress listed in either 1971 or 1974 ASME Code, Section III, or the code the component was purchased and manufactured under, minimum yield stress. The yield stress shall correspond to the highest metal temperature at the section under consideration during the condition under consideration.

(h) Q_p = local membrane stress, ksi. This stress is equal to the average stress across the solid section under consideration. It excludes discontinuities and concentrations and is produced only by pressure.

Load Combinations

(3.1.1)	$DE + P_n + T_n + D + N + O$
(3.1.2)	$DDE + P_a + T_a + D + N + O$
(3.1.3)	$HE + P_n + D + N + O$

where the following loads apply, as applicable:

HE	=	loads from Hosgri earthquake
P_n	=	Pressure, normal
P_a	=	Pressure, accident
T_n	=	Temperature, normal
T_a	=	Temperature, accident
DE	=	DE
DDE	=	DDE
D	=	Deadweight
N	=	Nozzle
O	=	Operating

3.9.2.2.4 Analytical and Empirical Methods for the Design of Pumps and Valves

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

The PG&E Quality/Code Class II and III pumps and valves were designed and constructed to PG&E Design Class I standards and manufactured under approved quality assurance programs. *PG&E inspectors routinely performed audits, inspections, and witnessed testing of Quality Code Class II and III components as they were manufactured.*

The PG&E Quality/Code Class II and III pumps and valves were designed in accordance with the codes and standards listed in the DCPP Q-List (refer to Reference 8 of Section 3.2) and Table 3.2-2. These were the codes and standards that were in effect when the items were purchased. The stress limits selected are sufficiently low to provide assurance that no gross deformations will occur in active components; therefore, the active components will perform as required.

The pumps purchased by Westinghouse were analyzed for the forces resulting from seismic accelerations in the horizontal and vertical directions applied simultaneously. The pumps were designed to have a natural frequency in excess of 30 Hz to eliminate any amplification of the seismic floor accelerations in the pump support structures.

The Westinghouse pumps were subjected to a series of tests prior to installation in the plant. The in-shop tests included (a) hydrostatic tests to 150 percent of the design pressure, (b) seal leakage tests, (c) net positive suction head (NPSH) tests to develop the minimum suction head necessary to allow operation, and (d) functional performance tests.

The pumps purchased by PG&E were designed in accordance with ASME standards or PG&E power plant pump standards. The design standards required were determined for each pump by reviewing the pump service conditions. Seismic calculations provided by the manufacturers were reviewed by PG&E to ensure that the loads developed from the combination of the design seismic horizontal and vertical acceleration did not exceed those allowed by applicable codes or standard engineering practices. Seismic calculations were not requested when the seismic adequacy could be ensured by testing or a comparative review of pump design. Hydrostatic tests to 150 percent of design pressure were performed on the pumps purchased by PG&E. The pumps also were subjected to performance tests consistent with the requirements of the Hydraulic Institute Standards or PG&E's power plant pump standards.

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED.

In addition to the above described tests, which were performed prior to installation in the plant, numerous tests were performed on the pumps during the preoperational test period. Cold hydrostatic pressure tests, hot functional qualification tests, periodic inservice inspections, and periodic inservice operational tests are performed on PG&E Quality/Code Class II and III pumps after installation in the plant. These tests verify the functional ability of the pumps and ensure the operability of active safety-related pumps for the design life of the plant.

The supports of all PG&E Quality/Code Class II and III pumps were designed to withstand the effects of the DE and reviewed for the DDE and HE. These considerations prevent supports of active safety-related pumps from deflecting and impairing the operability of the pump.

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Active pumps were qualified for operability by first being subjected to rigid tests both prior to installation in the plant and after installation in the plant. The in-shop test included (a) hydrostatic tests of pressure-retaining parts to 150 percent of the product of the design pressure times the ratio of material allowable stress at room temperature to the allowable stress value at the design temperature, (b) seal leakage tests, and (c) performance tests to determine total developed head, minimum and maximum head, net positive suction head (NPSH) requirements and other pump parameters. Bearing temperature and vibration levels were monitored during these operating tests. Bearing temperature limits and vibration levels were established by the manufacturer based on bearing materials, clearances, oil type and rotational speed. After a pump was installed in the plant, it underwent cold hydrostatic tests, and hot functional tests, and will undergo the required periodic inservice inspection operation. These tests demonstrated that a pump will function as required during all normal operating conditions for the design life of the plant.

In addition to these tests, the active pumps were qualified for operability by assuring that they will start, continue operating and not be damaged during the postulated Hosgri earthquake.

It was shown that the pumps will perform their design functions when subjected to loads imposed by the maximum seismic accelerations and maximum nozzle loads. It was required that test or analysis be used to show that the lowest natural frequency of each pump was greater than 33 Hz. A pump having a natural frequency above 33 Hz was considered rigid. This consideration avoids amplification between the component and structure for all seismic areas. A static shaft deflection analysis of rotors was performed with horizontal and vertical accelerations acting simultaneously. The deflections, determined from the static shaft analyses, were compared to the allowable rotor clearances. Pump and motor bearing loads were determined and shown to be below the manufacturer's specified levels.

To avoid damage during the postulated earthquake, the stresses caused by the combination of normal operating loads, earthquake, and dynamic system loads were limited to the limits indicated in Table 3.9-2. Pump casing stresses caused by the maximum nozzle loads were limited to the stresses outlined in Table 3.9-2. The maximum seismic nozzle loads combined with the loads imposed by the seismic accelerations were considered in the analysis of pump supports. Furthermore, calculated misalignment was shown to be less than that which could hinder pump operation. The stresses in the supports were below those of Table 3.9-3. Therefore, support distortion is elastic and of short duration (no longer than the duration of the seismic event).

Performing these analyses with the loads and the stress limits of Tables 3.9-2 and 3.9-3, assures that critical parts of pumps will not be damaged during the postulated earthquake.

If the natural frequency was found to be below 33 Hz, an analysis was performed to determine the amplified input accelerations necessary to perform the static analysis. The adjusted accelerations were determined with the same conservatisms used for rigid structures. The static analysis was performed using the adjusted accelerations; the stress limits stated in Tables 3.9-2 and 3.9-3 were satisfied.

To complete the seismic qualifications procedures, the pump motors were qualified for operation during the maximum seismic event. Any auxiliary equipment which is vital to the operation of the pump or pump motor, and which was not qualified for operation with the pump or motor was qualified separately.

The above program gives assurance that the active pumps and motors would not be damaged and would continue operating under seismic loadings. These requirements demonstrate that the active pumps will perform their intended functions.

Since it has been demonstrated that the pumps would not be damaged during the earthquake, the functional ability of the active pumps after the earthquake is assured. Normal operating loads and steady state nozzle loads are the most probable conditions following an earthquake. The ability of the pumps to function under these loads is demonstrated during normal plant operation.

The PG&E Quality/Code Class II and III valves purchased by Westinghouse were designed to the pressure and temperature requirements of the American Standard Association (ASA) B16.5 or the Manufacturers Standardization Society Standard Practice No. 66 (MSS SP66). The valves were tested to the requirements of MSS SP61. These tests included hydrostatic shell and seat leakage tests.

The PG&E Quality/Code Class II and III valves purchased by PG&E were designed, manufactured, and tested in accordance with the Draft ASME Code for Pumps and Valves for Nuclear Power, November 1968 or later editions, the ASME Code, Section III-1974, ANSI B16.5, and/or the MSS SP66.

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In situ seismic testing of five representative valves was performed at the DCPP site. The purpose of this testing was to demonstrate that valves would indeed function when subjected to simulated seismic loads. Each valve was subjected to a static load applied at the center of gravity of the extended structure. The load was applied in the direction that would yield the largest deflection for the given load. While the valve was held in the deflected position, the valve was stroked. Any differences in deflected valve stroking time, line voltage, and motor current (as compared to the undeflected readings) would indicate the effect of a seismic event on valve operability. The valve was again stroked open and closed after removal of the static load to demonstrate that the valve had returned to its initial condition.

The five selected valves tested by this method performed satisfactorily while subjected to the simulated load. The operability indicators of motor current, voltage, and valve stroke time were essentially the same for both the deflected and undeflected tests. During the preoperational piping dynamics effects test program described in Section 3.9.2.1.2, any excessive piping deflections and vibrations were noted and corrected. Since all valves are supported as part of adjoining piping, this program ensures that the deflections by the pipe (and valve) supports will not impair the operability of active safety-related valves. The attention given to the design, manufacture, and testing of the Quality Code Class II and III pumps and valves ensures that the components will operate as required during or following any expected plant transient.

An evaluation and tabulation of all active valves is presented in Table 3.9-9. An active valve is a valve that must perform a mechanical motion in order to shut down the plant or mitigate the consequences of a postulated event. Check valves with flow through the valves secured are considered passive devices. Check valves designed to close without operator action following an accident are considered active devices. The position each valve assumes on power failure is listed in this table.

The design approach and criteria used to ensure the protection of all critical systems and containment from the effects of pipe whip, are presented in Section 3.6. Section 3.6 also presents the criteria for postulated pipe breaks.

3.9.2.2.5 Design and Installation Criteria, Pressure-Relieving Devices

The main steam safety valves are located outside the primary containment directly on main steam leads 1 and 2, and on external headers on main steam leads 3 and 4. Five safety valves are provided for each steam generator, for a total of 20 safety valves. The safety valve headers and main steam lead connections were designed to ANSI B31.1-1967. Fabrication and erection were in accordance with the ASME Boiler and Pressure Vessel Code, Section I-1968.

The safety valves are of the single discharge type and were built in accordance with ASME Boiler and Pressure Vessel Code, Section III. The valve discharge consists of an elbow attached to the safety valve outlet. The elbow discharges into a stack that is structurally independent of the valve and is oriented at approximately 36° to the vertical centerline of the safety valve. The stacks are supported to restrain the discharge reactions. Provisions are made to ensure that the safety valve discharge elbow and stack have adequate clearances during all phases of operation.

With the above safety valve discharge arrangement, the sustained blow force developed during valve operation intersects the vertical centerline of the safety valve header nozzle and the base of the header nozzle extrusion. The safety valve nozzles on the headers and on the main steam leads have been analyzed to ensure that the sustained forces developed during valve operation will not develop stresses in excess of those allowed by ANSI B31.1-1967.

The main steam leads are anchored by the main steam flued heads which are structurally located in the reactor containment wall and are supported for the deadweight, thermal, seismic, and safety valve forces that may develop within the PG&E Design Class I portion. The two external safety valve headers on main steam leads 3 and 4 are independently supported in a similar fashion.

3.9.2.2.6 Mechanical Equipment Seismic Qualification

Table 3.9-12 identifies PG&E Design Class I equipment that has been seismically qualified. The location, qualification method, seismic spectra, and damping values are part of the qualification.

3.9.2.2.7 Small Bore Piping Systems

DCPP Unit 1 and Unit 2 small bore PG&E Design Class I piping is analyzed by computer or designed using simplified spacing criteria.

3.9.2.3 Components Not Covered by ASME Code

3.9.2.3.1 Core and Internals Integrity Analysis (Mechanical Analysis)

HISTORICAL INFORMATION IN ITALICS BELOW NOT REQUIRED TO BE REVISED

Stainless steel clad silver-indium-cadmium alloy absorber rods are resistant to radiation and thermal damage, thereby ensuring their effectiveness under all operating conditions. *Rods of similar design have been successfully used in the original and reload cores of San Onofre, Connecticut Yankee, and others.*

Two burnable poison rods (Reference 6) of smaller length but similar in design to those used in DCPP were exposed to in-pile test conditions in the Saxton Test Reactor in October 1967. A visual examination of the rods was made in early June 1968 and a visual and profilometer examination was made on July 30, 1968, after an exposure of 1900 effective full power hours (approximately 25 percent B^{10} depletion). The rods were found to be in excellent condition and profilometry results showed no dimensional variation from the initial condition.

An experimental verification of the reactivity worth calculations for borosilicate glass tubing has been accomplished. Similar rods have been successfully operated in the Ginna Reactor (Reference 7) with no evidence of deficiency.

Manufacturing defects did not appear during the hot functional tests because any manufacturing defects were detected in the shop or during the assembly period. The basic program that is currently being used to ensure adequacy of manufacturing practices consists of:

(1) *Extremely thorough nil ductility temperature and quality assurance programs at the internals vendors*

(2) *Extensive visual examination at the plant site prior to hot functional testing of the primary system*

(3) *Running the hot functional test with full flow for 240 hours that accumulates approximately 10^7 cycles on the majority of the core structure components*

(4) *Reexamining all areas of the internals after the 240-hour hot functional test*

The response of the reactor core and vessel internals under excitation produced by a simultaneous complete severance of a reactor coolant pipe and seismic excitation for a typical Westinghouse pressurized water reactor plant internals was determined. The following mechanical functional performance requirements applied:

(1) Following the DBA, the basic operational or functional requirement to be met for the reactor internals is that the plant shall be shut down and cooled in an orderly fashion so that fuel cladding temperature is kept within specified limits. This implies that the deformation of certain critical reactor internals must be kept sufficiently small to allow core cooling.

(2) For large breaks, the reduction in water density greatly reduces the reactivity of the core, thereby shutting down the core whether the rods are tripped or not. The subsequent reflooding of the core by the ECCS with borated water maintains the core in a subcritical state. Therefore, the main requirement is to ensure effectiveness of the ECCS. Insertion of the control rods, although not needed, gives further assurance of the ability to shut the plant down and keep it in a safe shutdown condition.

(3) The functional requirements for the core structures during the DBA are shown in Table 3.9-10. The inward upper barrel deflections are controlled to ensure no contacting of the nearest rod cluster control guide tube. The outward upper barrel deflections are controlled in order to maintain an adequate annulus for the coolant between the vessel inner diameter and core barrel outer diameter.

(4) The rod cluster control guide tube deflections are limited to ensure operability of the control rods.

(5) To ensure no column loading of rod cluster control guide tubes, the upper core plate deflection is limited to the value shown in Table 3.9-10.

(6) The reactor has mechanical provisions that are sufficient to maintain the design core and internals and to ensure that the core is intact with acceptable heat transfer geometry following transients arising from the DBA operating conditions (References 2, 8, and 13).

(7) The core internals are designed to withstand mechanical loads arising from DE, DDE, HE and pipe ruptures (References 2, 4, 8, and 13). Note these loads are combined as described in Section 3.9.2.3.3.

3.9.2.3.2 Faulted Conditions

The following events were considered in the faulted conditions category:

(1) Loads produced by a double-ended pipe rupture of the main coolant loop DBA for both the cold and hot leg breaks (LOCA event). The methods of analysis adopted were related to the type of accident assumed (cold leg break or hot leg break).

(2) Response due to a DDE or HE, as described previously in the seismic analysis

(3) Most unfavorable combination of DDE and DBA. Maximum stresses obtained in each case were added in the most conservative manner.

Maximum stress intensities are compared with allowables for each condition. When fatigue is of concern, the applicable stress concentrations factors are utilized and peak stresses are used to establish the usage factor. Elastic analysis is used to obtain the response of the structure and the stress analysis for each component is performed on an elastic basis. For faulted conditions, stresses are above yield in a few locations. For these cases only, when deformation requirements exist, a plastic analysis is independently performed to ensure that functional requirements are maintained (guide tubes deflections and core barrel expansions). The elastic limit allowable stresses are used to compare with the results of the analysis. No inelastic stress limits are used.

These analyses showed that the stresses and deflections that would result following a faulted condition are less than those that would adversely affect the integrity of the structures. Also, the natural and applied frequencies were such that resonance problems should not occur.

3.9.2.3.3 Reactor Internals Response Under LOCA and Seismic Excitations

The reactor vessel/internals/fuel system dynamic analyses for Diablo Canyon Units 1 & 2 were performed in the 1987-1988 timeframe to verify structural adequacy of the core during transition from 17x17 standard fuel to 17x17 VANTAGE 5 fuel. These dynamic analyses were performed for the LOCA and seismic design conditions of DE, DDE, and Hosgri; and details of these analyses are given in Reference 15.

The system mathematical models for Diablo Canyon Units 1 & 2 used in the LOCA and seismic analyses are three-dimensional nonlinear finite element models, which are described in detail in Reference 15. The major difference between the LOCA and seismic models is that the seismic model includes the hydrodynamic mass matrices in the vessel/barrel downcomer annulus to account for the fluid-solid interactions. The fluid-solid interactions in the LOCA analysis are accounted through the hydraulic forcing functions generated by Multiflex Code (Reference 3). Another difference between the LOCA and seismic models is the difference in loop stiffness matrices. The seismic model uses the unbroken loop stiffness matrix, whereas the LOCA model uses the broken loop stiffness matrix. Except for these two differences, the RPV system seismic model is identical to that of the LOCA model.

It is important to note that the LOCA analyses described below are the analyses originally performed for the RCS faulted conditions. With the acceptance of the DCPP leak-before-break (LBB) by the USNRC (Reference 14), the dynamic LOCA loads resulting from the pipe rupture events in the main reactor coolant loop piping no longer have to be considered in the design basis structural analyses and including the loading combinations. With LBB acceptance, the next most limiting breaks which need to be considered are the auxiliary line breaks consisting of accumulator line, pressurizer surge line and RHR line. The LOCA loads imposed on the RPV system from these auxiliary line breaks are generally significantly lower than those obtained from the main loop line breaks discussed below. This reduction in LOCA loads for the auxiliary line breaks is due to the fact that the auxiliary lines have a smaller break size, location of the breaks are farther away from the vessel nozzles, and the absence of cavity pressurization loads.

It should also be noted that, in general, for faulted conditions the imposed loading on the reactor vessel and its internals due to seismic (DDE) and LOCA conditions are additive by the square root of the sum of squares (SRSS) method. A Hosgri Earthquake (HE) and LOCA are also considered to occur simultaneously and, therefore, the combined loading is considered by SRSS. Therefore, with LBB invoked, the combination of LOCA and seismic loads on the RPV system will be considerably lower than those obtained from main loop piping breaks.

3.9.2.3.3.1 Reactor Internals Response Under Seismic Excitations.

The seismic analysis included the effects of simultaneous application of time history accelerations in three orthogonal directions. The Westinghouse generated synthesized time history accelerations for DE, DDE, and Hosgri response spectra were used in a 1987-1988 analyses. The references of these Westinghouse generated synthesized time histories are also given in Reference 15.

As mentioned earlier, fluid-structure or hydroelastic interaction is included in the reactor pressure vessel model for seismic evaluations. The horizontal hydroelastic interaction is significant in the cylindrical fluid flow region between the core barrel and the reactor vessel annulus. Mass matrices with off-diagonal terms (horizontal degrees-of-freedom only) attach between nodes on the core barrel, thermal shield and the reactor vessel shell (see, e.g., Figure 2-5 of Reference 15, assembled finite element model for Unit 1). The mass matrices for the hydro-elastic interactions of two concentric cylinders are developed using the work of Reference 17. For the case of an incompressible, frictionless fluid displaced in the annulus due to motion of the cylinders, the expression for the hydrodynamic mass matrix connecting the inner and outer cylinders is derived. The diagonal terms of the mass matrix are similar to the lumping of water mass to the vessel shell, thermal shield, and core barrel. The off-diagonal terms reflect the fact that all the water mass does not participate when there is no relative motion of the vessel and core barrel. It should be pointed out that the hydrodynamic mass matrix has no artificial virtual mass effect and is derived in a straight-forward, quantitative manner.

The matrices are a function of the properties of two cylinders with the fluid in the cylindrical annulus, specifically, inside and outside radius of the annulus, density of the fluid and length of the cylinders. Vertical segmentation of the reactor vessel and the core barrel allows inclusion of radii variations along their heights and approximates the effects of beam mode deformation. These mass matrices were inserted between the selected nodes on the core barrel, thermal shield, and the reactor vessel (see Figure 2-5 of Reference 15).

The WECAN computer code, which is used to determine the response of the reactor vessel and its internals, is a general purpose finite element code. In the finite element approach, the structure is divided into a finite number of discrete members or elements. The inertia and stiffness matrices, as well as the force array, are first calculated for each element in the local coordinates. Employing appropriate transformations, the element global matrices and arrays are assembled into global structural matrices and arrays, and used for dynamic solution of the differential equation of motion for the structure.

Note that the preceding paragraphs describe the RPV and internals system dynamic analyses for which the WECAN computer code was used. Current analyses (such as the dynamic analyses performed in support of the replacement vessel head project) utilize the ANSYS computer code. The methodology used to develop the ANSYS system models is consistent with the methodology used to develop historic WECAN models. The direct time integration method is used in ANSYS to solve the dynamic equations of motion for the system; whereas the nonlinear mode superposition method is used in WECAN to solve the dynamic equations of motion for the system.

3.9.2.3.3.2 Reactor Pressure Vessel Internal Hydraulic Loads

Depressurization waves propagate from the postulated break location into the reactor vessel through either a hot leg or a cold leg nozzle. After a postulated cold leg break, the depressurization path for waves entering the reactor vessel is through the nozzle that contains the broken pipe and into the region between the core barrel and the reactor vessel (that is, the downcomer region). The initial wave propagates up, around, and down the downcomer annulus, then up through the region circumferentially enclosed by the core barrel, that is, the fuel region. In the case of a cold leg break, the region of the downcomer annulus close to the break depressurizes rapidly but, because of the restricted flow areas and finite wave speed (approximately 3000 feet per second), the opposite side of the core barrel remains at a high pressure. This results in a net horizontal force on the core barrel and the reactor vessel. As the depressurization wave propagates around the downcomer annulus and up through the core, the core barrel differential pressure reduces and, similarly, the resulting hydraulic forces drop.

In the case of a postulated break in the hot leg, the wave follows a similar depressurization path, passing through the outlet nozzle and directly into the upper internals region depressurizing the core and entering the downcomer annulus from the bottom exit of the core barrel. Thus, after a hot leg break, the downcomer annulus would be depressurized with very little difference in pressure forces across the outside diameter of the core barrel. A hot leg break produces less horizontal force because the depressurization wave travels directly to the inside of the core barrel (so that the downcomer annulus is not directly involved), and internal differential pressures are not as large as for a cold leg break of the same size. Since the differential pressure is less for a hot leg break, the horizontal force applied to the core barrel is less for hot leg break than for a cold leg break. For breaks in both the hot leg and cold leg, the depressurization waves continue to propagate by reflection and translation through the reactor vessel and loops.

3.9.2.3.3.2.1 Reactor Internals Response During Loss-of-Coolant-Accident (LOCA) Conditions

The mechanical response of the reactor coolant system subjected to a LOCA transient is performed in three steps. First, the reactor coolant system is analyzed for the effects of loads induced by normal operation which include thermal, pressure and dead weight effects. From this analysis, the loop mechanical forces acting on the RPV that would result from the release of equilibrium forces at the break locations are obtained. In the second step, the loop mechanical loads, reactor internal hydraulic forces, jet impingement forces, and reactor cavity pressurization forces are simultaneously applied; and the RPV displacements due to the LOCA are calculated. Finally, the structural integrity of the reactor coolant loop and component supports to deal with the LOCA are evaluated by applying the reactor vessel displacements to a mathematical model of the reactor coolant loop.

In 1987-1988, the RPV system LOCA analyses for the Diablo Canyon units were performed for the most limiting breaks consisting of: (a) RPV inlet nozzle break, (b) RPV outlet nozzle break, and (c) RCP outlet nozzle break. These break locations have been determined by detailed stress and fatigue analyses of the reactor coolant loop piping system (Reference 18). As mentioned earlier, the RPV system finite element model for LOCA analysis is identical to that of the seismic model except that it does not have hydrodynamic mass matrices in the downcomer region.

In 2005, the RPV system LOCA analysis was performed for the Unit 2 barrel/baffle region conversion from downflow to the upflow configuration. For DCPP Unit 2, considering LBB acceptance, the next most limiting auxiliary line breaks are the pressurizer surge line break (98.31 in^2) on the hot leg and the accumulator line break (60.13 in^2) on the cold leg. Postulated residual heat removal (RHR) auxiliary line breaks are bounded by the pressurizer surge line break for Unit 2.

In order to study LOCA hydraulic forces for the DCPP Unit 2 Upflow Conversion Program, the following vessel/internal break cases were analyzed:

1. Pressurizer Surge Line Break
2. Accumulator Line Break

A 1 millisecond break-opening-time (BOT) was employed in the vessel forces analyses for DCPP Unit 2, consistent with the MULTIFLEX licensing requirements. All break cases used flexible beam modeling for the core barrel. The analysis conservatively assumed a limiting full power RCS cold leg temperature of 526°F (including uncertainty) which bounds the minimum cold leg temperature of 531.9°F (excluding uncertainty) for DCPP Unit 2. These conditions bound the most severe operating conditions, which encompasses Tavg coastdown conditions. Thus, the effects of a Tavg coastdown are accounted for in the vessel forces analysis. In addition, the Delta-54 replacement steam generator (RSG) was accounted for in the analyses. Previous studies have shown that the steam generator design has a relatively insignificant effect on the vessel forces analyses, so operation of Unit 2 with either the original Model 51 steam generators or the Delta-54 stream generators is acceptable with respect to the vessel forces analyses.

Following a postulated LOCA pipe rupture, forces are imposed on the reactor vessel and its internals. These forces result from the release of the pressurized primary system coolant. The release of pressurized coolant results in traveling depressurization waves in the primary system. These depressurization waves are characterized by a wave-front with low pressure on one side and high pressure on the other. The wave-front translates and reflects throughout the primary system until the system is completely depressurized. The rapid depressurization results in transient hydraulic loads on the mechanical equipment of the system. The release of coolant resulting from a postulated RPV nozzle break also results in a pressure increase in the region surrounding the postulated break. Pressurization occurs rapidly in the cavity around the reactor vessel, which can exert an asymmetric force on the outside of the vessel.

The loads on the RPV and internals that result from the depressurization of the system and from the pressurization of the area around the break may be categorized as: (a) reactor internal hydraulic loads (vertical and horizontal), (b) reactor coolant loop mechanical loads, (c) reactor cavity pressurization loads (only for breaks at the RPV safe end locations), and (d) jet impingement loads. Description of such loads acting for a typical reactor vessel inlet or outlet nozzle is given below (for more details, see Reference 6), and these loads are combined into a single time history forcing function which are then applied to the RPV system finite element model.

3.9.2.3.4 Blowdown Forces Due to Cold and Hot Leg Break

A USNRC approved FORTRAN-IV computer program called MULTIFLEX (Reference 3) is used to calculate the local fluid pressure, flow, and density transients that occur during a LOCA. MULTIFLEX is an extension of the BLODOWN-2 computer code and includes mechanical structure models and their interaction with the thermal-hydraulic system.

The MULTIFLEX computer code (Reference 3) calculates the hydraulic transients within the entire primary coolant system. It considers subcooled, transition, and two-phase (saturated) blowdown regimes. The MULTIFLEX code employs the method of characteristics to solve the conservation laws, and assumes one-dimensionality of flow and homogeneity of the liquid-vapor mixture. The MULTIFLEX code considers a coupled fluid-structure interaction by accounting for the deflection of constraining boundaries, which are represented by separate spring-mass oscillator system. A beam model of the core support barrel has been developed from the structural properties of the core barrel; in this model, the pressure as well as the wall motions are projected onto the plane parallel to the broken nozzle. The spatial pressure variation at each time step is transformed into ten horizontal forces, which act on the ten mass points of the beam model. Each flexible wall is bounded on either side by a hydraulic flow path. The motion of the flexible wall is determined by solving the global equations of motions for the masses representing the forced vibration of an undamped beam.

The analysis is performed for the subcooled decompression period of the transient, where the hydraulic loads are the greatest. These loads are used for the structural evaluation of the reactor pressure vessel support system, in conjunction with other loads associated with a LOCA and with a safe shutdown earthquake (SSE).

The reanalysis performed in support of the conversion of the barrel/baffle region for Unit 2 has made use of the MULTIFLEX 3.0 (Reference 9) computer code. The MULTIFLEX versions are an extension of the BLODWN-2 computer code and includes mechanical structure models and their interactions with the thermal-hydraulic system. Both versions of the MULTIFLEX code share a common hydraulic modeling scheme, with differences being confined to a more realistic downcomer hydraulic network and a more realistic core barrel structural model that accounts for non-linear boundary conditions and vessel motion. Generally, this improved modeling results in lower, more realistic, but still conservative hydraulic forces on the core barrel. The NRC staff has accepted (Reference 10) the use of MULTIFLEX 3.0 for calculating the hydraulic forces on reactor vessel internals (Reference 11).

3.9.2.3.4.1 FORCE2 and LATFORC Models for Blowdown

The MULTIFLEX code evaluates the pressure and velocity transients throughout the RCS. These pressure and velocity transients are made available to the programs FORCE2 and LATFORC, which utilize detailed geometric descriptions in evaluating the loadings on the reactor internals.

LATFORC (Reference 3) is used to calculate the horizontal force components on the vessel, core barrel, and thermal shield as a function of elevation and time using the MULTIFLEX hydraulic data. The force components significant to the horizontal forces are primarily a function of the pressure times area.

FORCE2 (Reference 3) is used to calculate the vertical force components acting on the reactor vessel and internals. Each reactor component for which FORCE2 calculations are required is designated as an element and assigned an element number. Forces acting on each of the elements are calculated summing the effects of:

(1) The pressure differential across the element.

(2) Flow stagnation on, and unrecovered orifice losses across, the element.

(3) Friction losses along the element.

The most significant assumption made for the analysis is that the thermal-hydraulic analysis has been performed to include mechanical structural models of the core barrel, which allows for fluid structure interaction in the downcomer region of the vessel to decrease the peak pressures calculated on the core barrel and vessel. No other fluid structure interaction has been modeled in the vessel LOCA forces calculation.

3.9.2.3.4.2 Reactor Coolant Loop Mechanical Loads

The loop mechanical loads result from the release of normal operating forces present in the pipe prior to the separation as well as transient hydraulic forces in the reactor coolant system. The magnitudes of the loop release forces are determined by performing a reactor coolant loop analysis for normal operating loads (that is, pressure, thermal, and deadweight). The loads existing in the pipe at the postulated break location are calculated and are "released" at the initiation of the LOCA transient by application of the loads to the broken piping ends. These forces are applied with a ramp time of one millisecond because of the assumed instantaneous break opening time.

3.9.2.3.4.3 Reactor Cavity Pressurization Loads

Reactor cavity forces arise from the steam and water that are released into the reactor cavity through the annulus around the broken pipe. These forces occur only for postulated breaks at the RPV nozzle safe end locations. The reactor cavity is pressurized asymmetrically, with high pressures on the side adjacent to the break. The horizontal differences in pressure across the reactor cavity result in horizontal forces on the reactor vessel. Vertical forces on the reactor vessel arise from similar variations in pressures on the upper and lower head and the tapered parts of the vessel.

3.9.2.3.4.4 Jet Impingement Loads

The jet impingement load is an axial force along the broken pipe centerline that is caused by the pressure of the escaping jet of coolant acting on the exposed pipe cross section at the break location. The jet force is calculated by multiplying the saturation pressure corresponding to the temperature of the coolant at break location times the cross-sectional area of the pipe. This force is applied with a ramp time of one millisecond.

3.9.2.3.5 Acceptance Criteria

3.9.2.3.5.1 Structural Adequacy of Reactor Internal Components

The reactor internal components of DCPP Unit 1 and Unit 2 are not ASME Code components because Sub-section NG did not exist in the ASME Boiler and Pressure Code. However, these components were originally designed to meet the intent of the 1971 Edition of Section III of the ASME Boiler and Pressure Vessel Code with addenda through Winter 1971 including the 1971 draft version of Subsection NG. The allowable stress limits for the design basis accident (DBA) for core support structures are based on limits specified in the ASME Code, Subsection NG, Winter 1973 Addenda, and Appendix F, Winter 1972 addenda.

3.9.2.3.5.2 Allowable Deflection and Stability Criteria.

The criterion for acceptability in regard to mechanical integrity analyses is that adequate core cooling and core shutdown must be ensured. This implies that the deformation of reactor internals must be sufficiently small so that the geometry remains substantially intact. Consequently, the limitations established on the reactor internals are concerned principally with the maximum allowable deflections and stability of the components.

For faulted conditions, deflections of critical internal structures are limited to values given in Table 3.9-10. In a hypothesized vertical displacement of internals, energy absorbing devices limit the displacement to 1.25 inches by contacting the vessel bottom head.

Upper Core Barrel

The upper core barrel has the following deformation limits:

(1) To ensure shutdown and cooldown of the core during cold leg blowdown, the basic requirement is a limitation on the outward deflection of the barrel at the locations of the inlet nozzles connected to unbroken lines. A large outward deflection of the upper barrel in front of the inlet nozzles, accompanied with permanent strains, could close the inlet area and restrict the cooling water coming from the accumulators. Consequently, a permanent barrel deflection in front of the unbroken inlet nozzles larger than a certain limit, called “no loss of function” limit, could impair the efficiency of the ECCS.

(2) During the hot leg break, the rarefaction wave enters through the outlet nozzle into the upper internals region and thus depressurizes the core and then enters the downcomer annulus from the bottom exit of the core barrel. This depressurization of the annulus region subjects the core barrel to external pressures and this condition requires a stability check of the core barrel during hot leg break. Therefore, to ensure rod insertion and to avoid

disturbing the control rod cluster guide structure, the barrel should not interfere with the guide tubes.

Control Rod Cluster Guide Tubes

The deflection limits of the guide tubes were established from test data (refer to Table 3.9-10).

Upper Package

The local vertical deformation of the upper core plate, where a guide tube is located, shall be less than 0.100 inch. This deformation will cause the plate to contact the guide tube, since the clearance between the plate and the guide tube is 0.100 inch. This limit will prevent the guide tubes from undergoing compression. For a plate local deformation of 0.150 inch, the guide tube will be compressed and deformed transversely to the upper limit previously established. Consequently, the value of 0.150 inch is adopted as the no loss function local deformation with an allowable limit of 0.100 inch. These limits are given in Table 3.9-10.

3.9.2.3.6 Methods of Analysis

Faulted condition LOCA analyses were originally performed for limiting breaks of the reactor vessel inlet nozzle and reactor vessel outlet nozzle with a limited displacement allowing a break area of 115 in^2. Subsequent calculations of the loop displacements found the maximum displacement at the reactor vessel inlet and outlet nozzles was 81in^2, confirming the 115 in^2 break area was a conservative assumption. These original 115 in^2 break area forces were later confirmed to be bounding relative to LOCA forces generated for 81 in^2 limited displacement breaks calculated at reduced operating temperatures consistent with temperature coastdown. Although the leak-before-break analysis (Reference 14) now allows for exclusion of main loop piping breaks from the design basis, no credit has yet been taken for the smaller branch line areas (60 in^2 for the largest cold leg branch line - the accumulator line) in the current reactor vessel LOCA forces analyses for Unit 1. When credit for this break area reduction is taken, it is expected to provide substantial margin relative to the existing design basis accident loads.

For the Unit 2 conversion to the upflow configuration, as previously mentioned, the pressurizer surge line break (98.31 in^2) on the hot leg and the accumulator line break (60.13 in^2) on the cold leg were analyzed with the MULTIFLEX 3.0 code. The analysis used a 1 millisecond BOT, consistent with the MULTIFLEX licensing requirements. All break cases used flexible beam modeling for the core barrel. The analysis conservatively assumed limiting full power RCS temperatures for DCPP Unit 2. These conditions bound the most severe operating conditions, which encompasses Tavg coastdown conditions. The effects of a Tavg coastdown are thus accounted for in the vessel forces analyses. Additionally, the Delta-54 RSG was included in the analyses. Previous studies have shown that the steam generator design has a relatively insignificant effect on the vessel forces analyses, so operation of either Unit 1 or Unit 2 with either the original Model 51 steam generator or the Delta-54 steam generator configuration is acceptable with respect to the vessel forces analyses.

3.9.2.3.6.1 Reactor Vessel/Internals/Fuel Analysis Under LOCA Conditions

The three dimensional LOCA analysis of the RPV system (i.e., reactor vessel/internals/fuel) is discussed in Section 3.9.2.1.3.1, and Section 3.9.2.1.3.2 provides insight into the description of major core support components during LOCA transients.

3.9.2.3.6.2 Reactor Vessel/Internals/Fuel Analysis Under Seismic Conditions

The three dimensional Seismic analysis of the RPV system (i.e., reactor vessel/internals/fuel) is discussed in Section 3.7.2, and the methods of analyses for seismic loads of major subsystems are discussed in Section 3.7.3.

3.9.2.3.6.3 Methods and Results (Mechanical)

To verify structural adequacy of the reactor internal components and the core under LOCA and seismic loading, nonlinear time history dynamic analyses of the RPV system were performed to generate component interface loads as well as the time history displacements of the lower core plate, upper core plate, and the core barrel. These time history displacements of the core plates and the barrel were then used to determine the fuel grid impact loads and the structural adequacy of the core components.

Reference 15 documents in detail the results of RPV system LOCA and seismic analyses. From these analyses it is seen that the reactor internals component interface loads for DCPP Unit 1 and Unit 2 during LOCA, seismic and combined (SRSS [LOCA + seismic]) are bounded by those of the Generic 4-Loop Stress Report (Reference 19). In the generic stress report, the four-loop reactor internals components are analyzed to meet the ASME Code stress requirements.

The results also indicate that the maximum deflections in the critical structures are below the established allowable limits (refer to Table 3.9-10). During the hot leg break, the core barrel does not buckle, and during the cold leg break has stresses that are within allowable limits. The design evaluation of the internals structure is presented in Section 4.2.2.

It should be reiterated that LOCA analyses described above are for the main loops line breaks; and with the LBB acceptance, the next most limiting breaks which need to be considered are the auxiliary line breaks consisting of the accumulator line, the pressurizer surge line, and the RHR line. The LOCA loads imposed on the RPV system from these auxiliary line breaks are generally significantly lower than those obtained from the main loop line breaks. Therefore, with LBB the combination of LOCA and seismic loads on the RPV system will yield higher margins of safety.

For DCPP Unit 2, an upflow conversion in conjunction with the upper head temperature reduction program has been implemented. The impacts due to these modifications were evaluated and documented in Reference 20.

3.9.2.3.7 Control Rod Drive Mechanisms

The control rod drive mechanisms are PG&E Quality/Code Class I components designed to meet the stresses of the ASME Boiler and Pressure Vessel Code and therefore are presented in Section 4.2.3.

3.9.2.4 Non-Design Class I Components

Several PG&E Design Class II and III components were also seismically qualified to preclude seismic interaction with PG&E Design Class I equipment and/or to ensure structural integrity of the component cooling water system.

3.9.2.5 Miscellaneous Pressurized Gas Containers

Table 3.9-11 provides a summary of all storage tanks containing significant quantities of gas under pressure in excess of 100 psig. These tanks are of both PG&E Design Class I and II.

3.9.3 Safety Evaluation

3.9.3.1 General Design Criterion 1, 1967 – Quality Standards

The PG&E Design Class I mechanical systems and components, as discussed in Section 3.2.2 and Chapter 17 implements quality standards that satisfy the requirements of GDC 1, 1967. Refer to Section 3.9.2.2.

3.9.3.2 General Design Criterion 2, 1967 – Performance Standards

The PG&E Design Class I mechanical systems and components are designed to withstand the effects of winds and tornadoes (refer to Section 3.3), floods and tsunamis (refer to Section 3.4), external missiles (refer to Section 3.5), earthquakes (refer to Section 3.7), and other natural phenomena, and to protect SSCs to ensure their safety-related functions and designs are maintained. Refer to Sections 3.9.2.1, 3.9.2.2, and 3.9.2.3 for detailed discussion of the mechanical components design basis.

3.9.3.3 General Design Criterion 4, 1987 – Environmental and Dynamic Effects Design Bases

Refer to Section 3.6.2.1.1.1 for discussion of the leak-before-break (LBB) methodology and application to the primary loops of DCPP Unit 1 and Unit 2.

3.9.3.4 General Design Criterion 9, 1967 – Reactor Coolant Pressure Boundary

The PG&E Quality/Code Class I mechanical systems and components of the reactor coolant pressure boundary are designed to have an exceedingly low probability of gross rupture or significant leakage throughout its design lifetime as described in Section 5.2. The loads imposed on the reactor coolant pressure boundary under normal operating conditions, abnormal loading conditions, such as seismic loading and pipe rupture are considered and discussed in Sections 3.6 and 3.7. Refer to Sections 3.9.2.1.5, 3.9.2.1.6, 3.9.2.1.7 for additional information.

3.9.3.5 General Design Criterion 33, 1967 – Reactor Coolant Pressure Boundary Capability

The capability of the reactor coolant pressure boundary is described in Section 5.2.

3.9.3.6 General Design Criterion 40, 1967 – Missile Protection

The PG&E Design Class I mechanical systems and components engineered safety features (ESF) as discussed in Sections 3.5, 3.6.3.2, 3.9.2.1.3 and 3.9.2.3 are designed to or are provided with protection from dynamic effects that might result from plant equipment failure.

3.9.3.7 10 CFR 50.55a, Codes and Standards

PG&E Quality/Code Class I is applied to those components of the reactor coolant pressure boundary and implements the quality standards that satisfy the requirements of 10 CFR 50.55a. PG&E Quality/Code Class I components of the reactor coolant pressure boundary are listed in the DCPP Q-List, along with the industry codes and standards used for their design, fabrication, erection, and testing (Refer to Table 3.2-2).

3.9.3.8 10 CFR 50.55a(f) – Inservice Testing Requirements

The IST requirements for the PG&E Design Class I mechanical systems and components (i.e. active pumps and valves) are contained in the DCPP Unit 1 and Unit 2 IST Program Plan. Refer to Section 3.9.2.2.4 for analytical and empirical methods for design of pumps and valves. Refer to Table 3.9-9 for an evaluation and tabulation of all active valves.

3.9.3.9 Safety Guide 20, December 1971 - Vibration Measurements on Reactor Internals

DCPP Unit 1 and Unit 2 reactor internals, as discussed in Table 14.1-1, following hot functional testing, the reactor internals were removed and inspected for signs of excessive vibration. Refer to Section 3.9.2.1.4 for additional discussion about preoperational testing of reactor internals.

3.9.3.10 NUREG-0737 (Item II.D.1), November 1980 - Clarification of TMI Action Plan Requirements

Item II.D.1 - DCPP Unit 1 and Unit 2 participated in the Electric Power Research Institute (EPRI) Test Program performance testing of the relief and safety valves and associated piping and supports which demonstrated the ability of the reactor coolant system power operated relief valves (PORVs), PORV block valves, safety valves, and associated piping to function under expected operating conditions for design basis transients and accidents. Refer to Section 3.9.2.1.7 for additional information.

3.9.4 References

1. Deleted in Revision 1.

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3. Takeuchi, K. et al., MULTIFLEX, A FORTRAN IV Computer Program for Analyzing Thermal-Hydraulic Structure System Dynamics," WCAP-8708-P-A (Westinghouse Proprietary Class 2), WCAP-8709-A, NES Class 3 (Non-Proprietary), September 1977.

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5. Bohm, and J. P. Lafaille, Reactor Internals Response under a Blowdown Accident, First Intl. Conf. on Structural Mech. in Reactor Tech., Berlin, September 20-24, 1971.

6. Wood et al., Use of Burnable Poison Rods in Westinghouse Pressurized Water Reactors, WCAP-7113, October 1967.

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3.10 SEISMIC DESIGN OF DESIGN CLASS I INSTRUMENTATION, HVAC, AND ELECTRICAL EQUIPMENT

3.10.1 SEISMIC DESIGN CRITERIA

The Design Class I instrumentation, HVAC, and electrical equipment are capable of performing their nuclear safety functions during and after a DDE or the postulated 7.5M HE. The seismic levels for DDE and HE are given in Section 3.7. Instrument Class IA instrumentation is capable of performing its active nuclear safety functions during and after a DDE or HE. Instrument Class IB Category 1 instrumentation is capable of performing its active nuclear safety functions after a DDE or HE. Other Design Class I instrumentation is capable of performing the passive function of maintaining Class I pressure boundary integrity during and after a DDE or HE. In addition, some of the Design Class I instrumentation may have active seismic qualification; these instruments are identified on a case-by-case basis.

Performance criteria for Design Class I instrumentation, HVAC, and electrical equipment are as follows: (a) The reactor protection system shall be able to shut down the unit and maintain it in safe shutdown condition. (b) The electrical equipment is able to perform its required functions of providing electrical power, control, instrumentation, and protection for the ESF. (c) No device shall fail to initiate and maintain its safety function, nor shall it prevent other safety devices from performing their safety function.

The original seismic qualification of most equipment was done in accordance with IEEE 344-1971 (Reference 1) for DDE levels. As a result of changes in spectra the Design Class I equipment has been reevaluated, based on response spectra derived from the HE as well as the DDE levels discussed in Section 3.7. In the process of reevaluation, some equipment had to be requalified because: (a) its previous qualification was not adequate to envelop the HE input, or (b) concerns had been raised about the adequacy of the justification for the previous qualification methods. Requalification of the equipment, according to the guidance contained in IEEE Standard 344-1975 (Reference 2) and NRC RG 1.100 (Reference 3) was performed where necessary.

Tables 3.10-1 and 3.10-2 list instrumentation and electrical equipment that have been seismically qualified. The tables provide references to appropriate sections where qualification is described. Table 3.10-3 lists HVAC equipment that has been seismically qualified.

The seismic qualification of the equipment is based on the free-field ground motions described in Section 3.7.1. Effects of amplification of ground accelerations due to the response of the building at the location of the equipment were derived from the time-history modal superposition analyses made for the structures, as described in Section 3.7.

In addition to direct seismic effects on Design Class I equipment, PG&E has also given consideration to possible seismically induced physical interactions between nonsafety-related SSCs and Design Class I SSCs. The methodology and results of this interaction study are presented in Reference 4, and are provided in summary form in Section 3.7.3.13. Appropriate design modifications were performed where the study indicated safety functions of Design Class I equipment might be affected due to seismic interaction.

3.10.2 SEISMIC ANALYSES, TESTING PROCEDURES, AND RESTRAINT MEASURES

The effects of seismic accelerations were determined either by physical tests, mathematical analyses, or engineering judgment. Mathematical analyses of structural elements were made for Design Class I exposed electrical raceways, for equipment supports, and also for some equipment. Physical tests of equipment were made either on one of the units being supplied or on one of a similar type. Choice of method used to determine seismic capability of the equipment and devices was based on the supplier's judgment of what would be adequate and appropriate.

3.10.2.1 Nuclear Reactor Instrumentation and Protection Systems

The seismic testing of Westinghouse-supplied electrical and control equipment is documented in WCAP-8021 (Reference 5). The testing conforms to the procedures of IEEE-344-1971. The radiation monitoring cabinet and the Tracerlab scintillation detector and liquid sampler equipment are not safety-related, and these portions of WCAP-8021 are not applicable. The radiation monitoring system cabinet at DCPP has been upgraded to Design Class I (see Section 3.10.2.26). Details of the original seismic analysis and testing procedures for Design Class I instruments and electrical equipment are summarized in Table 3.10-1 and the following sections.

Typical items of equipment have been type tested under simulated seismic motion in the form of sine beats. This testing was done with conservatively large accelerations over a range of applicable frequencies and conformed to the procedures given in IEEE 344-1971. The peak test input accelerations used in those tests were checked to verify that they are larger than the requirements derived by structural analyses of DDE and HE levels. Westinghouse Electric Company, the supplier, made dynamic tests of typical samples of this equipment to confirm its seismic adequacy. Included in this test program were the racks for the nuclear instrumentation system; the process control and protection sets; the solid-state protection system cabinets and its safeguards test cabinets and auxiliary safeguards cabinets; the inverters for the power supply; pressure and differential pressure transmitters; the reactor trip switchgear; and main coolant loop resistance temperature detectors. Details of these tests are given in Table 3.10-1 and in Sections 3.10.2.1.1 to 3.10.2.1.9.

3.10.2.1.1 Nuclear Instrumentation

As described in Reference 5, a typical two-cabinet unit of the Westinghouse nuclear instrumentation system (NIS) and radiation monitoring system (RMS) has been seismically tested. The NIS equipment was contained in one cabinet and the RMS equipment was mounted in the other cabinet. The two cabinets were attached and mounted on a two-cabinet base to simulate the support or adjoining cabinets. A typical NIS installation consists of four cabinets.

The NIS cabinet contained one source range channel, one intermediate range channel, and one power range channel located and mounted in the same configuration as the plant installation. Since any DBA described in this FSAR Update can be terminated within acceptable limits by the power range channels, only the power range channel was energized and monitored. The other NIS channels mounted in the cabinet served to simulate the mass distribution and weight in an actual installation.

Shutdown procedures contain the following provisions in the event that the source range channels are rendered inoperative due to a seismic event:

(1) The operator will take appropriate action to preclude boron dilution

(2) Prior to cooldown, boric acid will be added to the reactor coolant to ensure that the concentration is sufficient to maintain the reactor in a subcritical state

During seismic testing an external test signal was applied to the equipment so that the power range output signal was 100 percent full power. The test input signals, analog output signals, and bistable output signals were monitored during test. The tripping action of the bistable amplifier circuitry was checked after each series of tests to insure that the simulated earthquake had not impaired this function.

Only one instance of mechanical malfunction occurred as a result of this level of testing. Drawer latch damage occurred during side-to-side testing at higher g levels. A new fastening mechanism has been designed and the design submitted to the NRC (Reference 6). This modification was implemented at DCPP. A demonstration test for seismic operability of the NIS equipment was performed with multiple frequency, multiple axis inputs. The test, reported in Reference 7, indicated that the equipment will operate during a seismic event as required.

The neutron detector for the NIS power range channel has been tested using sinusoidal inputs in both the horizontal and vertical directions at accelerations at least equal to those calculated for the DCPP. Neutron current measurements were made during the tests, and current, resistance and capacitance checks were made after the tests. No significant changes were found and there was no mechanical damage to the detector.

In addition, a two-section power range excore neutron detector was tested using multiple frequency, multiple axis inputs in a support assembly which simulated a detector holder. The multiple frequency inputs were developed in accordance with the guidelines set forth in Reference 8. The test response spectra envelope the DDE and HE inputs.

During the multiple frequency test, the detector was energized from a high voltage power supply, and an AC signal was imposed in each of the two signal electrodes to determine proper electrical operability.

At the completion of the tests, there was no observable mechanical damage and the electrical recordings revealed only a transient type electrical disturbance of one of the two signals. The signal perturbations were small in amplitude and would not cause any loss of protection capability of the NIS during normal operation. Subsequent detector acceptance tests performed by the detector manufacturer did not disclose any abnormal permanent change in the electrical or neutron sensitivity characteristics. Thus the NIS Power Range Detector will operate as required during and after the DCPP postulated seismic events.

3.10.2.1.1.1 Radiation Monitoring

Qualification of the radiation monitoring panels in the control room is based on shake table tests performed on the panels for Victoreen. The racks were shaketable tested with the monitoring equipment in place. See Section 3.10.2.26

3.10.2.1.2 Solid-State Protection System

The three-bay, two-train SSPS was seismically tested as described in Section 2.5 of Reference 5. During the seismic test, a typical reactor trip matrix and typical safeguards actuation matrix were energized and monitored. Before each test, the circuitry was placed in a pre-trip condition. During the actual shaking, the circuitry was deliberately tripped and changed to a post-trip condition. The functional integrity of the system was thus demonstrated by observing a satisfactory change of state on demand. Relay contact positions necessary to show the operability were recorded during the tests.

No mechanical problems occurred during the vertical axis tests. In the side-to-side axis test at lower "g" levels, the two lefthand cabinet-to-base bolts repeatedly loosened and were deformed until it became necessary to replace these with more hardened bolts. During subsequent testing at these levels in the side-to-side axis, these bolts failed completely on the last sine-beat test. Before performing the front-to-back test, twelve additional cabinet-to-base bolts were installed making a total of 24 bolts fastening the cabinet to its base. This change has been incorporated into the SSPS cabinets at DCPP.

The functions monitored by recorders were: undervoltage trip, train trouble, and SI signal. Test switches were operated during the third sine beat simulating a reactor trip and safeguards actuation, causing a change in amplitude of the recorded signals for under voltage trip and SI.

During the front-to-back axis test, the signal indicated several momentary trips (contact closures) before the test switches were operated. Also, at 7 Hz and 9.5 Hz, this signal indicated a momentary trip and then a permanent change of state (latch up) on the first bounce on the output (slave) relays. The permanent change of state was caused by the armature of the same relays bouncing closed. This closing allowed their mechanical latch mechanism to operate. These maloperations could have initiated safeguards actuation. However, they would not have negated a valid safeguard actuation or reactor trip. Although SI was prematurely actuated, the under voltage coil tripped when called upon to do so.

The duration of the momentary contact closure was probably short enough not to cause a false safety injection signal. The probability of spurious initiation of safety injection due to any earthquake is therefore very small. The seismic tests performed have demonstrated that the postulated seismic event will not prevent a legitimate safety injection signal from being actuated, either during or after the event. Reference 9 presents an analysis of the consequences of seismic-induced actuation of protection system relays by considering the possible actuation of each contact of the relays studied and describing the resulting effect of inadvertent equipment actuation.

The relays which exhibited contact bounce and mechanical latching were replaced by a new type of relay which was seismically qualified by single-axis sine-beat and multiple frequency sine-beat testing. Input levels for the tests were determined from the measured acceleration response at the cabinet during the cabinet tests described in Section 2.5 of Reference 5. Seismic qualification of these replacement relays is documented in Reference 10.

Due to obsolescence issues, the original SSPS printed circuit boards (PCBs) can be replaced with newer vintage boards supplied by Westinghouse. The replacement PCBs have been seismically qualified by Westinghouse. The seismic qualification is documented in Reference 49.

3.10.2.1.3 Process Control and Protection Equipment

Originally, seismic testing was performed using a three-cabinet unit mounted on a common base as defined in Section 2.4 of Reference 5. The three-cabinet test assembly included at least one of each type of module used in all of the various process protection and safeguards actuation channels. Both analog and bistable output signals were recorded. All reactor trips and safeguards actuation signals were continuously recorded and some bistable signals of less importance (e.g., alarms circuits) were monitored with lights. The basis for determining the functional integrity of the reactor trip and safeguards actuation signals was that these signals should remain unchanged during the test and should be capable of changing state after the test if called upon to do so.

The tripping action of the bistable amplifier circuitry was checked after each series of tests to insure that the seismic test input had not impaired this function.

During front-to-back testing of the circuit board, an internal power supply circuit board disengaged from its connector causing complete failure of the module. Restraining clamps were installed on the circuit board and the test was repeated successfully. These clamps have since been installed on all similar modules. All recorded electrical signals performed properly during and after the tests.

In addition, as part of the overall program to demonstrate the adequacy of the seismic test previously conducted, multiple frequency, multiple axis test (Reference 11) were performed on an entire typical channel, including signal conditioning circuits and the bistables, of the process instrumentation system. The results of the bistable tests show that the electrical functions of each bistable module maintained electrical operability both during and after each seismic event. In addition, no spurious bistable actions were observed.

Subsequently, the Eagle 21 system replaced the Hagan protection system within the existing racks. The Eagle 21 system has been seismically qualified on a generic basis by Westinghouse (see References 40 through 42) in accordance with requirements from References 43 and 44. A site-specific seismic analysis was also performed to ensure that the Eagle 21 generic testing performed by Westinghouse encompasses the DCPP installed condition (see Reference 45), which included the effects of the top entry conduit stiffness.

Subsequently, the Hagan process control system (PCS) in instrument racks 17 through 32 (RNO1A through RNO4E) was replaced with a programmable logic controller based system manufactured by Triconex (DDP 1000000237 and DDP 1000000501). The system chassis, I/O and hardware have been seismically qualified by Triconex (Reference 55). As part of the design change process, a site-specific seismic evaluation and seismic calculation file was completed for the installed condition.

Supporting PCS equipment, such as loop power supplies, signal isolators, circuit breakers, terminal boards and line filters that support safety related equipment, was seismically qualified in accordance References 56, 57 and 58.

3.10.2.1.4 Instrument AC Inverters

A prototype UPS and regulating transformer of the DCPP UPS system was tested as described in PG&E engineering seismic file No. ES-68-1.

The UPS and regulating transformer were tested while loaded at 20 kVA; and the ac output voltage, current and frequency were monitored during the seismic test. The presence of a continuous ac output voltage both during and after the test formed the basis for determining the functional integrity of the UPS system.

During seismic testing the static inverter maintained structural integrity and functional operability. No variation or loss of 120 Vac output voltage was observed during or after the test. Therefore, the static inverter will perform its safety related functions during and after the postulated DCPP seismic events.

3.10.2.1.5 Pressure and Differential Pressure Transmitters (Westinghouse)

Originally the safety related pressure transmitters provided by Westinghouse for DCPP were installed to sense the following conditions:

- Containment Pressure (CP)
- Reactor Coolant Level using the Reactor vessel Level Instrumentation System (RVLIS)

The transmitters were mounted during seismic qualification to a rigid fixture. The pressure and differential pressure transmitters tested are the following:

Equipment	Function Group	Manufacturer	Model No.
Differential Pressure Transmitter	CP	ITT/Barton	332/351
Differential Pressure Transmitter	RVLIS	ITT/Barton	752

As described in Section 2.8 of Reference 13, the Barton Model 332 transmitter was seismically tested. Subsequently, the containment pressure transmitters were replaced with Rosemount differential pressure transmitter Model 1154 (refer to Section 3.10.2.11 for qualification of this model transmitter). The Barton Model 351 pressure sensors are used in conjunction with the Rosemount transmitter Model 1154 to measure containment pressure.

Seismic testing of the Barton Model 752 differential pressure transmitters is detailed in WCAP-8687, Supplement 2-E04A (Reference 15). Seismic testing was performed using multiple frequency, multiple axis tests. During seismic tests, the transmitters were pressurized to approximately mid-scale with a 2,000-psig static pressure. The output of the transmitters was monitored continuously. The Barton 752 differential pressure transmitters maintain their integrity and performed their safety-related functions as required during and after seismic testing. Subsequently, the RVLIS level transmitters manufactured by Barton were replaced with Rosemount differential pressure-transmitter Model 1153 (refer to Section 3.10.2.11 for qualification of this model transmitter). The RVLIS Rosemount transmitters retain the use of the Barton Model 353 pressure sensors.

3.10.2.1.6 Reactor Trip Switchgear

Seismic testing of a typical reactor trip switchgear was performed as described in Reference 16. The basis for determining the functional integrity of the equipment was the following: (a) the breakers should trip open on loss of voltage to the undervoltage trip device during the testing sequence, and (b) all breaker outputs, including secondary contact outputs to the various protection system, should maintain proper contact condition of open or closed position.

The electrical functions of the equipment were monitored both during and after the seismic test (Reference 16) to ensure that the equipment was operating properly and performing the required safety related functions. This monitoring consisted of recording output signal voltages, and the input signal voltage to the undervoltage trip. The tripping action of each breaker through the undervoltage trip circuitry was checked during the after each series of test to verify that the simulated earthquake had not impaired this function.

The recordings of all electrical signals indicated proper and complete functioning of the equipment both during and after all testing. No secondary contact chattering, no false breaker closing and no false breaker opening was observed.

The test results show that the functions of this equipment were maintained within the established criteria, both during and after each simulated seismic condition. During seismic testing a modification kit was installed within the reactor trip switchgear to enhance its seismic capabilities. This modification kit has been installed in the DCPP reactor trip switchgear.

3.10.2.1.7 Resistance Temperature Detectors

Resistance temperature detectors (RTDs) are ruggedly built devices designed to withstand the high temperature and pressure of the fluid in the reactor coolant system. They are also designed to withstand severe seismically induced vibration, and the reactor coolant RTDs are designed to withstand the flow-induced vibration from the reactor coolant flow.

The RTDs are mounted in the reactor coolant piping, on the containment sump wall, and on rigid support structures. The reactor coolant RTDs are installed in thermowells mounted into the main coolant piping.

The seismic testing described in Reference 35 was performed on the Weed reactor coolant RTDs. The test inputs were random frequency, biaxial sine wave vibrations for a range of 1 through 1000 Hz. During the test, the RTDs were operated and their input/output signals were monitored. No mechanical damage was observed, and the input/output signals remained within acceptable limits.

The seismic testing described in Reference 36 was performed on the Conax RTDs. The test inputs were random frequency, biaxial sine wave vibrations for a range of 1 through 200 Hz. During the test, the RTDs were operated and their input/output signals were monitored. No mechanical damage was observed, and the input/output signals remained within acceptable limits.

3.10.2.1.8 Safeguards Test Cabinet

As described in Reference 19, sine-beat testing was performed on a typical engineered safeguard test cabinet. The engineered safeguards test cabinet completed seismic testing without sustaining physical damage. The only functional anomaly observed during testing was the momentary opening of the normally closed contacts of certain test selection switches at particular frequencies.

These switches are used to set up and initiate individual tests. The normally closed contacts are used exclusively to reset the blocking relays of the engineered safeguards test cabinet upon completion of a test. When the blocking circuit is not in the test mode, the blocking relay is in the reset state. A momentary opening of the normally closed contacts, therefore, will have no effect on the state of the blocking relay.

Therefore, based on the seismic testing performed, the engineered safeguard test cabinet will perform its safety related function during and after the postulated DCPP seismic events.

3.10.2.1.9 Auxiliary Safeguards Cabinet

The auxiliary safeguards cabinet is structurally identical to the safeguards test cabinet (Section 3.10.2.1.8) and the component layout of the two cabinets is essentially the same. Therefore the results obtained from the test of the safeguards test cabinet were applied to the auxiliary safeguards cabinet.

The auxiliary safeguards cabinet was later analyzed by time history analysis to qualify the use of the rotary relay in the cabinet. This analysis is described in Reference 20.

The rotary relays have been tested separately using single axis, multiple frequency inputs. These tests are described in Reference 10.

The analysis response spectra of the auxiliary safeguards cabinet, at relay mount locations, was found to be enveloped by the relay test response spectra. Therefore both the auxiliary safeguards cabinet and relays will function properly during and after the DCPP postulated seismic event.

3.10.2.2 Main Control Board and Console

The main control board is located within the control room at elevation 140 ft in the auxiliary building. It has two major structures: main control board (MCB) section and control console.

Seismic qualification of the MCB and central console is demonstrated by analysis as described in Reference 21. The analysis consisted of the following tasks:

(1) Modeling of the MCB so that its analytical frequencies correlate to those obtained from the field test

(2) Response spectrum analysis of the model using the given spectra to evaluate structural adequacy

(3) Modification of MCB to address overstress condition

(4) Response spectrum analysis of the modified MCB model to compute loads for structural evaluation

(5) Transient dynamic analysis of the modified model to generate in-equipment response spectra (IERS) for device qualification

The MCB section and central console are modeled using the general purpose finite element computer code, WECAN (Reference 22). The MCB is modeled as a linearly elastic system of beam, plate and lumped mass elements.

In addition, In-situ testing of the unmodified structure was performed to identify local panel modes. This consisted of tap tests of the vertical and bench panels as described in Reference 23.

Response spectrum analyses were performed to compute structural loads using DDE and HE required response spectra. Two dimensional shocks were considered in the evaluation. Maximum elemental stresses were obtained from the two sets of response spectrum analyses.

The structural analyses of the unmodified design indicated some overstress which is a result of the changes in spectra from the original HE loadings. To reconcile the overstressed conditions and simultaneously provide the additional advantage of increasing the overall structural frequencies to above the peak of the floor response spectra, modifications have been done on top of the main control board. These modifications lead to a much lower stress condition and increase the board's overall structural frequencies to values exceeding the peak of the input spectra.

The results of stress analyses for the modified design show that the maximum member stress is about 85 percent of allowable. A comparison of the required and as-built weldments demonstrates that the existing weldments exceed the required weldments. Buckling stability of all MCB structural members has also been evaluated. There exists, at least a factor of safety of three against buckling.

Transient dynamic analyses of the modified structural model and performed to obtain the IERS for use in qualifying board mounted devices. Two sets of transient analyses are performed using two directional seismic excitation with one horizontal and one vertical direction. The synthesized floor excitation is employed in a transient dynamic analysis using the modal superposition integration procedure. Five percent of critical damping is assumed in the analyses.

The central console is a U-shaped electrical cabinet consisting of three bolted sections, welded to the main control room floor. The structure is modeled using WECAN, similar to the main control board. Results of modal analyses of the console structural model show the lowest overall fundamental frequency to be 70 Hz.

Since the console model has no frequencies below 33 Hz, it is classified as a rigid structure, and stress evaluations are performed using the static method. Uniform static acceleration equal to the floor response spectra ZPA are applied to the console structural model. Stresses computed by the SRSS method are observed to be well below the allowables. No weldments and buckling evaluations are performed due to the obvious integrity apparent from the low stress conduit.

As all console frequencies are in the rigid range, the IERS for console mounted devices are the floor response spectra.

The IERS obtained from MCB and console analysis were used for seismically qualifying the MCB mounted devices as described in Reference 24. Qualification tests were performed to determine if the structural integrity and functional operability of the devices are maintained for the seismic level.

Devices tested were supplied by PG&E and are representative of all the Design Class I devices used in the MCB. Indicators, recorders, switches, power supply, and light box were tested. The seismic qualification was achieved by subjecting the devices to multiple frequency, multiple axis seismic testing such that the response spectrum envelops the IERS obtained from analysis. It was concluded from the tests that all of the Design Class I devices will maintain their structural integrity and functional operability during and after the postulated seismic event. In addition, three cathode ray tube (CRT) displays have been located in the central console. Seismic tests were performed to insure that these CRTs will not become missile sources.

3.10.2.3 Hot Shutdown Panel

This panel is a backup panel used if the control room must be evacuated and the plant brought to a hot shutdown condition. It contains indicators, control switches, and hand-auto stations for proportional control. These give indication and control over various pumps and valves in the auxiliary feedwater, component cooling water, boration control, and containment fan cooler systems. Additionally, the 10 percent steam dump valves are controlled from the hot shutdown panel, but the control loops for that function are not Design Class I.

3.10.2.3.1 Qualification of the Panel

The panel consists of an enclosure 5 feet 10 inches wide, 6 feet 6 inches high, and 3 feet deep, with two panels inside, one on a vertical plane, the other tilted up 30° from the horizontal. The enclosure is mounted on four channels which are welded to a box comprised of 10-inch WF beams. This box is welded to steel plates embedded in the concrete floor of the auxiliary building at elevation 100 feet.

A three-dimensional response spectra analysis has been performed on a finite element model developed for the hot shutdown control panel. This analysis has shown the subject panel is qualified for DE, DDE, and HE seismic events.

3.10.2.3.2 Qualification of Individual Instruments

The types of components in the panel that are Class 1E are indicators (Westinghouse Model VX252), control switches (Cutler-Hammer Type 10250T), and hand-auto control stations. Other devices (e.g., Westinghouse Type KA-241 indicators) are not Class 1E and have no need to meet seismic qualifications. These devices are separated from Class 1E devices by a barrier.

All required devices (VX252 indicators, W-2 switches, 10250T switches) were qualified by test using multifrequency biaxial shake tests. The devices were mounted to closely simulate their mounting conditions on the panel. The indicator was calibrated before and after the tests to ensure that the tests had not affected the calibration. During the tests, an input was applied that produced a midscale output. The outputs were monitored for fluctuation, and no fluctuation or calibration shift greater than required accuracy was noted.

Replacement valve manual/auto hand stations manufactured by NUS were seismically qualified in accordance with Reference 59, 60 and 61.

The control switches were tested in both the neutral and the "switched" positions. The contacts were monitored for chatter during the event, and were tested for proper operation afterwards. (It should be pointed out that since the safety function of these devices is to give the operator manual control of various devices, and the operator will not be expected to change switch position during the seismic event, no requirement exists to change inputs during the event.) No malfunctions of the switches were noted.

3.10.2.4 Local Instrument Panels

Local instrument panels are used as enclosures for Design Class I and non-Design Class I instrumentation throughout the plant. The panels perform no Design Class I function except to provide support for the Design Class I devices. The panels are supported at the top by fastening them to a wall or other suitable structure. The bottom is fastened to the floor or to the same structure as the top.

The panels were originally qualified by analysis performed by the panel vendor. The criteria for the panels were that they have a resonant frequency greater than 20 Hz, and that all stresses be below allowables.

Due to the large number of panels requiring qualification, a worst case analytical method was used. It was based on determination of the panel having the highest calculated stresses resulting from simultaneous horizontal and vertical seismic accelerations. Subsequent to their installation in the plant, several of the panels were modified to increase their stiffness.

The qualification of the panels is based on finite element models of several representative panels which include the effect of equipment mounted in the panel. The analysis took into account the various sizes, configurations, and locations, using an envelope of 2 percent DDE and 4 percent HE spectra.
The results of the analyses show that all panel stresses are below allowables. In addition, the panel response at Design Class I transmitter locations was derived for comparison with the test response spectra for Design Class I transmitters in the panels (see Section 3.10.2.11).

3.10.2.5 Instrument Panels PIA, PIB, and PIC

These instrument panels house various devices used to power plant transmitters and perform the necessary signal conditioning to provide alarm functions and send linear signals to indicators on the main control board. The typical parameters involved are CCW flows and heat exchanger DP, and refueling water storage tank (RWST) level, etc. Most of the components in them were originally in the power generation instrument rack (PGIR). The panels are mounted on reinforced concrete columns at about the 132-foot elevation in the cable spreading room.

The panels were originally qualified by analysis. A review of the original stress calculations indicates that nowhere would the stresses (bending or shear) due to postulated seismic loading exceed 50 percent of the yield point of the material.

The relays were qualified by comparison with the same relays installed in the ventilating control panel, physically installed nearby.

The panels were reanalyzed for the current seismic criteria for both the DDE and the HE. In addition, the seismic qualifications of the devices within the panels have been reviewed to the same seismic criteria. The analysis and the review have demonstrated that the instrument panels PIA, PIB, and PIC are seismically qualified to perform their safety function after the postulated seismic conditions.

Subsequent to the original qualification testing, replacement or additional devices to the panels have been installed. These devices are qualified by testing, analyses, or a combination of the two. Their qualification is documented in the engineering seismic files associated with the panels and/or devices.

Subsequently, design changes 1000000237 and 1000000501removed much of the instrumentation in racks PIA, PIB and PIC and transferred their functions to the Process Control System. RWST level control logic relays, CVCS Letdown Heat Exchanger Room high temperature detection/isolation control, and Aux Steam Area K high temperature detection were retained due to interface requirements that could not be incorporated into the PCS. The original Moore direct current alarms (DCAs), thermocouple transmitters (TCTs), square root transmitters (SRTs) and signal conditioners (SCTs) were removed or replaced with new Moore (CPT) PC-programmable temperature transmitter and signal isolators/converters. The Moore CPTs were seismically qualified by NLI per references 56, 57 and 58.

3.10.2.6 Diesel Generator Excitation Cubicle and Control Cabinet

The diesel generator (DG) excitation cubicle and the control cabinet for DGs 1-1, 1-2, 1-3, 2-1, and 2-2, were originally seismically qualified for the DDE by the manufacturer. Subsequently, one excitation cubicle and one control cabinet were shake-table tested and qualified to the 1977 HE requirements.

The seismic qualification has been reviewed to the latest DDE and HE levels described in Section 3.7. Based on this review, it has been demonstrated that both the excitation cubicle and the control cabinet will perform their safety function during and after the specified seismic conditions.

The sixth excitation cubicle and control cabinet for DG 2-3 have been seismically qualified by shake-table testing.

3.10.2.7 Design Class I AC Electrical Distribution Equipment

The following sections describe the seismic qualification of Class 1E ac electrical distribution equipment.

3.10.2.7.1 4160 V Metal-Clad Switchgear

The original 4160 V metal-clad switchgear with General Electric (GE) 250 mVA 4.16 kV magneblast circuit breakers was seismically qualified by a combination of testing and analyses.

Later, it was discovered that 350 mVA circuit breakers should be used in place of the GE 250 mVA 4.16 kV magneblast circuit breakers. GE could not supply such breakers to the same switchgear. Consequently, PG&E decided to procure 350 mVA 4.16 kV breakers from NTS/PDS, which converted Japanese-made Yaskawa SF6 circuit breakers to fit the existing 4 kV switchgear. The new circuit breakers were installed during refueling outages 1R8 and 2R7.

New circuit breakers were seismically qualified by shake table testing (NTS report No. TR60431-95N-FR). The shake table testing was intended to achieve the following objectives.

(1) Demonstrate the structural integrity and functionality of the Yaskawa breakers.

(2) Demonstrate the structural integrity of as-installed 4 kV switchgear cubicles at DCPP with the Yaskawa breakers.

(3) Demonstrate the functional performance of the existing components (i.e., various relays and switches) installed in the existing 4 kV switchgear cubicles with replacement Yaskawa breakers.

(4) Instrument the test 4 kV switchgear cubicles with sufficient number of accelerometers to obtain accurate information on the dynamic response (response frequencies, test response spectra) at various cubicle locations. This information is to be used for further/future testing and analyses.

(5) Take immediate corrective actions to address significant anomalies observed during the test.

The initial seismic testing was performed at Wyle Laboratories in Huntsville, Alabama. Three seismic mock-up 4 kV switchgear cubicles were built to duplicate the design, material, and construction of cubicles G-5, G-12, and G-13 of Unit 1. A total of 18 OBE and SSE test runs were performed, including three runs of resonance search. Test results showed that the new breakers and mock-up cubicles successfully passed the minimum required 5 OBE tests.

For the SSE tests performed at Wyle Laboratories, excessive relay chatter at certain frequencies were noted. The excessive chatter was due to over-testing the equipment, which in turn was a result of Wyle Laboratories being unable to accurately control the test table response at 10 Hz and above due to resonance of the table. The over-test produced a significant amount of relay chatter, which caused the tripping and closing of breakers. The post test functional check showed that the breakers were functioning properly and had no structural damage.

To properly test the relays, supplemental SSE testing was performed at Farwell and Hendricks (F&H) Laboratories. The upper front doors of the G-12 and G-13 cubicles, where a majority of relays are mounted, were mounted on the F&H rigid test fixture. One 1200A breaker and one 2000A breaker, located adjacent to the test table, were fed by the relays. The SSE RRS obtained at relay locations on the G-12 and G-13 cubicles from the previous Wyle testing were reduced with the appropriate scaling factor to eliminate unnecessary over-testing. The supplemental SSE testing was successful. However, certain modifications (such as adding chokes to the breakers and removing the seal-ins from certain relays) were made when the new breakers were installed in the 4-kV switchgear.

Based on the above, the switchgear and its contents are qualified for the DE, DDE, Hosgri, and LTSP postulated seismic events at DCPP.

3.10.2.7.2 Potential Transformers 4160/120 V

There are a total of four potential transformers associated with each 4 kV vital switchgear. There is a potential transformer for each feeder; auxiliary, startup and the diesel generator and one for the bus itself. Potential transformers are normally an integral mechanical and electrical part of metal clad switchgear. However, the auxiliary and startup potential transformers were removed from the top of the 4160 V metal clad switchgear during the initial qualification testing performed in 1978.
In 1978, a potential transformer representing the auxiliary and startup potential transformer was separately shake-table tested and qualified for the Hosgri earthquake. A potential transformer representing the diesel generator and bus potential transformer was shake-table tested using a mock-up of cubicle H-7.

In 1995, a potential transformer was mounted above a mock-up of cubicle G-12 and a dummy weight, representing the weight of a potential transformer, was mounted above a mock-up of cubicle G-5. These two cubicles were included in the shake-table testing at Wyle Laboratories during the seismic qualification of the new SF6 breakers described in section 3.10.2.7.1.

The auxiliary and startup potential transformers that were originally at the 90-inch level of the vital switchgear have been relocated to rigid stands adjacent to the respective switchgear lineups. Electrically, they are still an integral part of the switchgear. The diesel generator and bus potential transformers located below the 90-inch level are still physically attached to the switchgear, with the exception of the diesel generator potential transformer on Unit 1 Bus F that was also moved to a rigid stand adjacent to the switchgear.

The seismic qualification has been reviewed for the latest DDE and HE levels. The comparison of the earlier test data with current seismic requirements and additional analysis demonstrate that the potential transformers are qualified to perform their safety function during and after the specified seismic conditions.

3.10.2.7.3 Safeguard Relay Boards

Originally, one safeguard relay board from Unit 1 was shake-table tested to qualify the relay boards for the seismic requirements of the DDE. New qualified relays were introduced to replace the ones that exhibited chatter. Relays whose chatter did not impair any safety function were not replaced. As a result of the 1978 HE reevaluation, one relay board of the six installed was shake-table tested to higher levels than the original test.

The qualification of the relay boards has been reviewed to the latest DDE and HE levels. Based on this review, it is concluded that the safeguard relay boards are seismically qualified to perform their safety function.

3.10.2.7.4 Vital 480 V Load Centers

The vital 480 V load centers were originally qualified for the DDE based on seismic tests on similar equipment conducted by the manufacturer. As a result of the 1978 HE reevaluation, the equipment was further qualified by shake-table testing. During the testing, the draw-out modules of the load center were equipped with hold-down brackets to prevent slamming of the modules and subsequent chatter of contacts. Chatter was detected also on the deenergized high-speed contactors of the Unit 2 fan cooler controllers while the low-speed contactor was energized.

As a result, all draw-out modules of the 480 V vital load centers have been equipped with hold-down brackets. The containment fan cooler motor controllers have been equipped with mechanical interlocks that prevent inadvertent closure of the deenergized high-speed contactor when the low-speed contactor is energized, and likewise prevent closure of the low-speed contactor when the high-speed is energized.

The load centers were qualified using a certain type of kickout spring. Subsequently, all load center contactors in question were checked in the field and the proper kickout springs (the one used in the qualification testing) were installed where necessary.

The qualification of the load centers has been reviewed to new 1983 seismic criteria for both the DDE and the postulated HE event described in Section 3.7. The comparison of the earlier test data with current seismic requirements and additional analysis demonstrates that the vital 480 V load centers are seismically qualified to perform their safety function during and after the specified seismic conditions. The qualification meets the requirements of IEEE 344-1975 and RG 1.100.

3.10.2.7.5 Vital Load Center Transformer

The vital load center transformers were originally seismically qualified for the DDE based on shake-table testing of a similar, but larger power transformer. Comparison was made which demonstrated that the test of the 1500-kVA transformers is applicable to qualify the 1000-kVA vital load center transformers installed.

The test results were further reviewed with regard to the 1977 requirements of the HE. It was found then that the earlier testing still qualified the transformer for the HE levels.

A further review of the original test with regard to current seismic requirements for both the DDE and HE concluded that the vital load center transformers are qualified for the above seismic criteria.

3.10.2.7.6 480 V Vital Load Center Auxiliary Relay Panels

The vital load center auxiliary relay panels were originally designed and constructed to meet DDE seismic requirements.

To qualify the panels and electrical components for the 1978 HE requirements, two typical panels were shake-table tested. This requalified all relay panels.

The qualification of the relay panels has been reviewed to current seismic criteria for both the DDE and the HE. The comparison of the earlier test data with the current seismic requirements and additional analysis demonstrates that the vital load center auxiliary relay panels are qualified to perform their safety function.

3.10.2.7.7 Instrument Power AC Panelboards

The instrument power ac panelboards were originally qualified for the DDE by seismic testing which includes multifrequency sine beat test and resonant frequency test. Subsequently, the equipment was requalified to the 1978 HE requirements based on comparison test data from the DDE tests above and shake-table testing for circuit breakers/panelboards of various manufacturers.

The qualification has been reevaluated to the current seismic requirements for both the DDE and the HE. The reevaluation includes the panelboard/component qualification by calculation and by comparison of 1978 shake-table test results to the current seismic criteria. The calculation verifies the equipment structure integrity, and the test results demonstrate that the current seismic criteria are met.

The reevaluation concludes that the instrument power ac panelboards remain seismically qualified.

3.10.2.8 Design Class I DC Electrical Equipment

The following subsections describe the seismic qualification of Class 1E dc electrical equipment.

3.10.2.8.1 Batteries

There are six vital "batteries" at DCPP, three in each unit. Each "battery" consists of 60 battery cells (see Section 8.3.2.3.6.3 for 59-cell configuration). The original Class 1E station batteries were C&D Model LCU-27. They were replaced in 1983 with C&D Model LC-25 battery cells. Currently, all vital batteries are C&D Model LCUN-33 cells.

Each vital battery (60 cells) is located in its own room in the auxiliary building at elevation 115 feet. In each battery room there are currently four battery racks; three are single-tier racks that hold 12 LCUN-33 battery cells each, and the fourth is a two-step rack that holds 24 of the cells.

The new battery cells (Model LCUN-33) were tested at Wyle Labs in Huntsville, Alabama. A rigid test rack was utilized. The test rack held four LCUN-33 cells. The new battery cells have been qualified by shake table testing for DE, DDE, and Hosgri design basis seismic events. They have also been evaluated for the LTSP requirements and were found to satisfy the LTSP acceptance criteria.

Two separate LCUN-33 tests were performed. First, unaged cells were tested to qualify the cells for installation in outage 1R5 and for a 5-year life. Next, another group of LCUN-33 cells were artificially aged and tested. The second test qualified the LCUN-33 cells for a 15-year life. The qualification was performed using the guidance of IEEE 535-1986 (Reference 51) and NRC Regulatory Guide 1.158 (Reference 52). The qualified life of the C&D LCUN-33 battery was extended to achieve a 20 year mean service life in 2006. Qualified life is determined per IEEE 535-1986 and is documented in the seismic calculation file (Reference 53).

Preventive maintenance is in place to replace the batteries before the expiration of their qualified life.

It is concluded that the station batteries C&D Model LCUN-33 will perform their safety function during and after DCPP design basis seismic events.

3.10.2.8.2 Station Battery Racks

Each vital battery (60 cells – see Section 8.3.2.2.1.3 for 59-cell configuration) is located in its own room in the auxiliary building at elevation 115 feet. In each battery room, there are currently four battery racks. Originally, all battery racks were single-tier racks. Each rack held 15 Model LC-25 cells. Since the new LCUN-33 battery cells are wider than the old cells, the existing single-tier rack would hold only 12 of the new cells. Accordingly, one of the existing single-tier racks was replaced with a new two-step rack that will hold 24 of the new battery cells. Currently, three of the existing racks are single-tier racks that hold 12 LCUN-33 battery cells each. The fourth is a two-step rack that holds 24 of the cells.

The original single-tier racks were supplied by C&D along with the original LCU-27 battery cells. The racks have since been modified by PG&E due to the 1978 Hosgri reevaluation, new stress analysis for the current DDE and Hosgri levels, and finally as a result of replacing the battery cells with the new Model LCUN-33.

Both the single-tier and the two-step racks have been seismically qualified by analysis and are qualified for DCPP design basis seismic events.

3.10.2.8.3 Battery Chargers

Originally, the battery chargers were seismically qualified for the DDE by dynamic testing. As a result of the 1978 HE reevaluation, one of the battery chargers was shake-table tested to qualify all chargers for the HE requirements.

Further shake table testing has been done on one of the battery chargers to current seismic requirement. The testing demonstrated that the battery charger will perform its safety function during and after the postulated seismic events. The testing qualifies all Class 1E battery chargers for both DDE and HE.

3.10.2.8.4 125 V DC Switchgear

The 125 Vdc switchgear was originally qualified for the DDE based on seismic tests performed on similar equipment conducted by the manufacturer.

As part of the 1978 HE reevaluation, a 125 Vdc switchgear from DCPP was shake-table tested. This switchgear is identical to the other five 125 Vdc vital switchgears installed in the plant. This test demonstrated the adequacy of the equipment's safety function during and after the postulated seismic condition.

The qualification of the dc switchgears has been re-reviewed to current seismic requirements for both the DDE and HE. The review confirms the adequacy of the qualification by comparison of the 1978 equipment test data to the current criteria. Based on the review, it is concluded that the 125 Vdc switchgears are qualified to perform their function during and after the DDE and the HE.

3.10.2.8.5 Motor Controller, 125 Vdc, for Valve FCV 95

The 125 Vdc motor controllers were installed at Units 1 and 2 in 1982. They have been seismically qualified to the seismic requirements of both the DDE and HE described in Section 3.7 by shake-table testing of one unit. The controller met all the test requirements; therefore, it is concluded that the motor controllers will perform their safety function during and after the specified seismic events.

3.10.2.9 Main Annunciator

Originally, the main annunciator cabinets were seismically qualified for the DDE by dynamic analysis. Visual annunciator components similar to those mounted in the cabinets were tested in operation by the supplier. The cabinets were reanalyzed again for the 1978 HE reevaluation. As a result, some bracing was added inside the cabinets. Components of the visual annunciator were shake-table tested and qualified for the HE requirements.

The seismic qualification of the annunciator cabinets and the visual annunciator components have been reviewed to the current requirements of both the DDE and HE. As a result of this review, the annunciator cabinets were further stiffened, particularly in the longitudinal axis. The components were found to meet the new seismic requirements.

The Sequence of Events Recorder (SER) components including the printer have been qualified by shake-table testing. The SER Cathode Ray Tube display and its microprocessor were shake-table tested to ensure they would not become a missile hazard but are not required to operate during and after a seismic event.

Subsequently, due to problems obtaining parts for the seismically-qualified printer, PG&E replaced the printer. The printer was replaced with a seismically qualified touch screen and a computer (PC) interfacing with a nonseismically qualified desktop printer. The PC and touch screen were qualified by testing. After a seismic event, the operators will be able to view the alarms on the touch screen. Any compatible desktop printer can be obtained and used to print the data.

The main annunciator communicates via data link to remote multiplexers and visual annunciator drivers associated with the main generator, which are not seismically qualified. There is no failure mechanism of the data link, remote multiplexer, or remote visual annunciator drivers that can adversely impact the function of the main annunciator system following an earthquake. The main generator alarms provided by the multiplexers are Design Class II and are not needed to maintain the plant in a safe shutdown condition or to mitigate the consequences of seismic events (Reference 46).

The main annunciator system is considered to be important to plant operation. However, the main annunciator system is not required for safe shutdown of the reactor. To achieve high reliability, the main annunciator system is designed to remain functional during and after a Hosgri earthquake, and to operate during a momentary or extended loss of offsite power. To meet these performance goals, the main annunciator system was originally classified as a Class I system. However, since the main annunciator system is not designed to meet the single failure criterion, the system was reclassified to Class II.

3.10.2.10 Electrical Penetrations

Electrical penetrations of the containment structure must withstand the forces caused by a LOCA. The header plates are made of forged steel welded to the containment steel liner and therefore have considerably more strength than is needed to meet seismic conditions. The penetrations are approximately 5 feet long and contain insulated electrical conductors of stranded copper. These conductors are supported within the penetration and at the terminal boxes attached to each end of the penetration.

The electrical penetrations were originally seismically qualified for the DDE by static analysis, meeting the requirements of paragraph 3.1.3 of IEEE 344-1971. A further seismic analysis was made for penetration of similar configurations for the Pilgrim 1 and Fitzpatrick 1 units. This analysis was used to qualify the penetration for the 1978 HE reevaluation.

Seismic testing performed by the manufacturer and a new analysis was used to qualify the penetrations to current requirements of the DDE and HE. The analysis demonstrates that the electrical penetrations will perform their safety function during and after the specified seismic conditions.

3.10.2.11 Pressure and Differential Pressure Transmitters

Seismic tests were performed on Barton Model 763 and 764 transmitters and Rosemount Model 1151, 1152, 1153, and 1154 transmitters as part of an environmental test programs conducted by their respective manufacturers. The transmitters were subjected to simultaneous independent biaxial excitation using a random test input. The test included a resonant search, 5-DEs, and 1-DDE, in each of two test positions.

The transmitters were pressurized and operational throughout the test. The output of each transmitter was monitored during the test to verify proper operation. The results of the test verified that the transmitters will operate properly for both DDE and HE excitation.

The seismic qualification of Rosemount Model 1154 transmitters is based on similarity between Model 1154 and Model 1153 Series D transmitters (Reference 37, paragraph 7.2). As documented in References 38 and 39, the Model 1153 Series D transmitters were shake table tested per IEEE 344-1975 to DCPP seismic requirements.

During the seismic testing leak test, calibration check and voltage variation tests were performed.

All tested transmitters successfully met the acceptance criteria. Anomalies observed were determined not to have an impact on the transmitters' qualification (Reference page v in Wyle Test Report No. 45592-3 -- Appendix A of Reference 38). The test response spectra curves enveloped the applicable DCPP-required response spectra curves (Reference 39).

Seismic testing and analysis (Reference 50) was performed to qualify Rosemount 'Smart' transmitter model 3051C for use in Instrument Class IC Systems as defined in Section 3.2.2.5 (3). The seismic testing and analysis in Reference 50 also qualifies the use of Rosemount 'Smart' transmitter model 3051N for use in Instrument Class IA Systems as defined in Section 3.2.2.5 (1).

Seismic tests were also performed on two typical models of Barton transmitters, Models 368 and 369, pressurized to mid-range operation. The tests were performed at Wyle Laboratories on a biaxial shaking table. Results show that the requirements were met. The transmitters operated throughout the tests without malfunctioning. A helium leak test was made on the pressure boundary of the transmitter after the seismic tests. No leakage was detected. The instruments were qualified in conformance to the requirements of IEEE 344-1971, Paragraph 3, Method 2, simulated seismic test.

Additional seismic qualification tests have been performed on the Barton Model 332 pressure transmitter. These were part of a series of tests, documented in WCAP-8021, where selected types of safety-related essential equipment were subjected to vibration tests in the range of 1 to 35 Hz.

A preliminary search of the 1 to 35 Hz frequency range, using a sinusoidal input, was performed to identify any resonant condition. Any resonant frequencies found would be included with the test frequencies of the sine beat seismic input. The amplitude of the sine beat was chosen such that it would be at least as great as the maximum acceleration that the equipment would experience during a DE horizontal ground acceleration of 0.4 g, augmented by building structural amplification. Tests were done independently for each of the two horizontal and the one vertical directions of motion. Throughout the duration of the testing, both the test and reference transmitters were energized, measuring a 50 psi input pressure on a 300 psi span. The 4 to 20 mA electric output of the transmitter was monitored during and after the test to check for any loss of function.

Based on the results of these tests, it is concluded that this transmitter will perform its required design function during, as well as following, a seismic event.

3.10.2.12 Raceway Supports

The Class 1E raceway systems (safety-related) consist of conduits, cable trays, and pull boxes supported by approximately 27,000 supports in each unit. The raceway supports are constructed primarily of bolted assemblies of cold-formed channel sections either of "Superstrut" (more than 90 percent) or "Unistrut" (approximately 10 percent) brand, which are spaced at 8-1/2 feet or less, unless otherwise approved by an engineering evaluation. The supports are attached to concrete or structure steel by bolted connections or welding. Based on the similarity of structural configuration, the raceway supports are grouped into more than 400 generic types.

3.10.2.12.1 Design and Acceptance Criteria

The raceway supports are required to withstand loads from DDE or HE. The supports, in their as-built conditions, are evaluated to ensure that they meet the following criteria.

Loading Combination

The horizontal component of seismic load (DDE or HE) either transverse or longitudinal to the raceways that results in the highest stress on the member under consideration is combined, by absolute sum, with the stresses or forces due to dead load and vertical seismic load.

Response Acceleration of Support System

Unless otherwise justified the floor response spectra where supports are located are used for evaluation. The horizontal response is taken as the greater of the building responses due to either the east-west or the north-south ground motion combined by absolute sum, with the corresponding torsional response, as appropriate.

Acceptance Criteria

The specifications used to review the design of the steel members are the AISI "Specification for Design of Cold Formed Steel Structural Members" (Reference 32, Section 3.10.3) and Part 1 of the AISC "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings" (Reference 33, Section 3.10.3) applicable to hot-rolled members. The allowable stress given in the AISC specification is increased by 60 percent as recommended by Standard Review Plan Section 3.8.4 (Reference 34, Section 3.10.2.12.3). The allowable stress for AISI is increased so that the margin against yielding is 1.0 or greater with the allowance for local yielding at connections. The allowable slip-shear capacity of bolted connections on strut members are established by statically testing support connections with various combination of nuts and bolt torque values. The design allowable is based on the support connection containing the type of nut (98 percent of the nuts actually used in the plant exhibit superior behavior) and the bolt torque value (more than 90 percent of the connections have significantly higher values) which very conservatively represent the as-built condition. In addition, a qualification criterion was established by performing dynamic tests on specimens having representative support configuration in determining acceptable shear capacity of the in-situ bolts. In some cases, the slip-shear capacities are based on manufacturer's recommended values. These shear capacities are for appropriate combination of nut types, bolt torque, and strut which have been verified by additional tests.

Permissible loads on conduit clamps are kept below 90 percent of the ultimate values. Clamps are also checked for interaction of pull-out and slip (either in transverse or longitudinal direction). The acceptance limit on fillet welds on cold-formed steel members is 60 percent greater than the allowable given in Section 4.2.1 of the AISI Specification. Spot-welds in composite superstrut channels are checked against allowable shear values developed from a testing program.

3.10.2.12.2 Evaluation

The electrical raceway systems are evaluated for seismic loading in the transverse, longitudinal, and vertical directions by following the methodology stated below.

Transverse Seismic Analysis

Each of the support types are evaluated against the acceptance criteria. The seismic loads used in the evaluation of the supports are based on system frequency of the support and adjacent span of the raceway. The damping value used for conduit supports is 7 percent. For cable tray supports, two separate frequency analyses are made to determine the spectral response. In the first analysis, the system frequency is obtained based on support frequency alone and 7 percent damping is used. In the second analysis, the system frequency is generated by combining the support frequency and the tray frequency. The seismic response is obtained based on

15 percent damped floor spectra. The second analysis is confirmatory and not a basis for the license. The larger of the two spectral values is used for evaluation.

Each support type is first evaluated for the generic case based on design, which results in maximum support response.
Any support that cannot be qualified for its generic case is investigated for its as-built condition.

Longitudinal Seismic Analysis

All Class 1E raceway systems are walked down and documented. The longitudinal seismic load is generated based on raceway system frequency and 7 percent damping. The peak response acceleration is used if the system frequency is less than 33 Hz; otherwise, the zero period acceleration is used. The seismic load is distributed among the supports in proportion to their longitudinal stiffness. The individual supports are evaluated for structural adequacy.

Vertical Seismic Analysis

The vertical seismic analysis uses the same methodology as the transverse seismic analysis.

3.10.2.13 Fire Pump Controller

The fire pump controllers for the plant interior system were upgraded to Class 1E when the fire protection system was upgraded. One controller was shake-table tested and qualified to 1978 HE seismic requirements.

The qualification has been reviewed to the current seismic criteria for both the DDE and HE. The comparison of earlier test data with the current seismic requirements demonstrates that the fire pump controllers are seismically qualified to perform their safety function during and after the specified seismic conditions.

3.10.2.14 Local Starters

Local starters were originally seismically qualified for the DDE based on shake-table testing by the manufacturer. As a result of the 1978 HE reevaluation, three representative starters were shake-table tested. The testing qualified all local starters for their respective locations. Other starters of different manufacture used for the HVAC systems were qualified by comparison to the starters tested. The qualification of the local starters has been reviewed to current seismic criteria for both the DDE and HE.

For the review of the qualification to current seismic requirements, the local starters were broken down into four groups:

(1) Starters located in the auxiliary and fuel handling buildings were qualified by comparison of the 1978 test data to the current seismic criteria. This includes contactors installed since the 1978 shake-table testing.

(2) Starters located at the turbine building 119-foot elevation were qualified by comparison of the 1978 test data to the current seismic criteria and by comparison to identical starters shake-table tested for the turbine building 140-foot elevation.

(3) Starters located at the turbine building 140-foot elevation were installed after the 1978 electrical equipment testing program. One of these starters was shake-table tested to qualify the starters for the current seismic criteria.

(4) Starters located at the auxiliary building 154-foot elevation, some of which are of different manufacture, have been qualified by comparison to the starters tested. The starters tested were qualified to spectra with much higher accelerations than are required for the 154-foot elevation of the auxiliary building.

The aforementioned review of the earlier seismic testing to current criteria for both the DDE and HE and additional testing demonstrates that all local starters are qualified to perform their safety function during and after the specified seismic conditions.

3.10.2.15 Ventilation Control Logic and Relay Cabinet

The ventilation control logic and relay cabinets were originally vibration-tested in July 1973, and seismically qualified for the DDE. The cabinets were found to be rigid. As part of the 1978 HE reevaluation, components of the cabinets were shake-table tested again and qualified for the HE.

The seismic qualification of the ventilating control logic and relay cabinet and their components has been reviewed to the current seismic requirements of both the DDE and the HE. Based on the review, it has been concluded that the ventilation control logic and relay cabinets are seismically qualified to perform their safety function during and after the DDE and the HE.

As part of the Unit 1 and 2 AFHBVS control system replacement, a new programmable logic controller (PLC) system was installed. The seismic qualification of the Plant Operating Vent panels, POV1 and POV2, and their components were reviewed to the current seismic requirements of both the DDE and the HE. The POV panels were evaluated using analytical evaluation and the PLC was shake table tested by the supplier. Based on the review, it was concluded that the POV1 and POV2 cabinets are seismically qualified to perform their intended safety function during and after the DCPP design basis seismic events.

3.10.2.16 Fan Cooler Motors

The fan cooler units were qualified for seismic adequacy by a combination of analysis and testing. A seismic analysis of the fan cooler unit, including the motor, was made to verify that the units will not exceed the allowable stresses or deflections. The response spectrum method of analysis was used.

The fan motor assembly natural frequencies were calculated using a lumped mass model. Because of high natural frequencies (after adding stiffeners to the fan cooler assemblies), this system was analyzed as a rigid structure and equivalent static loads were applied. An additional unbalanced load of 1 g was assumed to occur in all rotating assemblies.

Limit values were in accordance with the elastic provisions of the AISC-69 specification. Bearing limits were taken as failure by brinelling under dynamic load (basic rating) from the manufacturer's catalog.

An analysis and an impact test were also made on an end bell of the motor. Based on these results, Westinghouse concluded that the containment fan-motor-cooler assembly structure could withstand the combination of required loads.

3.10.2.17 Pump Motors

Electric motors for Design Class I pumps were procured with the pumps to equipment specifications that covered the pump/motor assembly as a unit. These equipment specifications required that the equipment be adequately designed to accommodate seismic accelerations appropriate for the DE and DDE.

At the time of procurement of this equipment, there were no industry standards for seismic qualification of electric motors. However, methods and criteria employed in the design of large, integral horsepower electric motors lead to motor designs that are inherently capable of tolerating high seismic loadings without loss of function. Design considerations for motors of this type include maximum torque, critical shaft speed, bearing life, vibration, and cyclical loading. These considerations lead to motor designs that would not be governed by the application of seismic loads in the range of those appropriate for DCPP. Experience with such motors in applications subject to severe vibration and shock provides additional confirmation of seismic adequacy. Based on these considerations, it is the consensus of competent engineering practice that these motors are adequately designed to perform their safety function before, during, and after the DDE or HE.

In the case of the auxiliary saltwater pumps, which are vertically mounted, calculations indicated that seismic bracing in the horizontal direction was necessary to ensure that the first vibrational mode of the pump-and-motor assembly would be in the rigid range of the design spectra. Other pump-and-motor assemblies have natural frequencies well within the rigid range.

Analyses of pump-and-motor assemblies representative of those considered here have been performed and have shown substantial margin in stresses, deflections, and bearing loads for seismic loadings in the range of those appropriate for DCPP. Selected pump-and-motor assemblies for DCPP have also been analyzed using the appropriate floor response spectra and both static and dynamic analysis methods. For all cases analyzed, seismic adequacy has been verified.

3.10.2.18 Electric Cables

Electric cables interconnecting pieces of equipment depend on raceways for support during seismic activity. Seismic criteria for raceway supports are described in Section 3.10.2.12. These cables are flexible and are fully supported along their entire length in conduit or tray. Adequate slack is provided to impose little or no tension on the wires. Where relative shifts between structures can occur, raceways are provided with flexible joints or are routed with adequate flexibility to ensure that the conductors remain undamaged and their associated supports meet their acceptance criteria. (Note that use of trays for Class 1E circuits is very limited, see Chapter 8.)

3.10.2.19 Motor-Operated Valves

PG&E-purchased motor-operated valves (MOVs) whose only safety function is to maintain a pressure boundary and which are not required to change position during or after an accident (passive valves), and those whose safety function includes both maintaining a pressure boundary and changing position during or after an accident (active valves), were qualified to acceleration levels less than or equal to allowable acceleration levels provided by the vendor or established by analysis. All Westinghouse-purchased MOVs were qualified to acceleration levels less than or equal to allowable acceleration levels provided by Westinghouse.

Limitorque MOV operators of the type installed on active Design Class I valves in DCPP Units 1 and 2 have been seismically qualified by test. Tests were conducted on various sized operators at g-levels from 3 to 10 g.

Operators were tested both while operating and while energized and not operating. The testing was single axis, performed along each of three mutually perpendicular major axes. Sine sweep testing was utilized over a range from 5 to 35 Hz, and was followed by 1- to 2-minute sinusoidal vibration at 34 Hz or at any resonant frequency below 35 Hz.

The test results for the MOV operators have been considered in the analysis of piping systems. The mass and resonant frequency of the operator is used as input in the piping analysis. The resulting acceleration levels are compared to the allowable levels to verify that the operator will function as required.

3.10.2.20 Control Room Ventilation System

The control relay and power panels are seismically qualified to the current seismic requirements for both the DDE and the HE. The qualification of the power panels is based on: shake-table testing on the similar equipment performed by the manufacturer, onsite resonance frequency test on the similar panelboard, extensive shake-table testing of electro-mechanical equipment containing circuit breakers or circuit breaker panelboards and testing of circuit breakers to the extent of the shake-table limit.

The qualification of the control relay panels is based on the actual shake-table testing of similar ventilation control relay cabinet installed in DCPP, containing the same control relays. Timing relays not found in this shake-test are seismically qualified based on the qualification of 4-kV switchgear equipment in which the identical timing relays were installed.

Test specimens similar to the installed equipment were evaluated for their adequacy of qualification. By comparison of the test data from those shake-table tests to the current seismic requirements, the test results of these equipment demonstrate that the specified seismic criteria are met for both the DDE and HE. It should be noted that the equipment is only needed for the control room ventilation and pressurization (CRVP) after an earthquake and not during the earthquake. Therefore it is concluded that the CRVP control relay and power panels are qualified to perform their safety function after the postulated seismic events such as the DDE and the HE.

The radiation and chlorine monitoring panel has been qualified by seismic simulation testing of a test specimen similar to the panel installed at the DCPP. The panel test specimen was welded to the test table and bolted to an adjacent structure in order to simulate the actual plant mounting conditions. The panel test specimen contained the following instruments: one radiation rate readout (Nuclear Measurements Corporation Model GA-2TMO), one radiation rate readout (Technical Associates Model FML-554), one chlorine analyzer (Capital Controls Model 1030), and one switch reset module. Dummy weights were used to simulate random biaxial seismic simulation tests in accordance with IEEE std. 344-1975. The function of the instruments was verified before and after the testing. The test response spectra have been verified to envelop the DDE and HE required response spectra. The chlorine detection function has been eliminated and the detectors are abandoned in place.

The radiation detector associated with the Nuclear Measurements radiation rate readout described above was also subjected to random biaxial seismic simulation tests. The test specimen was bolted to a rigid steel plate in order to simulate the mounting arrangement used at DCPP. The function of the device was verified before and after the test. The test table response spectra have been verified to envelop the DDE and HE required response spectra.

The radiation detector associated with the Technical Associates radiation readout and the chlorine detector associated with the Capitol Controls chlorine analyzer have also been submitted to random biaxial seismic simulation tests. These two detectors were mounted into a 14-inch steel duct in order to simulate the mounting arrangement used at the DCPP. The function of the detectors was verified before and after the tests. The test response spectra envelops the required response spectra for both the DDE and HE cases. The chlorine detection function has been eliminated and the detectors are abandoned in place.

3.10.2.21 Subcooled Margin Monitors

The subcooled margin monitors (SCMMs) are located in PAM Panels 3 and 4. Subcooled margin is calculated and displayed by the reactor vessel level instrumentation system (RVLIS); therefore, seismic qualification of each SCMM is covered by the seismic qualification of the RVLIS cabinets (see Section 3.10.2.32.1).

Train A of the SCCM provides output to a recorder on PAM1. PAM1 seismic qualification is addressed in Section 3.10.2.22.

Train B of the SCCM provides output to a display on VB2. This display was qualified by testing.

3.10.2.22 Postaccident Monitoring Panels PAM1 and PAM2

These panels are located in the main control room and house various indicators and recorders used for postaccident monitoring. Typical parameters involved are reactor vessel level, containment hydrogen gas concentration, containment gross activity, etc. The panels were manufactured for PG&E by Trayer Engineering, Inc. Design Class I indicators and recorders were manufactured by Westinghouse and other qualified suppliers.

Panel PAM1, with its associated instruments, was qualified by random biaxial seismic simulation tests. The tests were performed at Wyle Laboratories. The function of the devices was verified before and after seismic testing. The test response spectra have been verified to envelop the DDE and HE required response spectra.

Panel PAM2 has been qualified by an analysis that showed that the panel is rigid, with no resonant frequencies below 33 Hz. In addition, a static analysis was performed that showed that the combined seismic stresses do not exceed the allowable limits. The Design Class I instruments mounted in PAM2 have been qualified by seismic simulation tests. The test response spectra obtained from these random biaxial tests have been verified to envelop the DDE and HE spectra.

3.10.2.23 Pilot Solenoid Valves

Pilot solenoid valves are used to control the air supply to air-operated control valves. The pilot valves can be mounted either on or off the control valve actuator. The required acceleration level for the pilot valves is 9 g's, which is based on the maximum allowable response for control valve actuators having Design Class I pilot valves.

The pilot solenoid valves have been qualified by tests performed on a variety of valve models by both the vendor, ASCO, and PG&E. The tests consisted typically of sine beat tests performed over the frequency range of 1 to 33 Hz. The minimum acceleration value met or exceeded the required level for the valves except at low frequencies, where the level was limited by testing machine capabilities. The function of the valves was verified before and after the testing.

3.10.2.24 Process Solenoid Valves

Design Class I process solenoid valves are used as containment isolation valves in the post-LOCA sampling system and containment hydrogen monitoring system. The valves were manufactured by Valcor Engineering.

The valves are located both inside and outside of containment. The valves inside containment are mounted to the annulus steel structure. The valves outside of containment are mounted to the exterior of the containment structure. The required acceleration level for the valves is an envelope of the DDE- and HE-required response spectra for both these locations.

The valves were qualified by a test performed by the vendor as part of an environmental qualification program. The test used a random biaxial input, with the devices mounted to the test table simulating an actual installation. The test procedure conformed with IEEE 344-1975. The test levels were sufficient to qualify the valves to their required acceleration level. The function of the valves was verified before and after the testing.

3.10.2.25 Containment Hydrogen Monitoring System

The containment hydrogen monitoring system (CHMS) consists of two redundant systems each consisting of an analyzer panel and a remote control panel. The analyzer panels are anchored to the floor in plant area GE at elevation 100 ft. The remote control panels for both systems are mounted in a panel (RCHMC) located in the post-LOCA sampling room in plant area GE at elevation 85 ft.

The Containment hydrogen monitoring system is Class II, Type C, Category 3, non safety related. The analyzer panels are Class II and anchorage of the analyzer panel has been seismically evaluated. Although the panel inserts for the Containment hydrogen monitor are non-safety related, the remote control panel (RCHMC) is Class I for the Class I circuits powering the related containment isolation valves.

The RCHMC panel has been structurally analyzed which includes the seismic mounting of non-safety related panel inserts. A detailed stress analysis was conducted that showed that the rack assembly is structurally adequate to withstand the DDE and HE loads.

3.10.2.26 Containment Purge Exhaust

Each monitor consists of a detector assembly and a Local Radiation Processor (LRP) located in Area L on the 100 ft elevation. The remote readout is located in the Radiation Monitoring System panel (RNRMS) in the Control Room. The detectors and LRPs are qualified by analysis based on a test simulation performed on similar equipment. The Control Room mounted equipment is qualified based on shaketable tests on the RNRMS panels performed for Victoreen. See Section 3.10.2.1.1.1.

3.10.2.27 Limit Switches

Limit switches are used to detect the position of control valves. Limit switches for motor-operated valves that are an integral part of the actuator are qualified as part of the assembly (see Section 3.10.2.19). Limit switches for air-operated valves and for motor-operated valves that are mounted on the valve actuator are qualified separately. The required acceleration level for these limit switches is 9 g, which is based on the maximum response allowable for valves having Class 1E limit switches.

Class 1E limit switches have been qualified by testing performed on several different styles. Tests have been performed by both the vendor and PG&E. These tests typically consisted of sine beats or sine dwells at 9 g peak acceleration over the frequency range of 1 to 33 Hz. Test acceleration levels met or exceeded the required level except at low frequency ranges where the level was limited to test machine capabilities. The test specimens were functionally tested before and after the vibration testing.

3.10.2.28 Containment High-Range Radiation Monitoring System

The containment high-range radiation monitoring system is used to monitor ambient gamma radiation in the containment following a LOCA. The system consists of two redundant detectors located in the containment at elevation 145 ft and a remote readout located in the postaccident monitoring panel PAM2 in the main control room. The system was supplied and qualified by Victoreen.

The radiation detectors and readouts have been qualified by random biaxial seismic simulation tests that were conducted as part of an environmental qualification test program. The test response spectra have been verified to envelop the required DDE and HE response spectra. There were no malfunctions experienced throughout the seismic tests. The test conformed to IEEE 344-1975.

3.10.2.29 Pressurizer Safety Relief Valve Position Indication

The pressurizer safety relief valve position indication system is an acoustic flow detection system that verifies valve position by sensing flow through the pressurizer relief lines. There are three channels, one for each relief valve. The system consists of four components: detector, charge converter, signal conditioner, and remote readout. The detector is an accelerometer attached to the pressurizer relief line by a metal strap. The charge converters for all three channels are housed in a stainless steel enclosure, which is mounted to the wall adjacent to the pressurizer at elevation 145 ft in the containment. The signal conditioner is located in panel RCRM in the main control room. The remote readout is mounted in the main control board. All components of the system were supplied and qualified by Technology for Energy Corporation.

The equipment comprising the system was qualified as part of an environmental qualification program conducted by the vendor. The seismic portion of the testing was conducted in accordance with IEEE 344-1975. All of the system components were qualified by tests consisting of random input, independent triaxial excitation. Tests were conducted at Structural Dynamics Research Corporation. The test response spectra from these tests have been verified to envelop the applicable DDE and HE spectra. The function of the devices was verified before and after the testing.

3.10.2.30 Heating, Ventilating, and Air Conditioning Equipment

The qualification of safety-related heating, ventilating, and air conditioning (HVAC) equipment is reviewed according to DE, DDE, and HE criteria. This HVAC equipment is associated with the following safety-related systems:

(1) Forced draft shutter

(2) Diesel generator compartment ductwork

(3) Auxiliary saltwater compartment ventilation

(4) 4-kV switchgear ventilation

(5) 480 Vac switchgear ventilation

(6) Auxiliary building-fuel handling building ventilation

(7) Control room ventilation and pressurization system

The equipment and components of Class 1 HVAC systems are listed in Table 3.10-3. They have been reviewed for seismic qualification in accordance with the spectra, defined in Section 3.7.

The equipment listed in the table is organized into qualifying groups consisting of similar types of equipment. The component subject to the worst-case qualifying condition in each group has been reviewed for compliance with acceptance criteria. This worst-case analysis in turn envelops the other components in the respective groups.

All the items in the table were reviewed for identification of the qualifying spectra. Where the most current spectra exceed the conditions under which the component was previously analyzed, a new analysis was initiated. The results of the analysis confirmed the qualification of the component or identified a physical modification. Where analysis is not appropriate, equipment testing was used to demonstrate the design performed under the qualifying seismic conditions.

3.10.2.30.1 HVAC Duct and Duct Supports

The Class I HVAC duct system consists of ducts and approximately 2,000 supports in both units. HVAC ducts are made of cold-formed steel conforming to ASTM A525, A526, and A527 with the thickness varying depending upon the duct size. The duct supports are mostly structural steel angles. Supports are attached to concrete or structural steel by bolted connections or by welding. The ducts are fastened to the supports by means of screws, rivets, or stitch welds.

3.10.2.30.1.1 Design and Acceptance Criteria

The duct and duct supports are evaluated in their as-built condition to meet the following criteria:

Loading Combination

The ducts are evaluated for the concurrent dead weight, seismic load, and pressure load. The duct supports are evaluated for dead weight and seismic load. The pressure load is the negative operating pressure of the HVAC system and is not included in the evaluation of the duct supports. The seismic loads evaluated are DDE and HE loads. The horizontal component of seismic load (DDE or HE), either transverse or longitudinal to the ducts, that results in the highest stress in the member under consideration is combined, by absolute sum, with the stresses due to vertical seismic load. As an alternative, the seismic loads from each of the three directions are combined by SRSS method.

Response Acceleration of Support System

The applicable floor response spectra where the supports are located are used for evaluation of HVAC duct and duct supports. The corresponding horizontal spectra are combined, by absolute sum, with the corresponding torsional response, as appropriate.

Acceptance Criteria

The AISI "Specification for Design of Cold-formed Steel Structural Members" is used to evaluate the design of cold-formed steel members, and part 1 of the AISC "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings" is used for the design of hot-rolled members. The allowable stresses given in AISI and AISC Specification are increased by 60 percent.

3.10.2.30.1.2 Evaluation

For duct supports, the seismic loading is evaluated for vertical plus transverse horizontal loads and vertical plus longitudinal horizontal loads. The frequency of the coupled duct and duct support system is used in determining the spectral response. The damping values used are 2 percent for DDE and 7 percent for HE.

3.10.2.31 Electric Hydrogen Recombiner System

The model B electric hydrogen recombiner system (EHRS) is designed to control and reduce post-LOCA containment hydrogen levels. The system consists of three components; control panel, power supply, and recombiner. Two model B EHRSs are provided for the DCPP site.

The recombiner was subjected to multiple frequency multiple axis testing in accordance with IEEE 344-1975. The results of the seismic testing are provided in Reference 25. The recombiner was energized and at operating temperature before, during, and after each seismic test. Following the entire testing, the recombiner was inspected for damage. No disabling damage was found. An air flow test was conducted after testing and the results show no loss of air flow. The test response spectra were checked to envelope the DDE and HE response spectra, to confirm its seismic adequacy.

The power supply and control panel were tested at the same time as documented in Reference 26. Testing was done with conservatively large accelerations over a range of applicable frequencies and conformed to the procedures given in IEEE 344-1971. The peak test input accelerations used in the power supply and control panel tests were checked to verify that they are larger than the requirements derived by DDE and HE loadings.

After each seismic test the power supply was visually inspected for structural integrity and functional operability. Both units were found to operate satisfactorily.

3.10.2.32 Reactor Vessel Level Instrumentation System

The RVLIS consists of the following instrumentation:

- RVLIS/incore thermocouple cabinets (including remote display)
- Reactor coolant level differential pressure transmitters (see Section 3.10.1.5)
- Surface mounted RTDs
- High volume sensors
- Differential pressure indicating switches (hydraulic isolators)

Typical items of the above instrumentation and electronic equipment have been type tested using multiple frequency, multiple axis seismic testing. Testing was performed in accordance with the procedures given in IEEE 344-1975. The test response spectra obtained from those tests were checked to envelope the DDE and HE response spectra.

3.10.2.32.1 RVLIS/Incore Thermocouple Cabinets

Two RVLIS/incore thermocouple cabinets (PAMs 3 and 4) are provided for DCPP application. Located within each cabinet are the microprocessor electronics, reactor coolant pump (RCP) status panel, and a remote display. The above RVLIS instrumentation is only required to operate normally before and after seismic excitation. The RCP status panel assembly is shown to be operational by the signals recorded during testing and the functional checks made after each simulated SSE. The remote display electronics must function normally by providing microprocessor output display formatted information.

The results of seismic testing of the original RVLIS/incore thermocouple cabinets are provided in Reference 27. The original remote display was not included in the cabinet tested. The original remote display was tested later to worst-case (maximum) in-cabinet response for the RVLIS/incore thermocouple cabinets. The seismic testing of the original remote display is documented in Reference 28.

Because the original Westinghouse-supplied system is obsolete and due to the lack of availability of replacement components, the obsolete RVLIS/incore thermocouple systems were replaced. The replacement processors, signal conditioners, and displays are seismically qualified by testing and analysis as documented in References 47, 48, 54 and PG&E Calculation IS-66.

3.10.2.32.2 Surface Mounted RTDs

There are 14 surface mounted RTDs used in RVLIS to measure the temperature of the reference leg impulse lines. As described in Reference 29, the surface mounted RTDs were subjected to single frequency, multiple axis sinusoidal tests and multiple frequency, multiple axis seismic tests. The RTDs tested were operational throughout all phases of the test sequence. Measurement of performance was by an evaluation of the recorded RTD output, periodic static calibrations, and numerous insulation surface mounted RTDs maintain their structural integrity and functional accuracy required.

3.10.2.32.3 High Volume Sensors

The high volume sensors are bellows designed for large volumetric displacement to accommodate thermal expansion postulated postaccident environment. The safety related performance requirement is that the sensor must maintain this pressure boundary and sensing interface between the process and filled pressure/differential pressure system without introducing any sensing errors.

The results of seismic testing are provided in Reference 30. Based upon the information provided therein, the high volume pressure sensor can successfully fulfill its safety related requirements during and after seismic testing.

3.10.2.32.4 Differential Pressure Indication Switches

The differential pressure indicating switches (hydraulic isolators) are used to seal off full process pressure in either direction and will actuate switches to indicate a unbalanced condition. The safety-related performance requirement for RVLIS is that the switch provide the isolation function without contributing a sensing error to the accuracy of a downstream pressure of differential pressure transmitter.

During the seismic test, adherence to this requirement is verified by monitoring the output of two reference transmitters receiving the pressure signal. While no safety related use is made of the switch contacts in the reactor vessel level indicating system, testing was designed to demonstrate the suitability of indicating switch use for other applications.

Seismic testing of the hydraulic isolators is provided in Reference 31. The hydraulic isolators sustained no physical damage during the seismic testing and performed their process sensing line isolation function successfully, with no leakage of the water fill.

3.10.2.33 Incore Flux Mapping Cabinets and Transfer Device

The incore flux mapping cabinets and flux mapping transfer device are non-safety related but have been seismically qualified for structural integrity for HE loadings.

The incore flux mapping cabinet is structurally identical to the NIS cabinets (see Section 3.10.2.1.1). The weight distribution of equipment within the cabinet would produce essentially the same dynamic results. Therefore the results obtained from the test of the NIS cabinets are applicable for the incore flux mapping cabinet structure.

The flux mapping transfer device is an assembly used to support control equipment to drive detectors into and out of thimbles in the reactor core. The flux mapping transfer device has been evaluated to maintain its structural integrity to withstand the HE for DCPP.

3.10.3 REFERENCES

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2. IEEE Standard 344-1975, Recommended Practices for Seismic Qualification of Class IE Equipment for Nuclear Power Generating Stations.

3. USNRC RG 1.100, Revision 1, Seismic Qualification of Electrical Equipment for Nuclear Power Plants, August 1977.

4. Description of the Systems Interactions Program for Seismically Induced Events, Revision 4, August 1980.

5. Seismic Testing of Electrical and Control Equipment (PG&E Plants), WCAP-8021, May 1973.

6. Letter No. NS-CE-1609, Eicheldinger (Westinghouse) to Stolz (NRC), Subject: "Drawer Securing Method for NIS Rack," November 1977.

7. Seismic Operability Demonstration Testing of the Nuclear Instrumentation System Bistable Amplifier, WCAP-8830, October 1976.

8. General Method of Developing Multifrequency Biaxial Test Inputs for Bistables, WCAP-8624, September 1975.

9. Consequences of Seismic-Induced Actuation of Protection Systems Relays on the Diablo Canyon Nuclear Plant, Westinghouse Electric Corporation, July 1975.

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15. Equipment Qualification Test Report Barton Differential Pressure Transmitter - Qualification Group B, WCAP-8687, Supplement 2-E04A, Revision 2, March 1983.

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23. Forced Vibration Testing of the Diablo Canyon Unit 1 Main Control Board, ANCO Engineers Inc., Document No. A-000047, May 1983.

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25. Qualification Testing for Model B Electric Hydrogen Recombiner, WCAP-9346, July 1978.

26. Electric Hydrogen Recombiner for PWR Containments Equipment Qualification Report, WCAP-7709-L, Supplement 2, September 1973.

27. Equipment Qualification Test Report, Reactor Vessel Level Instrumentation System/Incore Thermocouple Cabinet With 8080 Microprocessor Electronics and Reactor Cabinet Pump Station Panel, WCAP-8687, Supplement 2-E51A, July 1984.

28. Equipment Qualification Test Report, Remote Digital Display for the 8080 Microprocessor Electronics, WCAP-8687, Supplement 2-E46A, July 1984.

29. Equipment Qualification Test Report, Surface Mounted RTDs, WCAP-8687, Supplement 2-E48A, January 1983.

30. Equipment Qualification Test Report, High Volume Sensor-Group A, WCAP-8687, Supplement 2-E48A, January 1983.

31. Equipment Qualification Test Report, Differential Pressure Indicating Switch-Group, WCAP-8687, Supplement 2-E49A, January 1983.

32. American Iron and Steel Institute, Specification for the Design of Cold-Rolled Steel Structural Members, AISI, Washington, D.C., 1968.

33. American Institute of Steel Construction, Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, AISC, Chicago, IL, Seventh Edition.

34. Office of Nuclear Reactor Regulation, Standard Review Plan, Section 3.8.4, NRC, Washington, D.C. (November 1975).

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38. Rosemount Report D8300040, Qualification Report for Pressure Transmitters Rosemount Model 1153 Series D, (PG&E DC 6000784-7-1).

39. PG&E Seismic Calculation No. IS-35, “Seismic Qualification of Rosemount Transmitters.”

40. Equipment Qualification Test Report, Eagle 21 Process Protection System (Environmental and Seismic Testing), WCAP-8687, Supplement 2-E69A, Revision 0, May 1988.

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42. Equipment Qualification Test Report, Eagle 21 Process Protection System (Environmental and Seismic Testing), WCAP-8687, Supplement 2-E69C, Revision 0, February 1991.

43. Seismic Qualification of Electrical Equipment for Nuclear Power Plants, NRC Regulatory Guide 1.100, Revision 2, June 1988.

44. Recommended Practices for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations, IEEE 344-1987.

45. Seismic Conformation of Eagle 21 Digital Process Protection System Upgrade for Pacific Gas and Electric Company Diablo Canyon Power Plant Units 1 and 2, WCAP-13384, Revision 0, PG&E, September 1992.

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47. Trentec Test Report No. 8Q017.0, dated 11/98.

48. Altran Calculation No. 98250-C-001, Revision 0, dated May 1999.

49. PG&E Seismic Calculation No. ES-66, "Seismic Qualification of Westinghouse Supplied SSPS Cabinets."

50. PG&E Seismic Calculation No. IS-88, "Seismic Qualification of Model 3015C/3015N Rosemount Transmitters."

51. IEEE Standard 535-1986, IEEE Standard for Qualification of Class 1E Lead Storage Batteries for Nuclear Power Generating Stations.

52. USNRC RG 1.158, Revision 0, Qualification of Safety-Related Lead Acid Storage Batteries for Nuclear Power Plants, February 1989.

53. PG&E Seismic Calculation No. ES-15-1, "C&D Model LCUN-33 Vital Batteries and Two Steps Battery Rack."

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61. AMS827 Qualification Report Rev. 2 (PG&E Document Number 6021767-8).

3.11 ENVIRONMENTAL DESIGN OF MECHANICAL AND ELECTRICAL EQUIPMENT

This section provides information on the environmental aspects of DCPP equipment design. Relevant background information is presented first. This information is necessary to understand the evolutionary nature of regulatory requirements in this area, and the impact of this evolution on DCPP.

Fundamental requirements for the environmental design of equipment are embodied in the following 1967 GDCs (Reference 1):

- 1967 GDC 1: "Quality Standards"
- 1967 GDC 5: "Records Requirements"
- 1967 GDC 26: "Protection Systems Fail-Safe Design"

The 1967 GDCs predated the promulgation, in February 1971, of 10 CFR 50, Appendix A (Reference 2) and were applied in the original design of DCPP. DCPP conforms with 1967 GDCs 1, 5, and 26 as stated in Section 3.1.

For purposes of an informative comparison, PG&E evaluated its conformance with the 1971 GDCs contained in 10 CFR 50, Appendix A. This evaluation is documented in Appendix 3.1A, and in PG&E's letter to the NRC dated September 10, 1981 (Reference 3). The 1971 GDCs applicable to the environmental design of equipment are:

- 1971 GDC 1: "Quality Standards and Records"
- 1971 GDC 2: "Design Basis for Protection Against Natural Phenomena"
- 1971 GDC 4: "Environmental and Missile Design Bases"
- 1971 GDC 23: "Protection System Failure Modes"

DCPP conforms with the intent of these 1971 GDCs as stated in Appendix 3.1A.

The capability of safety-related equipment to perform as required in accident environments is of particular concern. Established engineering specification and design practices, backed up by a comprehensive quality assurance program and years of actual operating experience (in the nuclear as well as other industries), afford high confidence that equipment will perform satisfactorily in its normal service environment. In addition, redundant trains of safety-related equipment are provided so that random single failures of equipment can be accommodated. Postulated nuclear plant accidents, however, can result in significant increases in environmental parameters (temperature, pressure, humidity, radiation, etc.). The adverse conditions necessarily affect redundant equipment in different safety trains, and there is no comparable wealth of demonstrated equipment operation in such adverse environments.

For these reasons, additional assurance of safety-related equipment operability in accident environments is warranted. Electric equipment is the most susceptible to failure in extreme environments. In April 1971 the IEEE issued a Trial-Use Standard 323-1971, General Guide for Qualifying Class I Electric Equipment for Nuclear Power Generating Stations (Reference 4). IEEE 323-1971 established standards for the environmental qualification (EQ) of safety-related electric equipment. EQ confirms through tests and/or analysis that equipment is capable of fulfilling its design function despite exposure to adverse accident environment conditions. PG&E committed to IEEE 323-1971 for DCPP.

The IEEE subsequently revised IEEE 323-1971. IEEE 323-1974 (Reference 5) was issued in December 1973 and contained substantial, detailed additional information. In November 1974, the NRC endorsed the later standard in Revision 0 of RG 1.89, Qualification of Class 1E Equipment for Nuclear Power Plants (Reference 6). However, this initial version of RG 1.89 exempted DCPP from conformance with IEEE 323-1974. The guide invoked the 1974 standard only on plants whose construction permit Safety Evaluation Report (SER) was dated July 1, 1974, or after. The construction permit SERs for DCPP Units 1 and 2 were dated January 23, 1968, and November 18, 1969, respectively.

In response to concerns in the late 1970s, the NRC reexamined EQ as an "area of regulatory review which heretofore had not been adequately addressed." Among other things, this effort produced the document NUREG-0588, Interim Staff Position on Environmental Qualification of Safety-Related Electrical Equipment, (For Comment version) (Reference 7), dated December 1979.

NUREG-0588 contains two "categories" of regulatory positions. Category I positions supplement IEEE 323-1974 for equipment qualified in conformance with that standard, and Category II positions supplement IEEE 323-1971 for equipment qualified in conformance with the earlier standard. As discussed above, PG&E had been exempted from the 1974 standard in accordance with RG 1.89, Rev. 0; therefore the Category II positions were applicable to DCPP. An NRC letter to All Construction Permit and Operating License Applicants dated February 5, 1980 (Reference 8), announced issuance of the NUREG and stated that "the staff will require that applicants for operating licenses document the degree to which their qualification programs comply with the staff's positions described in NUREG-0588, and their basis for any deviations." An NRC letter to PG&E, dated March 3, 1980 (Reference 9), announced a "Change in Review Procedures" for EQ documentation for DCPP. The letter requested that PG&E analyze the adequacy of its EQ program and identify, for each item required to be qualified, the degree to which the program "complies with the [NRC] staff's position described in NUREG-0588." On May 23, 1980, the NRC issued Commission Memorandum and Order CLI-80-21 (Reference 10), which sanctioned the positions in the "For Comment" version of the NUREG.

PG&E complied as directed by the NRC. In letters dated November 13, 1980 (Reference 11), June 10, 1981 (Reference 12), and September 2, 1981 (Reference 13), PG&E supplied item-by-item comparisons of its EQ program and the NUREG-0588 Category II positions. The letter of September 2, 1981, transmitted PG&E's

Environmental Qualification Report (Revision 1) (Reference 14) to the NRC. In addition to the item-by-item NUREG-0588 comparison, this report consolidated and summarized extensive information concerning the development and implementation of the EQ program for DCPP. NRC review and acceptance of the DCPP EQ program was documented in Appendix B of SSER No. 15, dated September 1981 (Reference 15).

In 1983, the NRC issued 10 CFR 50.49 (Reference 16) to codify requirements for the environmental qualification of electric equipment important to safety. This regulation now forms the basis of the EQ program at DCPP. The regulation applies to the following three categories of equipment:

- Safety-related electric equipment ("Class 1E"): This equipment is that relied upon to remain functional during and following design basis events to ensure (a) the integrity of the reactor coolant pressure boundary, (b) the capability to shut down the reactor and maintain it in a safe shutdown condition, and (c) the capability to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the 10 CFR 100 guidelines.

- Nonsafety-related electric equipment whose failure under postulated environmental conditions could prevent satisfactory accomplishment of the safety functions specified above by the safety-related equipment.

- Certain postaccident monitoring equipment as described in RG 1.97, Revision 3 (Reference 17).

The regulation explicitly excludes equipment located in a mild environment. A mild environment is defined as an environment that would at no time be significantly more severe than the environment that would occur during normal plant operation, including anticipated operational occurrences.

Also significant in 10 CFR 50.49 are the provisions in its paragraphs (k) and (l):

> "(k) Applicants for and holders of operating licenses are not required to requalify electric equipment important to safety in accordance with the provisions of this section [i.e., in accordance with 10 CFR 50.49] if the Commission has previously required qualification of the equipment in accordance with ‘Guidelines for Evaluating Environmental Qualification of Class 1E Electrical Equipment in Operating Reactors,’ November 1979 (DOR Guidelines), <u>or NUREG-0588 (For Comment version), ‘Interim Staff Position on Environmental Qualification of Safety-Related Electrical Equipment</u>.’" [Underscore added.]
>
> "(l) Replacement equipment must be qualified in accordance with the provisions of this section unless there are sound reasons to the contrary."

Based on these provisions, DCPP is required only to upgrade the qualification level of replacement equipment installed after the effective date of the rule (February 22, 1983), provided that the equipment is required to be environmentally qualified, and provided there are no sound reasons to the contrary. "Upgrading" means bringing the qualification level of the equipment into conformance with the standards contained in IEEE 323-1974 supplemented by the Category I positions in NUREG-0588.

Qualification of equipment installed prior to February 22, 1983, need only be to the level specified by IEEE 323-1971 supplemented by the Category II positions in NUREG-0588. Guidance relative to what constitutes "sound reasons" for not upgrading the qualification level of replacement equipment is contained in Revision 1 of RG 1.89 (Reference 18). New (i.e., nonreplacement) equipment that is required to be qualified, and that was/is installed after February 22, 1983, is required to be qualified to the level of IEEE 323-1974 and NUREG-0588 Category I.

Although the applicable GDC apply also to safety-related mechanical equipment, 10 CFR 50.49 does not prescribe any additional testing or documentation requirements for mechanical equipment. Electric equipment (the subject of the regulation) is inherently much more susceptible to failure in an adverse environment. However, 10 CFR 50.49 does cover:

- Environmental sealing appurtenances (e.g., gaskets, o-rings, conduit seals) for electric equipment that are required to demonstrate the environmental qualification of the associated electric equipment item

- Mechanical components that are integral to and required for the operation of electromechanical devices (e.g., switches, relays, valve operators, solenoid valves, pressure detectors, etc.)

The function of serving as part of the reactor coolant pressure boundary is considered to be a purely mechanical function. Hence, components that fulfill this function alone (i.e., that are not otherwise required to be environmentally qualified) are outside of the scope of 10 CFR 50.49. 10 CFR 50.49 is thus now the governing regulation for EQ at DCPP. PG&E has certified its compliance with the regulation as required by NRC Generic Letter 84-24, "Certification of Compliance to 10 CFR 50.49, Environmental Qualification of Electric Equipment Important to Safety for Nuclear Power Plants" (Reference 19). The PG&E certification is documented in letter DCL-85-072 to the NRC dated February 22, 1985 (Reference 20). In 1991, PG&E issued DCM T-20, Environmental Qualification, Revision 0 (Reference 21), to document in detail the design bases and PG&E commitments for the EQ program at DCPP.

3.11.1 EQUIPMENT IDENTIFICATION

The methodology described below was used to develop the EQ master list for DCPP. This methodology is documented in the DCPP Environmental Qualification Report, Revision 1 (Reference 14), dated September 1981.

3.11.1.1 Bounding List Development

In determining the safety-related electrical equipment that would require environmental qualification, PG&E considered the following items:

- Accidents and transients analyzed in the FSAR Update

- Electrical equipment required to achieve or support:

  - emergency reactor shutdown
  - containment isolation
  - containment and reactor heat removal
  - prevention of a significant release of radioactive material to the environment

- Electrical equipment required to perform safety-related actions noted in emergency procedures

At DCPP, a Class IA classification is assigned to instrumentation required to perform a safety function (including, but not limited to, the protection system and the engineered safety features system). This encompasses Class 1E instrumentation and related mechanical equipment (e.g., valves, etc.). In addition, PG&E assigns a Class 1E classification to electrical equipment other than instrumentation (e.g., cables, motors, etc.). The PG&E document Classification of Structures, Systems, and Components for Diablo Canyon Units 1 and 2 (Q-List) (Reference 22) defines the design classification criteria and lists the classification associated with DCPP mechanical, electrical, and instrumentation and control systems and components.

To develop the bounding list of equipment that would be considered for environmental qualification, PG&E prepared a list of all Class IA safety functions at DCPP, a list of the systems needed to perform the safety functions, and a list of the electrical equipment needed for those systems to operate. The equipment list was developed by integrating the Class IA list with the Class 1E equipment other than instrumentation for the required systems.

PG&E also conducted a review of its emergency operating procedures to identify all of the equipment included in these procedures. The instruments (for RG 1.97 Type A/Category 1 variables only) identified in this review, combined with the integrated IA/1E classification listing discussed above, was used as the base list of equipment eligible for environmental qualification at DCPP. (Note that activities related to compliance with NRC RG 1.97, Revision 3, proceeded independently and resulted in other instruments being added to the EQ list.)

3.11.1.2 Exemptions

PG&E then performed case-by-case evaluations to determine which of the identified equipment items may be exempted from qualification. In making these determinations, PG&E utilized certain evaluation criteria that were provided as guidelines by the NRC Staff. The list of exempt devices was developed by verifying whether specific equipment meets any of the following three criteria:

- The equipment does not perform essential safety functions in a harsh environment, and any failure of such equipment in a harsh environment will not impact safety-related functions or mislead an operator.

- The equipment performs its function before it is exposed to a harsh environment, the sufficiency of any time margin provided is adequately justified, and the subsequent failure of the equipment as a result of the exposure to the harsh environment does not degrade other safety functions or mislead the operator.

- The safety-related function of a particular piece of equipment can be accomplished by some other designated equipment that has been adequately qualified and satisfies the single failure criterion.

PG&E identified a number of devices that satisfied the above criteria and, therefore, could be exempted from environmental qualification.

3.11.1.3 Cable and Terminations

Consideration was also given to the wiring and terminations associated with each equipment item that was required to be qualified. As part of the initial NUREG-0588 qualification effort, PG&E chose the following course of action related to sealing for wiring and terminations.

All devices qualified to function in a harsh environment have environmentally qualified electrical sealing assemblies added unless they are qualified with their own sealing system, or do not require seals to maintain their environmental qualification (for example, equipment qualified for a harsh radiation environment only). All Class 1E wiring terminations for EQ equipment inside containment are made by means of environmentally qualified splices, thus avoiding terminal blocks, except for (a) terminal blocks within devices that have the sealing assemblies mentioned above, and (b) terminal blocks inside vendor-supplied equipment assemblies that were procured and environmentally qualified as an assembly. Connections between circuit conductors and the electrical containment penetrations, both inside and outside containment, are also made by means of qualified splices.

Wire and cable used for Class 1E application inside containment and in areas with potential for harsh environment outside containment were generically qualified to perform their safety function while being exposed to the postulated accident environment. Cable EQ files are segregated according to cable manufacturer. PG&E's procedures assure that only such qualified wire and cable are used for Class 1E application in these areas.

Class 1E cables and associated cable devices (penetrations, splices, seals, etc.) requiring environmental qualification were identified by a search of purchase documents and added to the list of equipment items requiring qualification.

3.11.1.4 Class 1E Electrical Equipment Qualification List Maintenance

To facilitate maintaining the DCPP EQ master list as a living document (i.e., to maintain it current on an ongoing basis), the SAP Functional Locations along with Controlled Drawing 050909 constitute the DCPP "Class 1E Electrical Equipment Qualification List." A hardcopy output of this EQ Masterlist information is available as a living document. Controlled Drawing 050909 lists the EQ equipment that does not have tag numbers (i.e., electrical cables, connectors, splices) and provides guidance to print a hardcopy of the EQ Masterlist Information and a report of changes since the last masterlist review.

There are two important "notes" that may be assigned to equipment on the EQ master list:

- "Note 11" - identifies devices that are located in a mild area; environmental qualification is not required.

- "Note 16" - identifies devices that are located in an area subject to a harsh environment but that are not required to function for accident mitigation or postaccident monitoring of the event that causes the harsh environment. Note 16 is also ascribed to devices that complete their required safety function prior to being exposed to the harsh environment.

The bases for "partial qualification" are also documented in the EQ files. Partial qualification is applied when the equipment item is only required to function for specific accidents that may cause a harsh environment. The EQ File for the item, therefore, contains a partial exemption to a specific design basis accident.

Changes to the EQ master list can occur as a result of plant design changes, maintenance, or as a consequence of revised postaccident environment analyses. For example:

- Design changes that add new equipment to the plant will result in additions to the list if the equipment is within the scope of 10 CFR 50.49(b).
- Design changes or reanalyses that add or eliminate areas of harsh environment can result in changes in the list.

- Design changes that change equipment location (either to a harsh environment from a mild environment, or vice versa) can result in changes in the list.

- Design changes (or analyses) that change equipment functional requirements can result in additions to or deletions from the list, or in changes to the "Note 16" assignments.

- Maintenance activities can result in equipment replacement/upgrade with consequent changes to the list (new vendor, model number, etc.).

Design and maintenance activities are procedurally controlled to ensure that any associated impact on EQ program implementation is identified, evaluated, and tracked.

3.11.2 QUALIFICATION TESTS AND ANALYSES

3.11.2.1 Accident Environments

Fundamental to the environmental qualification of equipment is the determination of the changes in environmental parameters to which the equipment would be subjected in the event of an accident. The environmental parameters considered are pressure, temperature, humidity, radiation, caustic spray, and submergence. Extensive modeling has been performed to determine the time-dependent behavior of these parameters following postulated accidents. This information is compiled and maintained in the following two controlled PG&E documents:

- DCM T-20, Environmental Qualification (Reference 21): Appendix A of DCM T-20 is titled "Environmental Conditions for EQ of Electric Equipment"

- DCM T-12, Pipe Break (HELB/MELB), Flooding, and Missiles (Reference 23)

These documents are living documents and are revised as changes occur in the derived postaccident environmental conditions. Such changes can occur as a result of plant design changes or new/revised analyses, and are evaluated for EQ impact. Consequent actions are taken as required to assure that equipment required to be environmentally qualified remains environmentally qualified.

3.11.2.2 Normal Environments

Normal service environments are of concern with respect to environmental qualification because of their effect on equipment's qualified life.
Equipment items requiring qualification typically comprise various different materials: metals, plastics, elastomers, etc. The non-metallic materials are often susceptible to slow degradation over time as a result of exposure to elevated temperatures or

radiation. This degradation can limit the capability of an equipment item to fulfill its design function in an accident environment. Qualified life is thus defined as "the period of time for which satisfactory performance can be demonstrated for a specific set of service conditions" (Reference 5). Qualified life does not include the time period for which the equipment item would be required to perform during and following an accident; EQ testing and analyses are based on the assumption that the accident for which the item would be required would occur right at the end of the item's qualified life. Thus, an item having a ten-year qualified life, for example, can be relied on until the very end of the tenth year.

The normal environments used for equipment environmental qualification are specified in Appendix A of DCM T-20 (Reference 21). A qualified life has been determined for every equipment item that requires environmental qualification. Equipment having a qualified life less than the 40-year life of the plant is replaced or refurbished, in accordance with applicable maintenance procedures, prior to the end of its qualified life. Procedures also ensure that any other required maintenance on an environmentally qualified equipment item be performed such that the item remains qualified.

PG&E uses 10^3 Rads total integrated dose (TID) outside containment as the threshold gamma exposure above which qualification for a radiation environment is required (Reference 14). Note that the basis for the radiation environment qualification of nonelectronic electrical devices subject to a gamma TID of between 10^3 and 10^4 Rads is documented in Design Calculation EZ-02 (Reference 25).

3.11.2.3 NUREG-0588 Category II Qualification

Equipment within the scope of 10 CFR 50.49 installed before February 22, 1983, is required, as a minimum, to be environmentally qualified in accordance with the standards contained in IEEE 323-1971 (Reference 4) supplemented by the Category II positions contained in NUREG-0588 (Reference 7). This is the minimum level of qualification for equipment at DCPP. Qualification to a higher level is at the discretion of PG&E for this equipment.

3.11.2.4 NUREG-0588 Category I Qualification

New (i.e., nonreplacement) equipment within the scope of 10 CFR 50.49 installed after (and including) February 22, 1983, is required to be environmentally qualified in accordance with the standards contained in IEEE 323-1974 (Reference 5) supplemented by the Category I positions contained in NUREG-0588 (Reference 7).

Replacement equipment, for equipment within the scope of 10 CFR 50.49, installed after (and including) February 22, 1983, is required to be environmentally qualified in accordance with the standards contained in IEEE 323-1974 (Reference 5) supplemented by the Category I positions contained in NUREG-0588 (Reference 7), unless there are sound reasons to the contrary. Acceptable "sound reasons to the contrary" are delineated in RG 1.89, Revision 1 (Reference 18), and are documented, if invoked, in the applicable EQ file.

3.11.3 QUALIFICATION TEST RESULTS

EQ testing and analysis have demonstrated that all equipment identified as requiring environmental qualification is capable of fulfilling its design function despite exposure to harsh accident environment conditions. Requirements for documentation are delineated in IEEE 323-1971 and the Category II positions in NUREG-0588, or in IEEE 323-1974 and the NUREG-0588, Category I positions, as applicable.

Dedicated EQ files have been established to document the environmental qualification of equipment based on the results of testing and analyses. The files are segregated on a discipline basis and according to manufacturer. The files include qualification summaries, test reports, applicable correspondence, and information that associates the installed equipment with the qualification documents.

The EQ files are prepared and maintained in accordance with procedures and are readily available to engineering and maintenance personnel. They are maintained as quality records (Reference 24).

3.11.4 LOSS OF VENTILATION

Safety provisions of the HVAC systems are discussed in Section 9.4.

3.11.5 REFERENCES

1. "General Design Criteria for Nuclear Power Plant Construction Permits," published in the Federal Register for public comment by the AEC on July 11, 1967 (see 32FR10213).

2. Title 10 of the U.S. Code of Federal Regulations, Part 50, Appendix A, "General Design Criteria for Nuclear Power Plants," promulgated in the Federal Register on February 20, 1971 (see 36FR3255).

3. PG&E (Mr. Philip A. Crane, Jr.) letter to NRC (Mr. Frank J. Miraglia, Jr., Chief, Licensing Branch No. 3, Division of Licensing), dated September 10, 1981, re: "An Itemized Review of the Diablo Canyon Power Plant's Compliance with the Requirements of 10 CFR Parts 20, 50 and 100."

4. IEEE Trial-Use Standard 323-1971, General Guide for Qualifying Class I Electric Equipment for Nuclear Power Generating Stations, dated April 1971.

5. IEEE Standard 323-1974, IEEE Standard for Qualifying Class 1E Equipment for Nuclear Power Generating Stations, dated December 1973.

6. RG 1.89, Qualification of Class 1E Equipment for Nuclear Power Plants, AEC, Revision 0, dated November 1974.

7. NRC, Interim Staff Position on Environmental Qualification of Safety-Related Electrical Equipment, NUREG-0588, For Comment version, dated December 1979.

8. NRC (Mr. D.F. Ross, Jr., Acting Director, Division of Project Management, Office of Nuclear Reactor Regulation) letter to All Construction and Operating License Applicants dated February 5, 1980, re: "Issuance of NUREG-0588, 'Interim Staff Position on Equipment Qualification of Safety-Related Electrical Equipment.'"

9. NRC (Mr. John F. Stolz, Chief, Light Water Reactors Branch No. 1, Division of Project Management) letter to PG&E (Mr. John C. Morrissey) dated March 3, 1980, re: "Change in Review Procedures for Equipment Qualification Documentation for the Diablo Canyon Nuclear Power Plants, Units 1 & 2."

10. NRC, Commission Memorandum and Order CLI-80-21, dated May 23, 1980, re: "Petition for Emergency and Remedial Action."

11. PG&E (Mr. Philip A. Crane, Jr.) letter to NRC (Mr. A. Schwencer, Acting Chief, Licensing Branch No. 3, Division of Licensing, Office of Nuclear Reactor Regulation) dated November 13, 1980, re: "PG&E's review of the DCPP Environmental Qualification Program Using NUREG-0588 as the Basis for the Evaluation."

12. PG&E (Mr. Philip A. Crane, Jr.) letter to NRC (Mr. Frank J. Miraglia, Jr., Chief, Licensing Branch No. 3, Division of Licensing, Office of Nuclear Reactor Regulation) dated June 10, 1981, re: "Transmittal of the DCPP Environmental Qualification Report (Rev. 0)."

13. PG&E (Mr. Philip A. Crane, Jr.) letter to NRC (Mr. Frank J. Miraglia, Jr., Chief, Licensing Branch No. 3, Division of Licensing, Office of Nuclear Reactor Regulation) dated September 2, 1981, re: "Transmittal of the DCPP Environmental Qualification Report (Rev. 1)."

14. DCPP Environmental Qualification Report, Revision 1, dated September 1981.

15. NRC, Safety Evaluation Report Related to the Operation of Diablo Canyon Nuclear Power Plant, Units 1 and 2, NUREG-0675, Supplement No. 15, dated September 1981.

16. Title 10 of the U.S. Code of Federal Regulations, Section 50.49 (10 CFR 50.49), "Environmental Qualification of Electric Equipment Important to Safety for Nuclear Power Plants," promulgated in the Federal Register on January 21, 1983 (see 48FR2729), effective February 22, 1983.

17. RG 1.97, Instrumentation for Light-Water-Cooled Nuclear Power Plants to Assess Plant and Environs Conditions During and Following an Accident, NRC, Revision 3, dated May 1983.

18. RG 1.89, Environmental Qualification of Certain Electric Equipment Important to Safety for Nuclear Power Plants, NRC, Revision 1, dated June 1984.

19. NRC Generic Letter 84-24, "Certification of Compliance to 10 CFR 50.49, Environmental Qualification of Electric Equipment Important to Safety for Nuclear Power Plants," dated December 27, 1984.

20. PG&E (Mr. J.D. Shiffer) letter DCL-85-072 to NRC (Mr. Darrell G. Eisenhut, Director, Division of Licensing, Office of Nuclear Reactor Regulation) dated February 22, 1985, re: "Response to NRC Generic Letter 84-24."

21. DCPP Design Criteria Memorandum T-20, Environmental Qualification, PG&E.

22. PG&E controlled document, Classification of Structures, Systems, and Components for Diablo Canyon Units 1 and 2 (Q-List); see also Reference 8 of Section 3.2.

23. DCPP Design Criteria Memorandum T-12, Pipe Break (HELB/MELB), Flooding, and Missiles, PG&E.

24. PG&E (Mr. J. O. Schuyler) letter DCL-84-088 to NRC (Mr. John B. Martin, Regional Administrator, NRC Region V) dated March 1, 1984, re: "Response to NRC Inspection Report 50-275/83-40."

25. Calculation EZ-02, Environmental Qualification Requirements: Bases for ‘Note 11’ and ‘Note 16’ Devices; Bases for the Required Post DBA Operating Time and ‘Minimum Required Qualification Time’; Review of Potential 10 CFR 50.49(b)(2) Devices; and Radiation of Non-Electronic Devices Subject to a TID between 10^3 and 10^4 Rads.

TABLE 3.1-1

GENERAL DESIGN CRITERIA APPLICABILITY

1967 GDC GROUP	1967 GDC	1971 GDC	1987 GDC
Overall Plant Requirements	1		
	2		
		3 [a]	
	4		
	5		
Protection by Multiple Fission Product Barriers	6 to 10		
Nuclear and Radiation Controls	11	19 [b]	
	12 to 18		
Reliability and testability of protection systems	19 to 26		
Reactivity Control	27 to 32		
Reactor Coolant Pressure Boundary	33 to 36		
Engineered Safety features	37		
	38		
		17 [c]	
		18 [c]	
	40		4 [d]
	41-65		
Fuel and Waste Storage Systems	66 to 69		
Plant Effluents	70		

(a) GDC 3 - 1971 replaces GDC 3 - 1967
(b) GDC 11 - 1967 is supplemented by GDC 19 - 1971 for Dose
(c) GDC 17 - 1971 and GDC 18 - 1971 replace GDC 39-1967
(d) GDC 40 - 1967 is supplemented by GDC 4 – 1987 for LBB only.

TABLE 3.1-2

MATRIX OF 1971 GDCs to ASSOCIATED 1967 GDCs

1971 GDC Number	1971 GDC Title	Associated 1967 GDC	1967 GDC Title
1	Quality Standards and Records	1	Quality Standards.
		5	Records Requirements.
2	Design Basis for Protection Against Natural Phenonema	2	Performance Standards.
3	Fire Protection	3	Fire Protection
4	Environmental and Missile Design Bases	40	Missile Protection.
5	Sharing of Structures, Systems, and Components	4	Sharing of Systems.
6 to 9	(Not issued, not used).	---	---
10	Reactor Design.	6	Reactor Core Design.
11	Reactor Inherent Protection.	8	Overall Power Coefficient
12	Suppression of Reactor Power Oscillations.	7	Suppression of Power Oscillations.
13	Instrumentation and Control.	12	Instrumentation and Control Systems.
		13	Fission Process Monitors and Controls.
		14	Core Protection Systems.
		15	Engineered Safety Features Protection Systems.
14	Reactor Coolant Pressure Boundary.	9	Reactor Coolant Pressure Boundary.
15	Reactor Coolant System Design.	No direct association with 1967 GDC.	---
16	Containment Design.	10	Containment.
		49	Containment Design Basis.

TABLE 3.1-2

MATRIX OF 1971 GDCs to ASSOCIATED 1967 GDCs

1971 GDC Number	1971 GDC Title	Associated 1967 GDC	1967 GDC Title
17	Electric Power Systems.	39 (Superseded by 1971 Criterion 17 and 18).	Emergency Power for Engineered Safety Features.
18	Inspection and Testing of Electric Power Systems.	39 (Superseded by 1971 Criterion 17 and 18).	---
19	Control Room.	11	Control Room.
20	Protection System Functions.	14 15 20 21 25	Core Protection Systems. Engineered Safety Features Protection Systems. Protection Systems Redundancy and Independence. Single Failure Definition. Demonstration of Functional Operability of Protection Systems.
21	Protection System Reliability and Testability.	19	Protection Systems Reliability.
22	Protection System Independence.	20 21 22 23	Protection Systems Redundancy and Independence. Single Failure Definition. Separation of Protection and Control Instrumentation Systems. Protection Against Multiple Disability of Protection Systems.
23	Protection System Failure Modes.	26	Protection Systems Fail-Safe Design.
24	Separation of Protection and Control Systems.	22	Separation of Protection and Control Instrumentation Systems.

TABLE 3.1-2

MATRIX OF 1971 GDCs to ASSOCIATED 1967 GDCs

1971 GDC Number	1971 GDC Title	Associated 1967 GDC	1967 GDC Title
25	Protection System Requirements for Reactivity Control Malfunctions.	31	Reactivity Control Systems Malfunction.
26	Reactivity Control System Redundancy and Capability.	27	Redundancy of Reactivity Control.
		28	Reactivity Hot Shutdown Capability.
		29	Reactivity Shutdown Capability.
27	Combined Reactivity Control Systems Capability.	30	Reactivity Holddown Capability.
28	Reactivity Limits.	30	Reactivity Holddown Capability.
29	Protection Against Anticipated Operational Occurrences.	19	Protection Systems Reliability.
		20	Reactivity Shutdown Capability.
30	Quality of Reactor Coolant Pressure Boundary.	9	Reactor Coolant Pressure Boundary.
		16	Monitoring Reactor Coolant Pressure Boundary.
31	Fracture Prevention of Reactor Coolant Pressure Boundary.	34	Reactor Coolant Pressure Boundary Rapid Propagation Failure Prevention.
		35	Reactor Coolant Pressure Boundary Brittle Fracture Prevention.
32	Inspection of Reactor Coolant Presssure Boundary.	36	Reactor Coolant Pressure Boundary Surveillance.
33	Reactor Coolant Makeup.	No direct association with 1967 GDC.	---
34	Residual Heat Removal.	No direct association with 1967 GDC.	---

TABLE 3.1-2

MATRIX OF 1971 GDCs to ASSOCIATED 1967 GDCs

1971 GDC Number	1971 GDC Title	Associated 1967 GDC	1967 GDC Title
35	Emergency Core Cooling.	37	Engineered Safety Features Basis for Design.
		44	Emergency Core Cooling Systems Capability.
36	Inspection of Emergency Core Cooling System.	45	Inspection of Emergency Core Cooling Systems.
37	Testing of Emergency Core Cooling System.	38	Reliability and Testability of Engineered Safety Features.
		46	Testing of Emergency Core Cooling System Components.
		47	Testing of Emergency Core Cooling Systems.
		48	Testing of Operational Sequence of Emergency Core Cooling Systems.
38	Containment Heat Removal.	49	Containment Design Basis.
		52	Containment Heat Removal Systems.
39	Inspection of Containment Heat Removal System.	58	Inspection of Containment Pressure-Reducing Systems.
40	Testing of Containment Heat Removal System.	59	Testing of Containment Pressure-Reducing Systems.
		60	Testing of Containment Spray Systems.
		61	Testing of Operational Sequence of Containment Pressure-Reducing Systems.
41	Containment Atmosphere Cleanup.	37	Engineered Safety Features Basis for Design.
42	Inspection of Containment Atmosphere Cleanup Systems.	62	Inspection of Air Cleanup Systems.

TABLE 3.1-2

MATRIX OF 1971 GDCs to ASSOCIATED 1967 GDCs

1971 GDC Number	1971 GDC Title	Associated 1967 GDC	1967 GDC Title
43	Testing of Containment Atmosphere Cleanup Systems.	63 64 65	Testing of Air Cleanup Systems Components. Testing of Air Cleanup Systems. Testing of Operational Sequence of Air Cleanup Systems.
44	Cooling Water.	No direct correlation with 1967 GDC.	---
45	Inspection of Cooling Water System.	No direct association with 1967 GDC.	---
46	Testing of Cooling Water System.	No direct association with 1967 GDC.	---
47 to 49	(Not issued, not used).	---	---
50	Containment Design Basis.	49	Containment Design Basis.
51	Fracture Prevention of Containment Pressure Boundary.	50	NDT Requirement for Containment Material.
52	Capability for Containment Leakage Rate Testing.	54 55	Containment Leakage Rate Testing. Containment Periodic Leakage Rate Testing.
53	Provisions for Containment Testing and Inspection.	56	Provisions for Testing Penetrations.
54	Piping Systems Penetrating Containment.	57	Provisions for Testing of Isolation Valves.

TABLE 3.1-2

MATRIX OF 1971 GDCs to ASSOCIATED 1967 GDCs

1971 GDC Number	1971 GDC Title	Associated 1967 GDC	1967 GDC Title
55	Reactor Coolant Pressure Boundary Penetrating Containment.	51	Reactor Coolant Pressure Boundary Outside Containment.
		57	Provisions for Testing Isolation Valves.
56	Primary Containment Isolation.	53	Containment Isolation Valves.
57	Closed System Isolation Valves.	53	Containment Isolation Valves.
58 to 59	(Not issued, not used).	---	---
60	Control of Releases of Radioactive Materials to the Environment.	17	Monitoring Radioactivity Releases.
		70	Control of Releases of Radioactivity to the Environment.
61	Fuel Storage and Handling and Radioactivity Control.	68	Fuel and Waste Storage Radiation Shielding.
		69	Protection Against Radioactivity Release from Spent Fuel and Waste Storage.
62	Prevention of Criticality in Fuel Storage and Handling.	66	Prevention of Fuel Storage Criticality.
63	Monitoring Fuel and Waste Storage.	18	Monitoring Fuel and Waste Storage.
64	Monitoring Radioactivity Releases.	17	Monitoring Radioactivity Releases.

TABLE 3.2-1

DESIGN CLASSIFICATION OF STRUCTURES, SYSTEMS, AND COMPONENTS (SSCs)

PG&E Design Class I	PG&E Design Class II	PG&E Design Class III
<u>Applicability</u>		
SSCs important to safety, including SSCs required to assure (1) the integrity of the reactor coolant pressure boundary, (2) the capability to shut down the reactor and maintain it in a safe shutdown condition, or (3) the capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of 10 CFR Part 100.	SSCs important to reactor operation, but not essential to safety, including plant features not required to be PG&E Design Class I.	SSCs not related to reactor operation or safety.

PG&E Design Class I		PG&E Design Class II		PG&E Design Class III
Requirements				
1. Quality Standards - SSCs required to meet AEC GDC 1, 1967.		1. Quality Standards – SSCs not required to meet AEC GDC 1, 1967.		1. Quality Standards - SSCs not required to meet AEC GDC 1, 1967.
2. Quality Assurance - SSCs required to meet Appendix B to 10 CFR Part 50.		2. Quality Assurance - SSCs not required to meet Appendix B to 10 CFR Part 50. Specific QA requirements may be applied to selected features. Refer to the DCPP Q-List for further details.		2. Quality Assurance – SSCs not required to meet Appendix B to 10 CFR Part 50. Specific QA requirements may be applied to selected features. Refer to the Q-List for further details.
3. Seismic Design – SSCs required to meet AEC GDC 2, 1967. SSCs are designed to maintain their structural/ pressure boundary integrity, may be designed to perform an active function, when subjected to loading associated with the double design earthquake (DDE) and the Hosgri earthquake (HE). Refer to the DCPP Q-List for the requirements associated with individual SSCs.		3. Seismic Design - SSCs not required to meet AEC GDC 2, 1967. SSCs not designed to withstand effects of design basis earthquakes, except where specifically designated in the Q-list.		3. Seismic Design – SSCs not required to meet AEC GDC 2, 1967. SSCs not designed to withstand effects of design basis earthquakes, except where specifically designated in the DCPP Q-list.

TABLE 3.2-2

PG&E DESIGN CLASS QUALITY/CODE CLASS, AND PIPING SYMBOL vs. INDUSTRY CODE FOR FLUID SYSTEMS – Note (a)(1)

	PG&E Engineering		Original Code Classification		
PG&E Design Class	**PG&E Quality/Code Class**	**PG&E Piping Symbol**	**Orig. Design Codes & Standards**	**Orig. Fab., Erection, & Inspection Code**	**Remarks/Notes**
I	I	NONE	ASA B31.1-1955	ASME B&PV Code, Section III-1971	Note (b), (p), (r), (t)
I	I	A	ANSI B31.1-1967	ANSI B31.7-1969 w/ 1970 Addenda, Class I	Note (h), (p), (r), (t), (u)
I	II	B	ANSI B31.7-1969 with 1970 Addenda, Class II	ANSI B31.7-1969 with 1970 Addenda, Class II	Note (p), (r), (t), (u)
I	II	@	ANSI B31.1-1967	ASME B&PV Code Section I-1968; Section III - 1968	Note (g), (n), (p), (r), (t), (u)
I	III	C	ANSI B31.7- 1969 with 1970 Addenda, Class III	ANSI B31.7-1969 With 1970 Addenda, Class III	Note (p), (r), (t), (u)
I	III	D	ANSI B31.1- 1967; ANSI B 31.7-1969 with 1970 Addenda, Class III	ANSI B31.1- 1967; ANSI B 31.7-1969 with 1970 Addenda, Class III	Note (i), (p), (q), (r), (t), (u)
II	-	G	ANSI B31.1-1967 and NFPA Standards	ANSI B31.1-1967 and NFPA Standards	Note (d), (j), (p), (t), (u)
I	III	J	ANSI B31.1-1967	ANSI B31.1-1967	Note (k), (p), (r), (t), (u)
II	-	E	ANSI B31.1-1967	ANSI B31.1-1967	Note (p), (t), (u)
II	-	F	ANSI B31.1-1967	ANSI B31.1-1967	Note (e), (c), (p), (t), (u)

PG&E Design Class	PG&E Engineering PG&E Quality/Code Class	PG&E Piping Symbol	Original Code Classification Orig. Design Codes & Standards	Orig. Fab., Erection, & Inspection Code	Remarks/Notes
II	-	G1	NFPA Standards	NFPA Standards	Note (d), (p), (s), (t)
II	-	H	ANSI B31.1-1967	ANSI B31.1-1967	Note (c), (p), (t), (u)
II	-	-	Applicable industry codes and standards	Applicable industry codes and standards	Note (o), (p), (t)
III	-	-	Applicable industry codes and standards	Applicable industry codes and standards	Note (o), (p), (t)
II	-	@	ASME B&PV Code, Section I-1980	ASME B&PV Code, Section I-1980	Note (m), (n), (p), (t)

Notes:

(a) Deleted.

(a)(1) It is recognized that during the design and construction of DCPP Units 1 and 2, significant industry and regulatory changes were made in establishing common methods of classification, e.g., ANSI N18.2 (Reference 1), Safety Guide 26 March 1972 (Reference 2), Safety Guide 29 June 1972 (Reference 3), and NRC Regulatory Guide 1.143, Rev. 1, October 1979 (Reference 6). However, these methods all differ slightly in detail from those used for the DCPP. Although the form and intent of these regulatory and industry documents are similar, these are not the DCPP Licensing Basis.

(b) Reactor coolant loop and pressurizer surge line piping. Design to ASA B31.1-1955 using Nuclear (N) Code Cases N-7, N-9, and N-10. Fabrication, erection, and inspection to ASME Boiler and Pressure Vessel (B&PV) Code Section III-1971.

(c) Radioactive system (PG&E QA Class R). Future activities such as repair, replacement, maintenance, or testing shall be performed per Regulatory Guide 1.143 (Reference 6) for those portions of the systems designated as PG&E Piping Symbol F or H on the piping schematics. As part of the DCPP Design and not Licensing Basis.

(d) Certain portion of the fire protection system have not been designed or constructed under a quality assurance program meeting all requirements of 10 CFR 50, Appendix B. However, activities such as repair, replacement, maintenance, or testing shall be performed in accordance with the QA recommendations described in Appendix A to NRC BTP 9.5-1 (Reference 5) and PG&E Program Directive (PD) OM8 (PG&E QA Class G). Quality requirements administered shall be commensurate with the safety function of the SSC.

(e) Piping is seismically qualified for the Design Earthquake.

(f) Deleted

(g) Feedwater piping from the (final) main feedwater check valve to the steam generator; auxiliary feedwater from the main feedwater line back to the second check valve; main steam piping from the steam generator to the main steam isolation valve; steam generator blowdown piping from the steam generator to the first valve outside containment; design to ANSI B31.1-1967; fabrication, erection, and inspection to ASME B&PV Code Section I-1968. Requirements for the main steam safety valves are in accordance with ASME B&PV Code Section III-1968.

(h) Design to ANSI B31.1-1967. Fabrication, erection, and inspection to ANSI B31.7-1969 with 1970 Addenda, Class I.

(i) This PG&E Piping Symbol applies to: (1) the spent fuel pool cooling loop; (2) the auxiliary feedwater pump suction piping from the fire water storage tank; and (3) the refueling water purification loop from and to the refueling water storage tank. This piping was upgraded from Design Class II, Piping Symbol E. ANSI B31.1 applies for work performed prior to the upgrade. ANSI B31.7 applies to work performed after the upgrade.

(j) Piping is seismically qualified for the Hosgri earthquake.

(k) Piping originally installed as Design Class II, but has been upgraded to require seismic qualification for the Hosgri earthquake. All repair, replacement, and new construction shall be in accordance with the requirements for QA Class Q SSCs.

(l) Deleted.

(m) Auxiliary Boiler No. 0-2 and its external piping conforms to ASME B&PV Code Section I-1980 through summer 1980 Addenda.

(n) The PG&E Piping Symbol '@' is referred to in the UFSAR and the Q-List. However, this symbol is not used on the piping schematics for designation purposes; instead, the line is bubbled (i.e., -0-0-) and the notes describe the applicable code(s).

(o) Refer to the HVAC system schematics for the symbols used for ductwork

(p) Refer to PG&E Drawings 102028 (Unit 1) and 104628 (Unit 2) for ASME Code Boundaries for Inservice Inspection.

(q) Original design, fabrication, construction, and testing of Piping Symbol D piping was performed in accordance with ANSI B31.1, 1967. ANSI B31.7 applies for new work. Refer to mechanical component drawings for codes and standards applicable to the component.

(r) The design, fabrication, construction, and testing of all PG&E Quality/Code Class I fluid systems and components are in accordance with the accepted industry codes and standards that were in effect during the design and construction of DCPP. If fluid systems and components were designed and constructed in accordance with codes and standards outside of the requirements of the above-mentioned documents, additional quality standards have normally been applied.

The design, fabrication, construction, and testing of all PG&E Quality/Code Class II fluid systems and components are in accordance with the accepted industry codes and standards that were in effect during the design and construction of DCPP. If fluid systems and components were designed and constructed in accordance with codes and standards outside of the requirements of the above-mentioned documents, additional quality standards have normally been applied.

The design, fabrication, construction, and testing of all PG&E Quality/Code Class III fluid systems and components are in accordance with the accepted industry codes and standards that were in effect during the design and construction of DCPP. If fluid systems and components were designed and constructed in accordance with codes and standards outside of the requirements of the above- mentioned documents, additional quality standards have normally been applied.

An exception exists to the above for PG&E Quality Code Class III, Piping Symbol D piping. These are systems or portions of systems which were originally constructed as Design Class II and were subsequently upgraded to Design Class I, usually because a change in the requirements for the system. For such piping, the design analysis is in accordance with PG&E Design Class I criteria. All construction, repair, or replacement performed after the upgrade is in accordance with PG&E Quality Assurance Class I requirements.

(s) Piping Symbol G1 applies to the South Site Fire Water System (SSFWS).The SSFWS was not installed under the QA Class R (fire protection system) program.

(t) The “Orig. Design Code & Standards” and the “Orig. Fab., Erection, & Inspection Code” columns list the title and year of the code or standard (e.g., ASME, ANSI, NFPA) that were approved for the design, fabrication, erection, and testing of the fluid systems in the original operating licenses for DCPP Units 1 & 2. Modifications implemented since the original licensing may meet newer codes and standards, which have been accepted through the code reconciliation process or through the NRC approval of License Amendments.

(u) Piping design stress equations are not defined in ANSI B31.1-1967, therefore, stress equations from ANSI B31.1-1973 Summer Addenda apply. Since ANSI B31.7 for PG&E Quality Class II and III piping refers to ANSI B31.1, the same stress equations must be met for piping designed to ANSI B31.7. Furthermore, for PG&E Piping Code Class G, the satisfaction of the given ANSI B31.1 equations is not meant to replace conformance to the applicable NFPA Codes and Standards.

TABLE 3.2-3

CLASSIFICATION OF STRUCTURES, SYSTEMS, AND COMPONENTS

Deleted in Revision 11

The classifications of the DCPP structures, systems, and components of DCPP are contained in the DCPP "Q-List" (Reference 8)

TABLE 3.2-4

PG&E DESIGN CLASS, QUALITY/CODE CLASS, AND PIPING SYMBOL COMPARISON TO ANSI N 18.2; SAFETY GUIDE 26; AND SAFETY GUIDE 29 - Notes (a) and (b)

PG&E CURRENT LICENSING BASIS (CLB) - Notes (b) and (h)		COMPARISON TO NON-CLB STANDARD AND SAFETY GUIDES - Note (b)			
PG&E Design Class	**PG&E Quality/Code Class**	**ANSI N 18.2 Safety Group**	**SG 26 Quality Group**	**SG 29 Category**	**Notes**
I	I	1	A	Category I	Note (c)
I	I	1	A	Category I	-
I	II	2	B	Category I	-
I	II	2	B	Category I	Note (d)
I	III	3	C	Category I	-
I	III	3	C	Category I	-
I	-	-	-	Category I	Note (e)
II	-	NNS	-	Non-Category I	-
II	-	NNS	D	Non-Category I	Note (d), (g)
II	-	-	-	Category I	-
II	-	-	-	Non-Category I	-
II	-	NNS	D	Non-Category I	Note (f)
II	-	NNS	-	Non-Category I	-
II	-	-	-	Non-Category I	-
III	-	-	-	Non-Category I	-

Notes:

(a) American Nuclear Society, Standard No. ANSI N18.2, "Nuclear Safety Criteria for the Design of Stationary Pressurized Water Reactor Plants," draft August 1970 (Reference 1); Atomic Energy Commission, Safety Guide 26, "Quality Group Classifications and Standards for Water, Steam, and Radioactive Waste Containing Components of Nuclear Power Plants," March 1972 (Reference 2), and Atomic Energy Commission, Safety Guide 29, "Seismic Design Classification," June 1972 (Reference 3).

TABLE 3.2-4

(b) It is recognized that during the design and construction of DCPP Units 1 and 2, significant industry and regulatory changes were made in establishing common methods of classification; e.g., ANSI N18.2, Draft August 1970 (Reference 1), Safety Guide 26, March 1972 (Reference 2), Safety Guide 29, June 1972, and NRC Regulatory Guide 1.143, Rev. 1, October 1979 (Reference 6). However, these methods all differ slightly in detail from those used for the DCPP. Although the form and intent of these NRC documents are similar, these are not the DCPP Licensing Basis.

(c) Reactor coolant loop and pressurizer surge line piping.

(d) Feedwater piping from the (final) main feedwater check valve to the steam generator; auxiliary feedwater from the main feedwater line back to the second check valve; main steam piping from the steam generator to the main steam isolation valve; steam generator blowdown piping from the steam generator to the first valve outside containment.

(e) Piping originally installed as Design Class II, but has been upgraded to require seismic qualification for the Hosgri earthquake.

(f) Radioactive system (PG&E QA Class R).

(g) Piping is seismically qualified for the Design Earthquake.

(h) Refer to Table 3.2-2 for Piping Symbols

Table 3.3-1

COMPARISON OF AUXILIARY BUILDING WIND PRESSURE VALUES, UNIFORM BUILDING CODE, AND ASCE PAPER 3269

Height Above Ground Surface[a]	UBC Wind Pressure[c]	Dynamic Pressure[d] q	Dynamic Pressure[d] 1.3 q
30	40.0	19.8	25.8
50	40.0	25.7	33.4
70	40.0	30.5	39.7
90	40.0	34.7	45.1
100	40.0	36.6	47.6
150	40.0	45.0	NA[b]
200	40.0	52.1	NA[b]
212	40.0	53.7	NA[b]

(a) Ground surface (plant grade) is 85 feet above mean sea level (MSL).

(b) Not applicable because the highest part of the auxiliary building is 105 feet above the ground surface.

(c) UBC Pressure is based on 1967 Edition, considering the site to be in a 25 psf zone instead of the code prescribed 20 psf zone and considering the ground surface to be at MSL instead of the actual ground surface at the structure.

(d) Dynamic pressure is based on ASCE paper #3269, considering a design wind speed of 80 mph and a gust factor of 1.1.

TABLE 3.3-2

TORNADO RESISTING CAPABILITY OF STRUCTURES, SYSTEMS AND COMPONENTS

Structure or No. Component	SAFE WIND VELOCITY: More than 300 mph – Wind	More than 300 mph – Wind & Missile	300 mph – Wind	300 mph – Wind & Missile	250 mph – Wind	250 mph – Wind & Missile	200 mph – Wind	200 mph – Wind & Missile	175 mph – Wind	175 mph – Wind & Missile	Less than 175 mph[a] – Wind	Less than 175 mph[a] – Wind & Missile
1. Auxiliary Bldg.												
(a) Reinforced concrete between El. 85' & 165'	X					275						
(b) Battery room ventilation system fan room					X			240				
(c) Control room	X					275						
(d) Doors & louvers											X	
(e) Component cooling water surge tank					X			X				
(f) Auxiliary building supply fan room equipment											X	
(g) Control room ventilation equipment											X	
(h) Fuel handling building supply fan room equipment											X	
2. Fuel Handling Area												
(a) Steel frame[b]					260							127
(b) Roof purlins									X			
(c) Girts, siding, roofing											X	

TABLE 3.3-2

Structure or No. Component	SAFE WIND VELOCITY: More than 300 mph Wind	More than 300 mph Wind & Missile	300 mph Wind	300 mph Wind & Missile	250 mph Wind	250 mph Wind & Missile	200 mph Wind	200 mph Wind & Missile	175 mph Wind	175 mph Wind & Missile	Less than 175 mph[(a)] Wind	Less than 175 mph[(a)] Wind & Missile
(d) Doors & Louvers											X	
(e) Fuel handling building ventilation equipment											X	
3. Containment												
(a) Reinforced concrete	X					275						
(b) Equipt., personnel & escape hatches			X			272						
(c) Exhaust vent											125	
(d) Exterior Class I raceway and instruments											X	
(e) Main steam and feedwater piping	X							X				
(f) Auxiliary feedwater piping			X									
(g) Pipe penetrations			X			275						
(h) Pipeway							X					
4. Turbine Bldg.												
(a) Steel frame[(b)]							225			X		
(b) Exterior concrete walls:												
12" thick							X	X				
24" thick	X					270						
(c) Girts, louvers siding, purlins												

Structure or No. Component	SAFE WIND VELOCITY: More than 300 mph		300 mph		250 mph		200 mph		175 mph		Less than 175 mph[a]	
	Wind	Wind & Missile	Wind	Wind & Missile	Wind	Wind & Missile	Wind	Wind & Missile	Wind	Wind & Missile	Wind	Wind & Missile
(d) Siding in 4.16-kV swgr. and cable spreading room areas							X					
(e) 4.16-kV switchgear/cable spreading room HVAC system											X	
5. Outdoor Tanks												
(a) Condensate storage[b]			X									150
(b) Refueling water storage[b]			X									150
(c) Fire water and transfer storage[b]			X									150
6. Intake Structure												
(a) Reinforced concrete incl. aux. Saltwater pumps compartment	X							240				
(b) ASW pump room shaft extensions								240				
7. Miscellaneous Design Class I Piping												
(a) CCW piping to SG blowdown tank[b]											X	
(b) Containment hydrogen purge lines[b]											X	

TABLE 3.3-2

Structure or No. Component	SAFE WIND VELOCITY: More than 300 mph		300 mph		250 mph		200 mph		175 mph		Less than 175 mph[a]	
	Wind	Wind & Missile	Wind	Wind & Missile	Wind	Wind & Missile	Wind	Wind & Missile	Wind	Wind & Missile	Wind	Wind & Missile
8. 480-V Switchgear and 125-Vdc Inverter Room Ventilation System											X	
9. Control Room Pressurization System[b]											X	

Notes:

[a] Structures, systems and components with an “X” in the “Less than 175 safewind velocity” typically do not have quantified analyses of tornado wind and missile resisting capability.

[b] A safe-wind velocity of combined wind and missile of less than 200 mph is acceptable for this SSC.

TABLE 3.3-3

TORNADO FAILURE ANALYSIS - COMPONENT COOLING WATER SURGE TANK AND RELATED INSTRUMENTATION

Component & Failure	Consequences
(1) Loss of vital conduit to control room	(a) Loss of liquid level indication and annunciators in control room. Any makeup to CCWS still indicated by annunciator on supply valves (LCV-69 and LCV-70).
	(b) Loss of automatic and manual actuation to, and position indication from vent valve (RCV-16). Valve fails closed and any overflow from CCWS discharges through relief valve to auxiliary building sump.
(2) Loss of control air to makeup valve (LCV-69 or LCV-70)	(a) One or both of the normally closed valves fail closed.
	(b) Makeup is not required during a tornado event and CCW system is maintained in normal operating mode.
	(c) Manual makeup bypass valves may be used.
(3) Loss of air supply to level controllers (LC-59 or LC-60) and control air to vent valve (RCV-16)	(a) One or both of the normally closed makeup valves (LCV-69 and LCV-70) fail closed.
	(b) Normally open RCV-16 fails closed.
	(c) Makeup is not required during a tornado event and CCW system is maintained in normal operating mode.
	(d) Tank overpressure protection is maintained by surge tank relief valve (RV-45).
(4) Loss of level instrumentation (one set)	(a) Makeup valve fails closed. Makeup to CCWS through redundant valve, which annunciates in the control room.
	(b) Loss of liquid level indication and annunciator on one compartment of CCWS surge tank.
	(c) Any leakage from instrument taps close to tank flows to the auxiliary building sump; other breaks leak to roof drains.
	(d) Operate on redundant system. Manually isolate compartment with damaged instrumentation, if possible.
	(e) Potential loss of N_2 pressurization - see item (10).

TABLE 3.3-3

Component & Failure	Consequences
(5) Break in vent line, isolation valve or back-pressure regulator	(a) A break between the back-pressure regulator and vent isolation valve will cause surge tank pressure to decrease and the low surge tank pressure alarm to annunciate in the control room. This break can be isolated from the surge tank by closing the vent isolation valve.
	(b) A break upstream of the vent isolation valve will cause surge tank pressure to decrease and the low surge tank pressure alarm to annunciate in the control room. The CCW system will continue to operate, but surge tank pressure must be restored to declare the system operable.
(6) Break in relief valve or header	(a) Surge tank pressure will decrease and low surge tank pressure annunciate in the control room. CCW system will continue to operate; surge tank pressure will have to be restored to declare the system operable.
	(b) Pressurization required to mitigate consequences of LOCA. LOCA not postulated simultaneously with a tornado.
(7) Break in one redundant surge line	(a) A maximum of 6,200 gallons of liquid discharged to the yard drains via the 164 ft and 140 ft roof drains.
	(b) Manually isolate that compartment of the surge tank.
	(c) Operate within an action statement with the redundant side of the surge tank and makeup water system until the faulted condition can be repaired.
	(d) Potential loss of N2 pressurization - see item (10).
(8) Break in two redundant Surge Lines	(a) A maximum of 8,100 gallons of liquid discharged to the yard drains via the 164 ft and 140 ft roof drains.
	(b) Technical Specifications require CFCUs (and hence the CCW system) be operable in Modes 1-4. Commence plant shutdown to Mode 5 (cold shutdown) and remain in cold shutdown until both surge lines have been repaired.
(9) Loss of all level instrumentation	(a) Makeup valves fail closed.
	(b) Loss of liquid level indication and annunciators in the control room.

TABLE 3.3-3

Component & Failure	Consequences
	(c) Instrument tap breaks close to tank leak to auxiliary building sump; other breaks leak to roof drains.
	(d) Some inventory probably remains in surge tank. System continues to operate satisfactorily with no makeup required.
	(e) Backup indication of adequate liquid level from pressure measurement at CCW pump discharge with low pressure annunciator in control room.
	(f) Alternate makeup through manual bypass valves around makeup valves.
	(g) Potential loss of N_2 pressurization - see item (10).
(10) Loss of nitrogen/ instrument air pressurization	(a) CCW system continues to function normally; CCW surge tank pressure annunciated in the control room.
	(b) Pressure required to mitigate the consequences of a LOCA; a LOCA is not postulated simultaneously with a tornado.
	(c) Failure of nitrogen/instrument air supply pressure regulators will not cause CCW system pressurization or prevent normal CCW system operation.

TABLE 3.3-4

TORNADO REVIEW - FAILURE ANALYSIS FOR EXPOSED RACEWAYS AND INSTRUMENTATION

Component					
No.	Dwg. No. [(a)]	Description	Normal Mode	Failure Mode	Consequences
FCV-24 FCV-25	3.2-04(31-B) 3.2-04(31-A) (7.3.42)	SG 1 and 2 main steam isolation bypass valves (redundant solenoid valves)	Close	Close	None - not involved in plant shutdown
FCV-37	3.2-04(31-C) (7.3-18)	SG 2 steam supply valve turbine-driven AFW pump	Open	In place	None - backed up by FCV-38 coming from main steam line from SG 3
FCV-41 FCV-42	3.2-04(31-A) 3.2-04(31-B) (7.3-42)	SG 1 and 2 main steam isolation valves (redundant solenoid valves)	Open	Open	None - not involved in plant shutdown unless there is a steam line break
FCV-438 FCV-439	3.2-03(45-D) 3.2-03(45-D)	SG 1 and 2 main FW isolation valves	Open	In place	None - backed up by FCV-510 and FCV-520 and MFP trip
FCV-510 FCV-520	3.2-03(38-C) 3.2-03(38-D)	SG 1 and 2 main feedwater control valves	Open	Close	Closing causes reactor trip followed by turbine trip
FCV-1510 FCV-1520	3.2-03(38-B) 3.2-03(38-D)	SG 1 and 2 feedwater bypass valves	Close	Close	None
LCV-106 LCV-107 LCV-110 LCV-111	3.2-03(46-C) 3.2-03(46-C) 3.2-03(46-C) 3.2-03(46-B)	SG 1 and 2 supply valves from turbine driven AFW pumps SG 1 and 2 supply valves from motor-driven AFW pumps	Open Open	As-is Open	Simultaneous loss of signal to valves LCV-106, 107, 110, and 111 results in inability to modulate AFW flow and may result in some excess feedwater addition

TABLE 3.3-4

Component					
No.	Dwg. No. [a]	Description	Normal Mode	Failure Mode	Consequences
PCV-19 PCV-20 PCV-21 PCV-22		10% atmospheric steam dump valves	Close	Close	Backed up by main steam safety valves
FT-50 FT-77	3.2-03(48-C) 3.2-03(48-C)	AFW lead 1 and 2 flow transmitters			Other indication available to monitor SG level; backup - shutdown with SGs 3 and 4
FT-510 FT-511 FT-520 FT-521	3.2-03(39-C) (7.2-1,Sh.7) 3.2-03(39-C)	SG 1 and 2 main feedwater flow transmitters (redundant)			None - if already switched to AFW by loss of all offsite power
PT-514 PT-515 PT-516 PT-524 PT-525 PT-526	3.2-04(31-A) 3.2-04(31-B)	SG 1 and 2 steam pressure transmitters			Loss of two pressure transmitters on either loop to initiate safety injection signal and steam line isolation signal
PT-516A PT-526A	3.2-04(31-A) (7.2-1,Sh.7) 3.2-04(31-B)	SG 1 and 2 steam pressure transmitters for 10% steam dump logic			Loss of transmitters causes loss of automatic control of the atmospheric dump valve; backed up by main steam safety valve

(a) 3.2-04 denotes P&ID; (31-A) is grid location of P&ID; (7.3-42) denotes instrumentation logic diagram common to SG 1 and 2 components

TABLE 3.3-5

SUMMARY OF THE VELOCITY CHARACTERISTICS OF ADDITIONAL
TORNADO-BORNE MISSILES FOR 250 MPH TORNADO WIND VELOCITY

	Input Data						Result				
Additional Tornado-borne Missile	C_l	C_d	A_p	ρ_m	W	Ejection Velocity, fps	Injection Height, ft	Missile Suspension ft	Modified Missile Velocity, fps	Design Missile Velocity, fps	Maximum Missile Elevation, ft
Plank 4 x 12 x 12 ft ρ = 50 lb/ft^3	0.5	1.0	12.0	1.55	200	247	14	Yes	-	247	68
Utility Pole 13.5 in. ϕ x 35 ft lg. ρ = 43 lb/ft^3	0.7	0.37	39.4	1.34	1500	144	5	No	65	65	5
1 in. ϕ Solid$_{\text{Stl. Rod}}$ 3 ft Lg. ρ = 490 lb/ft^2	0.7	1.12	0.25	15.20	8	203	7	Yes	-	203	92
6 in. ϕ Pipe Sch. 40, 15 ft lg. ρ = 490 lb/ft^3	0.7	0.37	8.29	2.47	285	149	6	No	84	84	6
12" ϕ Pipe Sch. 40, 15 ft lg. ρ = 490 lb/ft^3	0.7	0.38	15.9	1.74	744	134	3	No	42	42	3
3 in. ϕ Pipe Sch. 40, 15 ft lg. ρ = 490 lb/ft3	0.7	0.37	4.38	3.54	114	165	11	Yes		165	24
4000# Auto 1.67 x 6 x 17 ft	0.7	0.70	102	0.73	4000	165	5	Yes		165	37

TABLE 3.3-5

LEGEND:

C_1 = Lift coefficient (dimensionless)
C_d = Drag coefficient (dimensionless)
A_p = Maximum projected area of the missile (ft^2)
ρ_m = $W/g \times 1/V_1 = \rho/g$ ($slug/ft^3$)
W = Total weight of the missile (lb)
V_1 = Total volume of the missile (ft^3)
g = 32.2 ft/sec^2

TABLE 3.3-6

REQUIRED THICKNESS OF A REINFORCED CONCRETE MISSILE BARRIER
TO PRECLUDE MISSILE PERFORATION OR THE CREATION OF SECONDARY MISSILES

	Input Data For 250 MPH Tornado			Results (Inches)			
Additional Tornado-Borne Missile	Missile Velocity, ft/sec	Weight of Missile, lbs	Min. Impact Area, ft^2	Penetration Into An Infinite Thick Conc.	Just Perforated Thickness	Thickness Without Creating Secondary Missiles	Min. Design Thickness, in.
Plank	247	200	0.333	2.7	5.5	8.2	9
Utility Pole	65	1500	0.93	0.6	1.2	1.7	6
1 in. ϕ Solid Stl Rod	203	8	0.00545	4.7	9.4	14.1	15
6 in. ϕ Pipe	84	285	0.239	0.7	1.4	2.1	6
12 in. ϕ Pipe	42	744	0.885	0.1	0.3	0.4	6
3 in. ϕ Pipe	165	114	0.0667	3.7	7.4	11.2	12
4000 lb Auto	165	4000	10.0	0.9	1.7	2.6	6

TABLE 3.5-2

CONTROL ROD DRIVE SHAFT - MISSILE CHARACTERISTICS

Diameter = 1.75 inches
Length = 300 inches
Weight = 120 pounds

Drive Shaft Travel Outside Housing[(a)] ft	Drive Shaft Velocity ft/sec	Drive Shaft Kinetic Energy ft-lb
1	151	42,900
2	162	49,000
3	171	55,000
4	179	60,200
5	189	66,500

(a) Distance from top of rod travel housing to bottom of missile shield

TABLE 3.5-3

CONTROL ROD DRIVE SHAFT AND MECHANISM - MISSILE CHARACTERISTICS

Missile weight: 1500 pounds
Impact OD: 3.75 inches

Travel ft	Drive Shaft Velocity ft/sec	Drive Shaft Kinetic Energy ft-lb
1	14.3	4,600
2	20.2	9,200
3	24.8	13,800
4	28.6	18,400
5	32.0	23,000

TABLE 3.5-4

VALVE - MISSILE CHARACTERISTICS

Missile Description	Weight lb	Flow Discharge Area, in^2	Thrust Area, in^2	Impact Area, in^2	Wt. to Imp. Area Ratio	Velocity fps
Safety relief valve bonnet, (3 in. x 6 in. x 6 in.)	350	2.86	80	24	14.6	110
3 in. motor-operated isolation valve bonnet (plus motor and stem) (3 in.)	400	5.5	113	28	14.1	135
2 in. air-operated relief valve bonnet (plus stem)	75	1.8	20	20	3.75	115
3 in. air-operated spray valve bonnet (plus stem)	120	5.5	50	50	2.4	190
4 in. air-operated spray valve bonnet	200	9.3	50	50	4.0	190

TABLE 3.5-5

PIPING TEMPERATURE ELEMENT ASSEMBLY - MISSILE CHARACTERISTICS

1. For a radial fracture around the weld between the thermowell (well) and the pipe:

Characteristics	"without well"	"with well"
Flow discharge area, in^2	0.11	0.60
Thrust area, in^2	7.1	9.6
Missile weight, lb	11.0	15.2
Area of impact, in^2	3.14	3.14
$\left(\frac{\text{Missile weight}}{\text{Impact area}}\right)$, psi	3.5	4.84
Velocity, fps	20.0	120.0

2. For a radial fracture at the junction between the temperature element assembly and the thermowell for the "without well" element and at the junction between the thermowell and the well for the "with well" element:

Characteristics	"without well"	"with well"
Flow discharge area, in^2	0.11	0.60
Thrust area, in^2	3.14	3.14
Missile weight, lb	4.0	6.1
Area of impact, in^2	3.14	3.14
$\left(\frac{\text{Missile weight}}{\text{Impact area}}\right)$, psi	1.27	1.94
Velocity, fps	75.0	120.0

TABLE 3.5-6

CHARACTERISTICS OF OTHER MISSILES
POSTULATED WITHIN REACTOR CONTAINMENT

	Reactor Coolant Pump Temperature Element	Instrument Well of Pressurizer	Pressurizer Heaters
Weight, lb	0.25	5.5	15.0
Discharge area, in^2	0.50	0.442	0.80
Thrust area, in^2	0.50	1.35	2.4
Impact area, in^2	0.50	1.35	2.4
$\left(\frac{\text{Missile weight}}{\text{Impact area}}\right)$, psi	0.5	4.1	6.25
Velocity, fps	260.0	100.0	55.0

TABLE 3.6-1

CHECKLIST OF DYNAMIC EFFECTS FROM POSTULATED RUPTURE OF PIPE CONNECTED TO THE REACTOR COOLANT SYSTEM

Large Break (>4 in. ID) Resulting in a Loss of Coolant	Location Description	Loop	Size in.	Is the Contain-ment Liner Protected?	Is Break Propagation Prevented and Low-head Safety Injection Maintained to the Un-affected Loops?	Is Break Propagation in the Affected Loop Limited to 20%?	Is Damage to Steam System Prevented?						Is the Integrity of Supports Maintained?		Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?
							Steam Line	Feed Line	Aux Feed	Blow-down	Level Taps	Sample Line	SG	Pump	
Pressurizer surge (16)[a]	Hot leg to pressurizer	2	14	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Accumulator injection (253)	Check valve 8948A to cold leg	1	10	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Accumulator injection (254)	Check valve 8948B to cold leg	2	10	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Accumulator injection (255)	Check valve 8948C to cold leg	3	10	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Accumulator injection (256)	Check valve 8948D to cold leg	4	10	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Residual heat removal supply (109)	Hot leg to valve 8702	4	14	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Low-head safety injection (235)	Loop to check valve	1	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

TABLE 3.6-1

Large Break (>4 in. ID) Resulting in a Loss of Coolant	Location Description	Loop	Size in.	Is the Containment Liner Protected?	Is Break Propagation Prevented and Low-head Safety Injection Maintained to the Unaffected Loops?	Is Break Propagation in the Affected Loop Limited to 20%?	Is Damage to Steam System Prevented? Steam Line	Is Damage to Steam System Prevented? Feed Line	Is Damage to Steam System Prevented? Aux Feed	Is Damage to Steam System Prevented? Blowdown	Is Damage to Steam System Prevented? Level Taps	Is Damage to Steam System Prevented? Sample Line	Is the Integrity of Supports Maintained? SG	Is the Integrity of Supports Maintained? Pump	Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?
Low-head safety injection (236)	Loop to check valve	2	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Low-head safety injection (237)	Loop to check valve	3	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Low-head safety injection (238)	Loop to check valve	4	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Pressurizer relief line (730)	Pressurizer to 1171 takeoff	-	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(b)
Pressurizer safety lines (727, 728, 729)	Pressurizer to safety valves	-	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(b)
Accumulator injection (253, 1294)	Accumulator to check valve 8948A to cold leg	1	10	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Accumulator injection (254, 1295)	Accumulator to check valve 8938B to cold leg	2	10	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

TABLE 3.6-1

Large Break (>4 in. ID) Resulting in a Loss of Coolant	Location Description	Loop	Size in.	Is the Containment Liner Protected?	Is Break Propagation Prevented and Low-head Safety Injection Maintained to the Unaffected Loops?	Is Break Propagation in the Affected Loop Limited to 20%?	Is Damage to Steam System Prevented? Steam Line	Feed Line	Aux Feed	Blowdown	Level Taps	Sample Line	Is the Integrity of Supports Maintained? SG	Pump	Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?
Accumulator injection (255, 1296)	Accumulator to check valve 8948C to cold leg	3	10	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Loop 4 only	Yes
Accumulator injection (256, 1297)	Accumulator to check valve 8948D to cold leg	4	10	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Loop 3 only	Yes
Residual heat removal supply (109, 927)	Valve 8702 to cont. pen. 27	4	14	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Residual heat removal return (2576, 120)	Accumulator disch. to cont. pen. 26	3	8, 14	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	(e)
Residual heat removal return (2575, 120)	Accumulator disch. to cont. pen. 26	4	8, 12	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	(e)
Low-head safety injection (235)	Loop isol. valve to cont. pen. 24	-	6	Yes	-	-	-	-	-	-	-	-	-	-	(c)
Low-head safety injection (236)	Loop isol. valve to cont. pen. 24	-	6	Yes	-	-	-	-	-	-	-	-	-	-	(c)

TABLE 3.6-1

arge Break (>4 in. ID) Resulting in a Loss of Coolant	Location Description	Loop	Size in.	Is the Contain-ment Liner Protected?	Is Break Propagation Prevented and Low-head Safety Injection Maintained to the Un-affected Loops?	Is Break Propagation in the Affected Loop Limited to 20%?	Is Damage to Steam System Prevented?						Is the Integrity of Supports Maintained?		Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?
							Steam Line	Feed Line	Aux Feed	Blow-down	Level Taps	Sample Line	SG	Pump	
Low-head safety injection (237)	Loop isol. valve to cont. pen. 25	-	6	Yes	-	-	-	-	-	-	-	-	-	-	(c)
Low-head safety injection (238)	Loop isol. valve to cont. pen. 25	-	6	Yes	-	-	-	-	-	-	-	-	-	-	(c)
Pressurizer relief line (17)	From 4x6 to header (23)	-	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(d)
Pressurizer safety line (19, 20, 21)	Safety valve to header (23)	-	6	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(d)
Pressurizer spray (12, 15)	Cold Leg to pressurizer	1	4	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Pressurizer spray (14)	Cold leg to pressurizer	2	4	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

TABLE 3.6-1

Large Break (>4 in. ID) Resulting in a Loss of Coolant	Location Description	Loop	Size in.	Is the Contain-ment Liner Protected?	Is Break Propagation Prevented and Low-head Safety Injection Maintained to the Un-affected Loops?	Is Break Propagation in the Affected Loop Limited to 20%?	Is Damage to Steam System Prevented?						Is the Integrity of Supports Maintained?		Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?
							Steam Line	Feed Line	Aux Feed	Blow-down	Level Taps	Sample Line	SG	Pump	
Charging (50)	Check valve 8379A to RC piping	3	3	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Loop 4 only	Yes
Charging (246)	Check valve 8379B to RC piping	4	3	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Loop 3 only	Yes
Auxiliary spray (51)	Check valve 8377 to line (15)	-	2	Yes	Yes	-	-	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)
RC pump seal water injection (54)	Check valve 8368 to RC pump	1	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)
RC pump seal water injection (55)	Check valve 8368 to RC pump	2	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)
RC pump seal water injection (58)	Check valve 8368 to RC pump	3	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)

TABLE 3.6-1

Large Break (>4 in. ID) Resulting in a Loss of Coolant	Location Description	Loop	Size in.	Is the Containment Liner Protected?	Is Break Propagation Prevented and Low-head Safety Injection Maintained to the Unaffected Loops?	Is Break Propagation in the Affected Loop Limited to 20%?	Is Damage to Steam System Prevented? Steam Line	Feed Line	Aux Feed	Blowdown	Level Taps	Sample Line	Is the Integrity of Supports Maintained? SG	Pump	Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?
RC pump seal water injection (57)	Check valve 8368 to RC pump	4	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)
Letdown (24)	RC piping to to support downstream of valve LCV 460	2	3	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)
Excess letdown (63)	RC piping to to support downstream of valve LCV 8167	2	1	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)
RC seal vent (1495)	Pump to valve	-	-	-	-	-	-	-	-	-	-	-	-	-	(d)
RC seal vent (1496)	Pump to valve	-	-	-	-	-	-	-	-	-	-	-	-	-	(d)
RC seal vent (1497)	Pump to valve	-	-	-	-	-	-	-	-	-	-	-	-	-	(d)
RC seal vent (1498)	Pump to valve	-	-	-	-	-	-	-	-	-	-	-	-	-	(d)
RC leakoff (58)		-	-	-	-	-	-	-	-	-	-	-	-	-	(d)

TABLE 3.6-1

Large Break (>4 in. ID) Resulting in a Loss of Coolant	Location Description	Loop	Size in.	Is the Containment Liner Protected?	Is Break Propagation Prevented and Low-head Safety Injection Maintained to the Unaffected Loops?	Is Break Propagation in the Affected Loop Limited to 20%?	Is Damage to Steam System Prevented? Steam Line	Feed Line	Aux Feed	Blow-down	Level Taps	Sample Line	Is the Integrity of Supports Maintained? SG	Pump	Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?
RC leakoff (59)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	(d)
RC leakoff (60)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	(d)
RC leakoff (61)	-	-	-	-	-	-	-	-	-	-	-	-	-	-	(d)
Drain 958	RC piping downstream of valve	1	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)
Drain 959	RC piping downstream of valve	2	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)
Drain 960	RC piping downstream of valve	3	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)
Drain 961	RC piping downstream of valve	4	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)
Pressurizer relief lines (1171, 1172, 1195)	1171 take off to valve (N.C.)	3	2	Yes	-	-	-	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Charging high-head SIS (50, 49)	Upstream of check valve 8379A	3	3	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)

TABLE 3.6-1

Large Break (>4 in. ID) Resulting in a Loss of Coolant	Location Description	Loop	Size in.	Is the Containment Liner Protected?	Is Break Propagation Prevented and Low-head Safety Injection Maintained to the Unaffected Loops?	Is Break Propagation in the Affected Loop Limited to 20%?	Is Damage to Steam System Prevented? Steam Line	Feed Line	Aux Feed	Blow-down	Level Taps	Sample Line	Is the Integrity of Supports Maintained? SG	Pump	Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?
Charging high-head SIS (246)	Upstream of check valve 8379B	4	3	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(e)
Auxiliary spray (51)	Charging line to isolation valve	-	2	-	-	-	-	-	-	-	-	-	-	-	(d)
RC pump seal water injection (54)	Upstream of check valve	1	-	-	-	-	-	-	-	-	-	-	-	-	(d)
RC pump seal water injection (55)	Upstream of check valve	2	-	-	-	-	-	-	-	-	-	-	-	-	(d)
RC pump seal water injection (56)	Upstream of check valve	3	-	-	-	-	-	-	-	-	-	-	-	-	(d)
RC Pump seal water injection (57)	Upstream of check valve	4	-	-	-	-	-	-	-	-	-	-	-	-	(d)
Letdown (24)	Beyond restraint downstream of isolation valve	2	3	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

TABLE 3.6-1

Large Break (>4 in. ID) Resulting in a Loss of Coolant	Location Description	Loop	Size in.	Is the Contain-ment Liner Protected?	Is Break Propagation Prevented and Low-head Safety Injection Maintained to the Un-affected Loops?	Is Break Propagation in the Affected Loop Limited to 20%?	Is Damage to Steam System Prevented? Steam Line	Feed Line	Aux Feed	Blow-down	Level Taps	Sample Line	Is the Integrity of Supports Maintained? SG	Pump	Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?
Excess letdown (63)	Beyond re-straint downstream of isolation valve	2	1	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(f)
Pressurizer relief lines (1171, 1172, 1195)	Downstream of valves (N.C.)	-	3	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

(a) PG&E line number.

(b) Affected area limited by enclosure.

(c) Operates only during the injection and recirculation phase following a LOCA. Rupture not postulated.

(d) Due to pressure and flow conditions in these pipes during operation of these lines, whipping is not assumed to occur.

(e) Whipping allowed with no services affected.

(f) Whipping allowable with no services affected with restraints at valves to prevent LOCA.

TABLE 3.6-2

CHECKLIST OF DYNAMIC EFFECTS OF OTHER POSTULATED PIPE RUPTURES INSIDE THE CONTAINMENT

Steam System Break Not Resulting in a Loss of Coolant	Location Description	Loop	Size	Is Loss of Coolant Prevented From Occurring?	Is Break Propagation Prevented to Steam Piping in the Unaffected Loops?	Is Safety Injection Maintained to All RC Loops?	Is the Contain-ment Liner Protected?	Is Inte-grity of the Steam Generator Maintained?	Is Boration Capability Maintained?	Is Cold Shutdown Capability Maintained?	Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?
Main steam line (225)[(a)]	From steam generator nozzle to containment penetration	4	28	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Main steam line (226)	From steam generator nozzle to containment penetration	3	28	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Main steam line (227)	From steam generator nozzle to containment penetration	2	28	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Main steam line (228)	From steam generator nozzle to containment penetration	1	28	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Feedwater line (554)	From steam generator nozzle to containment penetration	1	16	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Feedwater line (555)	From steam generator nozzle to containment penetration	2	16	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

TABLE 3.6-2

Steam System Break Not Resulting in a Loss of Coolant	Location Description	Loop	Size	Is Loss of Coolant Prevented From Occurring?	Is Break Propagation Prevented to Steam Piping in the Unaffected Loops?	Is Safety Injection Maintained to All RC Loops?	Is the Contain-ment Liner Protected?	Is Inte-grity of the Steam Generator Maintained?	Is Boration Capability Maintained?	Is Cold Shutdown Capability Maintained?	Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?
Feedwater line (556)	From steam generator nozzle to containment penetration	4	16	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Feedwater line (557)	From steam generator nozzle to containment penetration	3	16	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Steam generator blowdown line (1059)	From steam generator nozzle to containment penetration	1	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(b)
Steam generator blowdown line (1060)	From steam generator nozzle to containment penetration	2	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(b)
Steam generator blowdown line (1061)	From steam generator nozzle to containment penetration	3	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(b)
Steam generator blowdown line (1062)	From steam generator nozzle to containment penetration	4	2	Yes	Yes	Yes	Yes	Yes	Yes	Yes	(b)

TABLE 3.6-2

Steam System Break Not Resulting in a Loss of Coolant	Location Description	Loop	Size	Is Loss of Coolant Prevented From Occurring?	Is Break Propagation Prevented to Steam Piping in the Unaffected Loops?	Is Safety Injection Maintained to All RC Loops?	Is the Contain-ment Liner Protected?	Is Inte-grity of the Steam Generator Maintained?	Is Boration Capability Maintained?	Is Cold Shutdown Capability Maintained?	Is Line Restrained to Meet Criteria with Regulatory Guide 1.46 as a Minimum?

(a) PG&E line number.
(b) In case of pipe rupture, pipe is allowed to whip with no services affected.

TABLE 3.6-6

PIPE BREAK PROTECTION FEATURES ON UNIT 2 DIFFERENT FROM UNIT 1

Area	Description	Protection Feature Used
Turbine	Reheater drain system has different configuration, due to piping layout, causing different line numbers to appear for Unit 2.	Protection features used were the same as for Unit 1. The same vital structures, equipment, etc., were protected from pipe whip and jet impingement.
GE/GW	Cables and conduits servicing vital equipment were routed below the floor at el. 115 feet and returned through floor penetrations.	Protection was accomplished through separation.
GE/GW	Elimination of containment barrier plates, two on MS and two on FW.	Vital conduits affected by jet impingement were rerouted.
GE/GW	Elimination of impingement barrier surrounding the main steam line riser pipes.	Vital conduits were rerouted, vital equipment was relocated away from the jet impingement.
GE/GW	Elimination of impingement sleeve end barriers on main steam and feedwater line. Required only on MS nodes 4070 and 3180, and FW node 1403.	Conduits and instrumentation were relocated away from sleeve end jet impingement.
FW	Elimination of sleeve and barriers on main steam and feedwater lines. Required only on MS nodes 1180 and 2187, and FW nodes 1213 and 1113.	Conduits were rerouted away from sleeve end jet impingement.

TABLE 3.7-1

CONTAINMENT AND AUXILIARY BUILDING
CRITERIA COMPARISON

Parameters	HE	DE	DDE
Seismic input, horizontal	0.75g	0.2g	0.4g
Seismic input, vertical	2/3 of horizontal dynamic amplification considered	Static - 2/3 of horizontal ground spectra	Static-2/3 of horizontal ground spectra
Accidental torsion	5% and 7% eccentri-city	Not considered	Not considered
Foundation filtering	Tau = 0.040[(a)]	Not applicable	Not applicable
Response combination	3-D SRSS	2-D ABSUM	2-D ABSUM
Damping values	7%	2% concrete[(b)] 2% steel	5% concrete 2% steel
Ductility	Allowed in some areas	Not considered	Not considered
Material properties	Based on test values	Min specified values	Min specified values
Response spectra broadening (based on frequency)	+5%, -15%	±10%	±10%
Response spectra peaks clip		10% (for containment only)	10% (for containment only)

(a) 0.052 for auxiliary building.

(b) 5% for auxiliary building.

TABLE 3.7-1A

TURBINE BUILDING
CRITERIA COMPARISON

Parameters	HE	DE and DDE[(a)]
Seismic input, horizontal	0.75g	0.2g (DE) 0.4g (DDE)
Seismic input, vertical	2/3 of horizontal dynamic amplification Considered	Static - 2/3 of horizontal ground spectra
Accidental torsion	5% and 7% eccentricity, or equivalent Note (b)	Not considered
Foundation filtering	Tau = 0.080 (Blume input) Tau = 0.067 (Newmark input)	Not applicable
Response combination	3-D SRSS	2-D ABSUM
Damping values	7%	5% concrete 2% steel
Ductility	Concrete 1.3 Steel 3 (6 locally)	Not considered
Material properties	Based on test values	Min specified values
Response spectra broadening (based on frequency)	+5%, -15%	±10%
Response spectra peaks clip		10%

(a) DE and DDE analysis is performed only to generate response spectra for systems qualification.

(b) Equivalent method is used as described in Section 3.7.2.10.

TABLE 3.7-1B

INTAKE STRUCTURE
CRITERIA COMPARISON

Parameters	Hosgri	DE and DDE for Systems Qualifications Only
Seismic input, horizontal	Hosgri 7.5M	DE (0.20g) DDE (0.40g)
Seismic input, vertical	2/3 of horizontal spectra with Tau = 0.0 Dynamic amplification Considered	Static 2/3 of ground horizontal spectra
Accidental torsion	Horizontal floor response spectra increased by 10%	Not considered
Foundation filtering	Tau - 0.04	Not applicable
Response combination	3-D-SRSS	Not applicable
Damping values % critical	7%	5%
Ductility	Concrete 1.3; Steel 3, with up to 6 locally[(a)]	Not considered
Material properties	Based on test values	Minimum specified values
Floor response spectra broadening (based on frequency)	+5%, -15%	Structural peaks clipped 10% and widened by ± 10%

____

(a) Or as may be required to demonstrate that function of Design Class I equipment will not be adversely affected.

TABLE 3.7-1C

OUTDOOR STORAGE TANKS
CRITERIA COMPARISON

Parameters	Hosgri	DE and DDE
Seismic input, horizontal	Hosgri 7.5M	DE (0.20g) DDE (0.40g)
Seismic input, vertical	2/3 ZPA (0.75g) of horizontal spectra with Amplification considered Tau = 0.0	Static 2/3 ZPA of horizontal ground spectra
Accidental torsion	Not applicable	Not applicable
Response combination	3-D SRSS	2-D ABSUM
Damping	7%-All tanks with concrete cover	5%-All tanks with concrete cover
	4%-Firewater tank without concrete cover	1%-Firewater tank without concrete cover
Material properties	Based on test values	Minimum specified values

TABLE 3.7-2

CONTAINMENT STRUCTURE
PERIODS OF VIBRATION

Mode No.	Period, T, in sec
1	0.255
2	0.093
3	0.088
4	0.073
5	0.060
6	0.058
7	0.057
8	0.051
9	0.051

Table 3.7-3

CONTAINMENT STRUCTURE
MAXIMUM ABSOLUTE ACCELERATIONS

Structure	Nodal Point[a]	Elevation, ft	Maximum Absolute Acceleration, g DE Analysis	DDE Analysis
Exterior structure	2	301.64	1.275	2.083
	8	274.37	1.032	1.736
	10	258.27	0.907	1.567
	14	231.00	0.743	1.177
	17	205.58	0.837	1.358
	23	181.08	0.911	1.369
	26	155.83	0.866	1.292
	34	130.58	0.713	1.080
	37	109.67	0.492	0.793
Interior structure	19-22	140.00	0.735	1.195
	24	127.00	0.597	0.982
	27-30	114.00	0.478	0.773
	32	110.00	0.455	0.726
	38	102.00	0.384	0.601
Base slab	47-58	88.58	0.291	0.483

(a) See Figure 3.7-5.

Table 3.7-4

CONTAINMENT STRUCTURE
MAXIMUM DISPLACEMENTS

Structure	Nodal Point[(a)]	Elevation, ft	Maximum Displacement Inches DE Analysis	DDE Analysis
Exterior structure	2	301.64	0.666	1.063
	8	274.37	0.602	0.967
	10	258.27	0.562	0.911
	14	231.00	0.480	0.807
	17	205.58	0.389	0.695
	23	181.08	0.314	0.587
	26	155.83	0.248	0.459
	34	130.58	0.180	0.327
	37	109.67	0.115	0.212
Interior structure	19-22	140.00	0.083	0.139
	24	127.00	0.069	0.114
	27-30	114.00	0.056	0.090
	32	110.00	0.053	0.084
	38	102.00	0.043	0.068
Base slab	47-58	88.58	0.030	0.050

(a) See Figure 3.7-5.

Table 3.7-5

CONTAINMENT STRUCTURE
MAXIMUM SHELL FORCES AND MOMENTS[(a)] - DE ANALYSIS

Nodal[(b)] Point	Elevation, ft	Shell Moments, kip-ft/ft			Shell Forces, kips/ft		
		M_{SS}	M_{TT}	M_{ST}	F_{SS}	F_{TT}	F_{ST}
2	301.64	0.21	0.21	28.99	2.74	3.84	3.75
8	274.37	0.33	0.44	2.96	14.47	32.07	23.85
10	258.27	1.76	0.91	1.63	21.04	40.91	32.80
14	231.00	9.17	2.94	0.36	37.68	42.73	48.97
17	205.58	5.74	1.26	0.27	63.59	33.27	66.44
23	181.08	7.58	2.54	0.31	91.25	37.79	79.59
26	155.83	5.69	1.49	0.50	110.72	36.31	91.43
34	130.58	4.31	1.01	0.27	151.69	31.20	108.65
37	109.67	8.26	2.75	0.19	174.13	18.99	122.66
57	88.58	1.01	0.14	2.23	209.79	63.73	127.22

(a) See Figure 3.7-7.

(b) See Figure 3.7-5.

Table 3.7-6

CONTAINMENT STRUCTURE
MAXIMUM SHELL FORCES AND MOMENTS[a] - DDE ANALYSIS

Nodal[b] Point	Elevation, ft	Shell Moments, kip-ft/ft			Shell Forces, kips/ft		
		M_{SS}	M_{TT}	M_{ST}	F_{SS}	F_{TT}	F_{ST}
2	301.64	0.36	0.37	47.17	4.30	6.33	6.04
8	274.37	0.62	0.76	4.77	22.00	53.37	39.37
10	258.27	2.71	1.46	2.63	32.58	67.71	54.58
14	231.00	15.29	4.92	0.50	60.01	71.93	83.06
17	205.58	8.14	1.64	0.37	103.39	53.31	110.30
23	181.08	11.39	3.96	0.45	154.79	56.72	132.95
26	155.83	8.27	2.21	0.77	190.50	54.24	162.53
34	130.58	6.07	1.36	9.42	251.35	46.24	195.36
37	109.67	15.95	5.31	0.34	282.88	30.75	217.34
57	88.58	1.74	0.23	4.18	338.73	110.90	220.62

(a) See Figure 3.7-7.

(b) See Figure 3.7-5.

Table 3.7-7

CONTAINMENT STRUCTURE
MAXIMUM TOTAL SHEARS

Structure	Associated[a] Nodel Point	Elevation, ft	Maximum Shears, kips x 10^3 DE Analysis	Maximum Shears, kips x 10^3 DDE Analysis
Exterior structure	2	301.64	0.19	0.66
	8	274.37	5.81	9.38
	10	258.27	8.49	13.91
	14	231.00	11.39	19.55
	17	205.58	15.00	25.02
	23	181.08	17.95	29.98
	26	155.83	20.63	36.66
	34	130.58	24.53	44.18
	37	109.67	27.83	49.42
	57	88.58	29.55	51.39
Interior structure	19 & 22	140.00	8.06	13.23
	27 & 30	114.00	10.27	16.87
	49 & 54	88.58	18.85	30.96
Total base shear	49, 54, & 57	88.58	35.05	59.99

(a) See Figure 3.7-5.

Table 3.7-8

CONTAINMENT STRUCTURE
MAXIMUM TOTAL OVERTURNING MOMENTS

Structure	Associated[a] Nodel Point	Elevation, ft	Maximum Overturning Moment kips-ft x 10^6 DE Analysis	Maximum Overturning Moment kips-ft x 10^6 DDE Analysis
Exterior structure	2	301.64	0.00	0.00
	8	274.37	0.12	0.18
	10	258.27	0.27	0.41
	14	231.00	0.61	0.97
	17	205.58	1.03	1.67
	23	181.08	1.48	2.50
	26	155.83	1.79	3.08
	34	130.58	2.45	4.07
	37	109.67	2.82	4.58
	57	88.58	3.39	5.48
Interior structure	19 & 22	140.00	0.06	0.10
	27 & 30	114.00	0.20	0.33
	49 & 54	88.58	0.76	1.24
Total O.T.M. at base	49, 54 & 57	88.58	3.48	5.62

(a) See Figure 3.7-5.

TABLE 3.7-8A

PERIODS OF VIBRATION AND PERCENT PARTICIPATION FACTORS

	Containment Exterior Structure					
	Translational[a] Horizontal Model		Coupled Translation[b] Plus Torsion Model		Vertical Model	
Mode No	Period (sec)	Percent Participation Factor	Period (sec)	Percent Participation Factor	Period (sec)	Participation Factor
1	0.225	44.97	0.217	58.85	0.081	51.72
2	0.081	22.75	0.109	0.32	0.051	16.72
3	0.053	3.73	0.074	25.86	0.046	2.62
4	0.053	7.88	0.041	12.35	0.046	3.81
5	0.047	1.96	0.039	5.63	0.044	11.84
6	0.045	0.043			0.043	0.05
7	0.043	9.91			0.040	7.96
8	0.041	1.66			0.038	3.15
9	0.037	1.70			0.035	1.25
10	0.036	1.71			0.035	0.88
11	0.033	1.18				
12	0.032	2.12				

(a) Axisymmetric model (see Figure 3.7-5A).
(b) Lumped-mass model with 5% accidental eccentricity (see Figure 3.7-5B).

TABLE 3.7-8B

CONTAINMENT EXTERIOR STRUCTURE
MAXIMUM ABSOLUTE HORIZONTAL AND VERTICAL ACCELERATIONS

		Absolute Horizontal Acceleration (g)		Absolute Vertical Acceleration (g)	
Nodal[b] Point	Elevation (ft)	Blume-Hosgri[a] Horizontal Input	Vertical Input	Blume-Hosgri Horizontal Input	Vertical Input
2	301.64	2.21	0.02	0.075	1.600
8	274.37	2.07	0.15	0.450	1.020
10	258.27	1.95	0.28	0.511	0.882
14	231.00	1.70	0.28	0.532	0.810
17	205.58	1.44	0.11	0.475	0.759
19	181.08	1.23	0.14	0.416	0.703
20	155.83	1.00	0.17	0.334	0.633
22	130.58	0.80	0.18	0.228	0.575
23	109.67	0.75	0.16	0.122	0.538

(a) Effective horizontal acceleration at containment shell due to absolute sum of horizontal response and torsional response from 5% eccentricity.

(b) See Figure 3.7-5A.

TABLE 3.7-8C

CONTAINMENT EXTERIOR STRUCTURE
MAXIMUM HORIZONTAL AND VERTICAL DISPLACEMENTS

		Horizontal Displacement (in.)		Vertical Displacement (in.)	
Nodal[a] Point	Elevation (ft)	Blume-Hosgri Horizontal Input[b]	Vertical Input	Blume-Hosgri Horizontal Input	Vertical Input
2	301.64	1.124	0.002	0.032	0.108
8	274.37	1.036	0.009	0.198	0.076
10	258.27	0.969	0.020	0.228	0.066
14	231.00	0.828	0.020	0.240	0.056
17	205.58	0.665	0.008	0.221	0.049
19	181.08	0.534	0.009	0.196	0.041
20	155.83	0.394	0.011	0.158	0.031
22	130.58	0.262	0.012	0.109	0.020
23	109.67	0.156	0.012	0.058	0.010

(a) Refer to Figure 3.7-5A.
(b) Effective horizontal acceleration at Containment Shell due to absolute sum of translational response and torsional response from 5% eccentricity.

TABLE 3.7-8D

CONTAINMENT EXTERIOR STRUCTURE
MAXIMUM SHELL FORCES AND MOMENTS

		Shell Forces (kip/ft)[a]						Shell Moments (kip-ft/ft)[a]					
		Blume-Hosgri Horizontal Input			Vertical Input			Blume-Hosgri Horizontal Input			Vertical Input		
Nodal[b] Point	Elevation (ft)	F_{SS}	F_{TT}	F_{ST}	F_{SS}	F_{TT}	F_{ST}	M_{SS}	M_{TT}	M_{ST}	M_{SS}	M_{TT}	M_{ST}
2	301.64	4.48	7.35	6.85	26.04	10.32	0	0.80	0.39	57.55	3.68	2.94	0
8	274.37	23.05	59.90	44.80	27.97	28.90	0	0.72	0.86	5.87	2.66	1.09	0
10	258.27	34.80	77.10		31.00	28.24	0	2.85	1.59	3.24	3.12	1.36	0
				63.15									
14	231.00	65.60	86.55	101.00	42.95	9.45	0	17.40	5.42	0.64	11.24	1.24	0
17	205.58	118.00	51.40	143.00	57.41	13.38	0	7.80	1.27	0.23	5.86	1.12	0
19	181.08	185.50	47.00	173.50	63.95	11.60	0	11.75	3.86	0.32	3.14	1.40	0
20	155.83	235.00	40.15	200.00	65.85	7.89	0	12.40	2.96	0.69	3.51	1.37	0
22	130.58	325.50	37.80	219.50	66.95	10.76	0	5.99	1.13	0.27	2.08	2.32	0
23	109.67	376.00	36.25	240.50	67.06	8.78	0	19.05	6.26	0.26	6.99	2.78	0

(a) Refer to Figure 3.7-7.

(b) Refer to Figure 3.7-5A.

TABLE 3.7-8E

CONTAINMENT EXTERIOR STRUCTURE
MAXIMUM TOTAL SHEARS AND MAXIMUM OVERTURNING MOMENTS

Nodal[b] Point	Elevation (ft)	Maximum Shear Force (kips x 10^3)[a] Blume-Hosgri Horizontal Input	Maximum Overturning Moment (kip-ft x 10^6)[a] Blume-Hosgri Horizontal Input
2	301.64	0.34	--
8	274.37	10.50	0.18
10	258.27	15.94	0.44
14	231.00	23.67	1.06
17	205.58	32.49	1.91
19	181.08	39.44	3.00
20	155.83	45.39	3.80
22	130.58	49.70	5.26
23	109.67	55.05	6.08
27	88.58	55.81	7.31

(a) Vertical Input does not produce a net shear force.

(b) See Figure 3.7-5A.

TABLE 3.7-8F

CONTAINMENT EXTERIOR STRUCTURE
MAXIMUM TOTAL TORSIONAL MOMENTS AND AXIAL FORCES

Nodal[b] Point	Elevation (ft)	Blume-Hosgri Total Torsional Moment (kip-ft x 10^3)[a] Horizontal Input	Axial Force(kips x 10^3)[c]
2	301.64	5.49	1.52
8	274.37	67.61	9.93
10	258.27	136.94	12.82
14	231.00	216.36	19.36
17	205.58	293.17	25.88
19	181.08	353.32	28.83
20	155.83	400.00	29.69
22	130.58	427.78	30.18
23	109.67	439.99	30.23

(a) Vertical input does not produce a net torque.

(b) See Figure 3.7-5A.

(c) Due to vertical input.

TABLE 3.7-8G

CONTAINMENT INTERIOR STRUCTURE
MAXIMUM ABSOLUTE HORIZONTAL ACCELERATIONS AND DISPLACEMENTS

		Newmark-Hosgri			
Nodal[(a)] Point	Elevation (ft)	Horizontal Acceleration (g)	Torsional Acceleration (rad/sec^2)	Displacement (in.)	Rotation (rad x 10^{-5})
28	140.00	0.92	0.07	0.06	1.21
34	114.00	0.70	0.05	0.03	0.82

(a) See Figure 3.7-5A.

TABLE 3.7-8H

CONTAINMENT INTERIOR STRUCTURE
MAXIMUM TOTAL SHEARS, OVERTURNING MOMENTS,
AND TORSIONAL MOMENTS[a]

		Shear (kips x 10^3)		Overturning Moment (kip-ft x 10^3)		Torsional Moment[c] (kip-ft x 10^3)	
Nodal[b] Point	Elevation (ft)	Blume-Hosgri	Newmark-Hosgri	Blume-Hosgri	Newmark-Hosgri	Blume-Hosgri	Newmark-Hosgri
34	114.00	6.52	6.64	74.55	78.82	136.68	146.48
37	114.00	10.80	11.23	227.89	239.55		
48	88.58	10.08	10.40	219.60	229.31	266.06	283.99
49	88.58	13.26	13.73	544.10	560.76		

(a) Due to horizontal input only.

(b) See Figure 3.7-5A.

(c) Values obtained from lumped-mass model shown in Figure 3.7-5C.

TABLE 3.7-8I

UNIT 1
VERTICAL DYNAMIC ANALYSIS - FRAME NO. 6
SUMMARY OF MODAL PARTICIPATION FACTORS AND FREQUENCIES

Mode	Frequency	X-Direction	Y-Direction	Z-Direction	X-Rotation	Y-Rotation	Z-Rotation
1	11.4	0.00	0.00	0.71	0.0	-541.0	0.0
2	16.8	0.00	0.00	0.09	0.0	-63.0	0.0
3	20.6	0.00	0.00	-0.08	0.0	54.0	0.0
4	21.9	0.00	0.00	0.09	0.0	-65.0	0.0
5	24.1	0.00	0.00	0.03	0.0	-31.0	0.0
6	24.4	0.00	0.00	-0.01	0.0	13.0	0.0
7	24.9	0.00	0.00	-0.03	0.0	22.0	0.0
8	26.2	0.00	0.00	-0.01	0.0	-5.0	0.0
9	28.4	0.00	0.00	-0.50	0.0	304.0	0.0
10	29.3	0.00	0.00	0.20	0.0	-124.0	0.0
11	33.2	0.00	0.00	-0.04	0.0	30.0	0.0

NOTE: X is in the Radial direction.
Y is in the Longitudinal direction.
Z is in the Vertical direction.

TABLE 3.7-8J

UNIT 2
VERTICAL DYNAMIC ANALYSIS - ANNULUS FRAME 6
SUMMARY OF MODAL PARTICIPATION FACTORS AND FREQUENCIES

Mode	Frequency	X-Direction	Y-Direction	Z-Direction	X-Rotation	Y-Rotation	Z-Rotation
1	11.6	.00	0.70	.00	.00	.00	82.0
2	16.0	.00	-0.09	.00	.00	.00	-7.7
3	16.46	.00	0.07	.00	.00	.00	6.6
4	22.9	.00	-0.07	.00	.00	.00	-13.0
5	23.87	.00	0.02	.00	.00	.00	-2.0
6	24.0	.00	0.00	.00	.00	.00	7.0
7	27.8	.00	0.00	.00	.00	.00	-8.1
8	32.28	.00	0.08	.00	.00	.00	-15.0

NOTE: X is in the Radial direction.
Y is in the Longitudinal direction.
Z is in the Vertical direction.

TABLE 3.7-8K

CONTAINMENT ANNULUS STRUCTURES UNITS 1 AND 2
NATURAL FREQUENCIES FOR HORIZONTAL
SEISMIC GROUND MOTION

Unit	Elevation	Mode	Frequency (cps)
1	101	1	20.95
		2	21.69
	106	1	20.16
		2	21.47
	117	1	20.24
		2	22.78
2	101	1	22.46
		2	22.79
	106	1	19.98
		2	22.20
	117	1	19.98
		2	25.79

TABLE 3.7-8L

POLAR GANTRY CRANE
MIXIMUM DISPLACEMENTS, HOSGRI
(UNIT 1)

Condition	Node	Longitudinal Direction (in.)	Transverse Direction (in.)	Vertical Direction (in.)
Unloaded	31	6.16	6.11	1.77
	32	6.29	6.11	2.10
	39	6.16	5.27	1.49
	40	6.30	5.28	1.47
	47	6.16	5.40	1.71
	48	6.29	5.40	1.81
	75	-	-	1.57
	76	-	-	1.92
	77	-	-	1.71
	78	-	-	1.81
Loaded, 200 tons	31	6.61	5.22	1.69
	32	6.67	5.22	1.74
	39	6.62	4.76	2.12
	40	6.68	4.76	2.09
	47	6.62	4.56	1.12
	48	6.67	4.56	1.24
	75	-	-	1.49
	76	-	-	1.34
	77	-	-	1.12
	78	-	-	1.26

Notes:

1. All displacements are measured relative to base of crane.

2. For node numbers, refer to Figure 3.7-7A.

3. See PG&E Calculation No. 2252C-2 (Reference 37).

TABLE 3.7-8L Sheet 2 of 2

POLAR GANTRY CRANE
MAXIMUM DISPLACEMENTS, HOSGRI
(UNIT 2)

Condition	Node	Longitudinal Direction (in.)	Transverse Direction (in.)	Vertical Direction (in.)
Unloaded	31	6.16	6.11	1.77
	32	6.29	6.11	2.10
	39	6.16	5.27	1.49
	40	6.30	5.28	1.47
	47	6.16	5.40	1.71
	48	6.29	5.40	1.81
	75	-	-	1.57
	76	-	-	1.92
	77	-	-	1.71
	78	-	-	1.81
Loaded, 200 tons	31	6.61	5.22	1.69
	32	6.67	5.22	1.74
	39	6.62	4.76	2.12
	40	6.68	4.76	2.09
	47	6.62	4.56	1.12
	48	6.67	4.56	1.24
	75	-	-	1.49
	76	-	-	1.34
	77	-	-	1.12
	78	-	-	1.26

Notes:

1. All displacements are measured relative to base of crane.
2. For node numbers, refer to Figure 3.7-7A.
3. See Civil Calculation 2252C-4 (Reference 39).

TABLE 3.7-8M

POLAR GANTRY CRANE
MAXIMUM FORCES,
HOSGRI - UNLOADED CONDITION
(UNIT 1)

Type of Element	Element	Node	Axial Force (kips)	Bending Moment About Axis Y (kip-in.)	Bending Moment About Axis Z (kip-in.)
Girder Beam	13	35	212	111,700	17,950
Girder Beam	18	36	172	122,900	18,100
Gantry Leg	2	11	679	23,240	15,810
Gantry Leg	3	15	637	24,660	81,950
Gantry Leg	6	12	635	24,020	16,270
Gantry Leg	7	16	575	25,060	86,170
Gantry Leg	29	59	646	27,740	81,400
Gantry Leg	28	63	720	26,000	16,140
Gantry Leg	33	60	679	26,140	80,930
Gantry Leg	32	64	703	24,080	16,590
Sill Beam	11	9	38	28,800	2,652
Sill Beam	26	68	39	29,470	2,914
Leg Tie BM	10	20	15	28,640	7,176
Leg Tie BM	25	57	6	26,750	4,227
Girder Tie BM	9	34	9	8,853	12,080
Girder Tie BM	24	50	8	14,280	13,480
Trolley	22	71	56	23,280	3,480

Notes:

1. For node and element numbers, refer to Figure 3.7-7A.

2. See PG&E Calculation No. 2252C-3 (Reference 38).

3. Y-axis represents the major axis of cross section. Z-axis represents the minor axis.

TABLE 3.7-8M

POLAR GANTRY CRANE
MAXIMUM FORCES,
HOSGRI - UNLOADED CONDITION
(UNIT 2)

Type of Element	Element	Node	Axial Force (kips)	Bending Moment About Axis Y (kip-in.)	Bending Moment About Axis Z (kip-in.)
Girder Beam	13	35	212	111,700	17,950
Girder Beam	18	36	172	122,900	18,100
Gantry Leg	2	11	679	23,240	15,810
Gantry Leg	3	15	637	24,660	81,950
Gantry Leg	6	12	635	24,020	16,270
Gantry Leg	7	16	575	25,060	86,170
Gantry Leg	29	59	646	27,740	81,400
Gantry Leg	28	63	720	26,000	16,140
Gantry Leg	33	60	679	26,140	80,930
Gantry Leg	32	64	703	24,080	16,590
Sill Beam	11	9	38	28,800	2,652
Sill Beam	26	68	39	29,470	2,914
Leg Tie BM	10	20	15	28,640	7,176
Leg Tie BM	25	57	6	26,750	4,227
Girder Tie BM	9	34	9	8,853	12,080
Girder Tie BM	24	50	8	14,280	13,480
Trolley	22	71	56	23,280	3,480

Notes:

1. For node and element numbers, refer to Figure 3.7-7A.
2. See PG&E Calculation No. 2252C-4 (Reference 39).
3. Y-axis represents the major axis of cross section. Z-axis represents the minor axis.

TABLE 3.7-8N

POLAR GANTRY CRANE
MAXIMUM FORCES,
HOSGRI - LOADED CONDITION
(UNIT 1)

Type of Element	Element	Node	Axial Force (kips)	Bending Moment About Axis Y (kip-in.)	Bending Moment About Axis Z (kip-in.)
Girder Beam	14	39	190	210,700	25,310
Girder Beam	19	40	181	219,000	25,570
Gantry Leg	3	15	824	27,550	105,000
Gantry Leg	4	21	698	10,750	114,700
Gantry Leg	7	16	881	28,290	107,000
Gantry Leg	8	22	675	10,380	115,700
Gantry Leg	30	51	670	11,710	149,700
Gantry Leg	29	59	762	26,430	113,900
Gantry Leg	34	52	681	11,570	147,900
Gantry Leg	33	60	738	25,230	112,100
Sill Beam	11	9	45	34,760	2,410
Sill Beam	26	68	47	30,560	2,370
Leg Tie BM	10	20	15	36,710	7,683
Leg Tie BM	25	58	6	24,640	4,626
Girder Tie BM	9	33	9	11,530	13,550
Girder Tie BM	24	50	7	15,900	14,850
Trolley	22	71	57	88,950	3,269

Notes:

1. For node and element numbers, refer to Figure 3.7-7A.

2. See PG&E Calculation No. 2252C-3 (Reference 38).

3. Y-axis represents the major axis of cross section. Z-axis represents the minor axis.

TABLE 3.7-8N

POLAR GANTRY CRANE
MAXIMUM FORCES,
HOSGRI - LOADED CONDITION
(UNIT 2)

Type of Element	Element	Node	Axial Force (kips)	Bending Moment About Axis Y (kip-in.)	Bending Moment About Axis Z (kip-in.)
Girder Beam	14	39	190	210,700	25,310
Girder Beam	19	40	181	219,000	25,570
Gantry Leg	3	15	824	27,550	105,000
Gantry Leg	4	21	698	10,750	114,700
Gantry Leg	7	16	881	28,290	107,000
Gantry Leg	8	22	675	10,380	115,700
Gantry Leg	30	51	670	11,710	149,700
Gantry Leg	29	59	762	26,430	113,900
Gantry Leg	34	52	681	11,570	147,900
Gantry Leg	33	60	738	25,230	112,100
Sill Beam	11	9	45	34,760	2,410
Sill Beam	26	68	47	30,560	2,370
Leg Tie BM	10	20	15	36,710	7,683
Leg Tie BM	25	58	6	24,640	4,626
Girder Tie BM	9	33	9	11,530	13,550
Girder Tie BM	24	50	7	15,900	14,850
Trolley	22	71	57	88,950	3,269

Notes:

1. For node and element numbers, refer to Figure 3.7-7A.
2. See PG&E Calculation No. 2252C-4 (Reference 39).
3. Y-axis represents the major axis of cross section. Z-axis represents the minor axis.

TABLE 3.7-8O

FUEL HANDLING CRANE SUPPORT STRUCTURE
MAXIMUM ABSOLUTE ACCELERATIONS

		Acceleration,[a] g		
Load Case	Location	NS	EW	Vertical
HE	El 188 ft	1.7	1.6	1.1
	Columns, El 166 ft	1.6	1.3	0.6
DE	El 188 ft	0.8	0.5	(b)
	Columns, El 166 ft	0.7	0.4	(b)

(a) Accelerations are average of accelerations from models 2.1 and 2.2.

(b) DE vertical equivalent static analysis coefficient is 0.13 g.

TABLE 3.7-8P

FUEL HANDLING CRANE SUPPORT STRUCTURE
MAXIMUM RELATIVE DISPLACEMENTS

Load Case	Location	Displacements[a], in. NS	EW
HE	El 188 ft	2.0	8.8
	Columns, El 166 ft	1.8	7.1
DE	El 188 ft	0.9	2.8
	Columns, El 166 ft	0.9	2.3

(a) Displacements are from static analysis of detailed fuel handling crane support structure model described in Section 3.8.2.4. Displacements are relative to elevation 140 ft.

TABLE 3.7-9

AUXILIARY BUILDING
PERIODS OF VIBRATION - DE ANALYSIS

	N-S Direction		E-W Direction	
Mode No.	Period, T, (sec)	Translational Participation Factor	Period, T, (sec)	Translational Participation Factor
1[(a)]	0.641	0.0	0.688	8.6
2[(a)]	0.327	8.9	0.641	0.0
3	0.073	48.1	0.072	68.7
4	0.059	48.7	0.065	0.0
5	0.037	20.0	0.040	20.6
6	0.031	1.5	0.031	0.0

(a) Steel superstructure modes (one translational and the other torsional).

TABLE 3.7-10

AUXILIARY BUILDING HORIZONTAL MODEL
PERIODS AND PARTICIPATION FACTORS - HE ANALYSIS

COUPLED - TRANSLATIONAL PLUS TORSION

	North-South Model with 5% Eccentricity to East		North-South Model with 5% Eccentricity to West		East-West Model with 5% Eccentricity	
Mode No.	Period (sec)	Translation Participation Factor	Period (sec)	Translation Participation Factor	Period (sec)	Translation Participation Factor
1[a]	.641	-0.0	.641	0.0	.688	8.6
2[a]	.327	8.9	.327	8.9	.641	0.0
3	.070	-41.2	.075	-47.6	.074	60.2
4	.061	54.7	.056	49.4	.062	33.2
5	.036	-19.7	.037	-19.4	.039	-20.8
6	.030	-3.8	.030	7.0	.030	3.1
7	.025	-15.3	.026	14.6	.028	-17.9
8	.024	19.6	.023	-19.2	.023	14.8
9	.015	2.8	.016	-7.8	.017	11.3
10	.014	13.9	.014	-11.7	.014	9.0

(a) Steel superstructure modes (one translational and the other torsional).

TABLE 3.7-11

AUXILIARY BUILDING VERTICAL MODEL
PERIODS AND PARTICIPATION FACTORS - HE ANALYSIS

Mode Number[a]	Period (sec)	Participation Factor
1[b]	0.085	-6.0
2	0.033	67.9

(a) Only modes below 33 Hz are listed.

(b) Steel superstructure roof mode.

TABLE 3.7-11A

FUEL HANDLING CRANE SUPPORT STRUCTURE
HORIZONTAL MODELS FREQUENCIES OF VIBRATION[a]

DE, DDE, AND HE ANALYSES

	Model 2.1		Model 2.2	
First[b] Fundamental Modal Direction	Frequency (cps)	Modal Effective Mass %	Frequency (cps)	Modal Effective Mass (%)
E-W	1.6	85.0	1.6	86.0
N-S	3.1	88.0	2.7[a]	92.0

(a) For Model 2.2 see Figure 3.7-13B. Model 2.1 represents six end bay frames and is similar.

(b) Other modes have insignificant contributions, and are not included.

TABLE 3.7-11B

FUEL HANDLING CRANE SUPPORT STRUCTURE VERTICAL MODEL FREQUENCIES OF VIBRATION[(a)(b)]

DE, DDE, AND HE ANALYSES

Model 2.1		Model 2.2	
Frequency (hz)	Modal Effective Mass (% of roof)	Frequency (hz)	Modal Effective Mass (% of roof)
10.6	19	10.1	23
16.9	23	16.6	26
20.7	2	22.8	1

(a) Only significant modes with frequencies less than 33 hz are shown. For models 2.1 and 2.2, 99 modes and 105 modes were extracted, respectively, with frequencies up to 105 hz.

(b) For model 2.2, see Figure 3.7-13B. Model 2.1 represents six end bay frames and is similar.

TABLE 3.7-12

AUXILIARY BUILDING
MAXIMUM ABSOLUTE ACCELERATIONS-DE ANALYSIS

		Maximum Absolute Accelerations		
		N-S Direction		E-W Direction
Mass Point[a]	Elevation, ft	Horizontal Acceleration, g	Rotational Acceleration rad/sec^2	Horizontal Acceleration, g
6	188.0	0.554	0.0004	0.313
1	163.0	0.375	0.0217	0.435
2	140.0	0.300	0.0187	0.324
3	115.0	0.259	0.0115	0.291
4	100.0	0.230	0.0055	0.257

(a) See Figure 3.7-13.

TABLE 3.7-13

AUXILIARY BUILDING
MAXIMUM RELATIVE DISPLACEMENTS-DE ANALYSIS

		Maximum Relative Displacement		
		N-S Direction		E-W Direction
Mass[(a)] Point	Elevation, ft	Horizontal Translation, in.	Rotation radians x 10^{-6}	Horizontal Translation, in.
6	188.0	0.575	1.624	1.447
1	163.0	0.022	4.087	0.025
2	140.0	0.015	3.308	0.018
3	115.0	0.009	2.014	0.012
4	100.0	0.004	0.975	0.006

(a) See Figure 3.7-13.

TABLE 3.7-14

AUXILIARY BUILDING
MAXIMUM STORY SHEARS-DE ANALYSIS

Element[a]	Maximum Story Shears, kips x 10^3 N-S Direction	Maximum Story Shears, kips x 10^3 E-W Direction
5	1.3	0.7
1	3.8	4.8
2	26.3	24.6
3	40.0	42.7
4	28.5	28.4

(a) See Figure 3.7-13.

TABLE 3.7-15

AUXILIARY BUILDING
MAXIMUM OVERTURNING MOMENTS-DE ANALYSIS

Element[a]	Maximum O.T. Moments, kips - ft x 10^6 N-S Direction	E-W Direction
5	0.06	0.04
1	0.08	0.10
2	0.74	0.68
3	1.32	1.30
4	1.76	1.74

(a) See Figure 3.7-13.

TABLE 3.7-16

AUXILIARY BUILDING
MAXIMUM TORSIONAL MOMENTS DUE TO
EARTHQUAKE IN N-S DIRECTION-DE ANALYSIS

Element[(a)]	Maximum Torsional Moments, kip - ft x 10^5
5	0.004
1	0.265
2	14.080
3	19.810
4	9.139

(a) See Figure 3.7-13.

TABLE 3.7-17

AUXILIARY BUILDING
MAXIMUM ABSOLUTE ACCELERATIONS - HE ANALYSIS
EARTHQUAKE IN N-S DIRECTION

Mass[a]	Elevation,	Blume-Hosgri Horizontal Acceleration, g		Newmark-Hosgri Horizontal Acceleration, g		Blume-Hosgri Rotational Acceleration, rad/sec^2		Newmark-Hosgri Rotational Acceleration, rad/sec^2	
Point	ft	5% E	5% W	5% E	5% W	5% E	5% W	5% E	5% W
6	188.0	1.57	1.56	1.37	1.36	-	-	-	-
1	163.0	1.13	1.10	1.25	1.21	.0981	.1477	.1102	.1646
2	140.0	0.84	0.77	0.90	0.84	.0604	.0895	.0710	.1018
3	115.0	0.71	0.70	0.72	0.66	.0370	.0516	.0431	.0590
4	100.0	0.67	0.66	0.58	0.61	.0175	.0239	.0206	.0275

(a) See Figure 3.7-13.

TABLE 3.7-18

AUXILIARY BUILDING
MAXIMUM ABSOLUTE ACCELERATIONS - HE ANALYSIS
EARTHQUAKE IN E-W DIRECTION

Mass[a] Point	Elevation, ft	Blume-Hosgri Horizontal Acceleration, g	Newmark-Hosgri Horizontal Acceleration, g	Blume-Hosgri Rotational Acceleration, rad/sec^2	Newmark-Hosgri Rotational Acceleration, rad/sec^2
6	188.0	1.11	1.24	.0015	.0017
1	163.0	1.11	1.22	.1162	.1292
2	140.0	0.94	1.00	.0769	.0881
3	115.0	0.72	0.75	.0452	.0527
4	100.0	0.66	0.62	.0213	.0246

(a) See Figure 3.7-13.

TABLE 3.7-19

AUXILIARY BUILDING
MAXIMUM RELATIVE DISPLACEMENTS - HE ANALYSIS
EARTHQUAKE IN N-S DIRECTION

Mass[a] Point	Elevation, ft	Blume-Hosgri Horizontal Translation, in.		Newmark-Hosgri Horizontal Translation, in.		Blume-Hosgri Rotation radians x 10^{-6}		Newmark-Hosgri Rotation radians x 10^{6}	
		5% E	5% W	5% E	5% W	5% E	5% W	5% E	5% W
6	188.0	1.63	1.62	1.42	1.42	-	-	-	-
1	163.0	0.06	0.06	0.06	0.07	9.589	18.901	10.961	21.001
2	140.0	0.04	0.04	0.04	0.04	8.034	14.091	9.135	15.472
3	115.0	0.02	0.02	0.02	0.02	4.785	8.530	5.468	9.361
4	100.0	0.01	0.01	0.01	0.01	2.256	4.044	2.576	4.444

(a) See Figure 3.7-13.

TABLE 3.7-20

AUXILIARY BUILDING
MAXIMUM RELATIVE DISPLACEMENTS - HE ANALYSIS
EARTHQUAKE IN E-W DIRECTION

Mass[a] Point	Elevation, ft	Blume-Hosgri Horizontal Translation, in.	Newmark-Hosgri Horizontal Translation in.	Blume-Hosgri Rotation radians x 10^{-6}	Newmark-Hosgri Rotation radians x 10^{6}
6	188.0	5.08	5.63	6.153	6.897
1	163.0	0.07	0.07	14.616	16.354
2	140.0	0.05	0.05	11.219	12.484
3	115.0	0.03	0.03	7.078	7.873
4	100.0	0.02	0.02	3.471	3.854

(a) See Figure 3.7-13.

TABLE 3.7-21

AUXILIARY BUILDING
MAXIMUM STORY SHEARS - HE ANALYSIS

	Earthquake in N-S Direction				Earthquake in E-W Direction	
	Blume-Hosgri Shear kips x 10^3		Newmark-Hosgri Shear kips x 10^3		Blume-Hosgri Shear kips x 10^3	Newmark-Hosgri Shear kips x 10^3
Element[(a)]	5% E	5% W	5% E	5% W	±5%	±5%
5	3.6	3.6	3.2	3.2	2.6	2.9
1	12.4	12.4	13.7	13.6	12.3	13.6
2	65.6	58.4	71.9	63.3	71.0	76.2
3	106.2	100.0	115.2	101.5	115.0	122.5
4	84.5	86.7	90.2	85.8	75.8	80.2

(a) See Figure 3.7-13.

TABLE 3.7-22

AUXILIARY BUILDING
MAXIMUM OVERTURNING MOMENTS - HE ANALYSIS

	Earthquake in N-S Direction				Earthquake in E-W Direction	
	Blume-Hosgri Moment kips x 10^6		Newmark-Hosgri Moment kips x 10^6		Blume-Hosgri Moment kips x 10^6	Newmark-Hosgri Moment kips x 10^6
Element[(a)]	5% E	5% W	5% E	5% W	±5%	±5%
5	0.18	0.17	0.15	0.15	0.12	0.14
1	0.27	0.27	0.30	0.29	0.27	0.29
2	1.85	1.66	2.05	1.74	1.96	2.11
3	3.41	3.12	3.73	3.21	3.65	3.90
4	4.68	4.42	5.11	4.46	4.82	5.15

(a) Refer to Figure 3.7-13.

TABLE 3.7-23

AUXILIARY BUILDING
MAXIMUM TORSIONAL MOMENTS - HE ANALYSIS

	Earthquake in N-S Direction				Earthquake in E-W Direction	
	Blume-Hosgri Torsional Moment kips x 105		Newmark-Hosgri Torsional Moment kips x 10^5		Blume-Hosgri Torsional Moment kips x 10^5	Newmark-Hosgri Torsional Moment kips x 10^5
Element[a]	5% E	5% W	5% E	5% W	±5%	±5%
5	0.01	0.02	0.01	0.02	0.01	0.02
1	0.82	1.71	0.89	1.88	1.30	1.45
2	35.36	60.41	39.93	66.53	44.98	50.14
3	48.24	85.27	55.16	93.73	68.81	76.67
4	21.14	38.00	24.14	41.73	32.59	36.19

(a) See Figure 3.7-13.

TABLE 3.7-23A

TURBINE BUILDING HORIZONTAL MODEL (LOADED CRANE CASE 2)[a]
FREQUENCIES OF VIBRATION[b] - HE ANALYSIS[d]

Mode No.	Frequency Hz	Participation Factor North-South	East-West
1[c]	1.39	0.00	3.01
4[c]	3.32	2.59	0.00
18	5.81	-4.75	0.77
19	5.86	-0.02	-3.32
21	6.03	-0.32	-2.29
22	6.19	0.76	-2.60
23	6.37	-1.16	0.81
24	6.43	-2.66	-0.66
26	6.78	0.24	-1.74
27	7.10	-2.02	0.50
28	7.18	1.08	-1.32
29	7.32	-2.13	0.62
30	7.36	1.02	-0.03
31	7.43	-1.51	-0.61
32	7.57	0.44	-2.59
33	7.62	1.83	1.42
36	7.99	-2.04	-0.95
38	8.32	-1.70	-0.94
39	8.38	-3.16	-0.70
40	8.47	2.47	1.17
44	9.09	0.30	-1.78
56	10.71	0.28	-1.54
65	11.52	1.54	-0.14
82	13.10	-0.39	2.02
83	13.15	0.29	-1.43
89	14.02	0.01	1.79
92	14.32	-0.11	1.87
93	14.78	0.33	1.04
100	16.60	-0.31	1.54
101	16.74	0.13	2.00
102	16.81	0.52	-1.78
103	17.03	1.65	-0.22
104	17.26	-1.46	-0.54
111	18.16	0.12	1.11
115	18.93	-1.68	-0.10
147	22.29	1.00	-0.41
165	23.89	-0.89	0.49

TABLE 3.7-23A

(a) For Case 2, the Unit 1 crane has a 15-ton load and is located at column line 9, and the Unit 2 crane has a 50-ton load and is located at column line 12.2.

(b) 210 modes were extracted with frequencies ranging from 1.39 Hz to 33.01 Hz. Shown above are the modes with the twenty highest participation factors in each direction. The cumulative modal masses of the 210 modes represent 95% of the total weight of the building.

(c) Modes 1 and 4 are the principal modes of the superstructure.

(d) Note that the results in this table correspond with the seismic analysis performed for the operating license review (Reference 18) and may not reflect the latest as-built configuration of the turbine building.

TABLE 3.7-23B

TURBINE BUILDING VERTICAL MODEL NO. 1
FREQUENCIES OF VIBRATION[(a)] - HE ANALYSIS[(c)]

Mode No.	Frequency Hz	Participation Factor
1[(b)]	2.80	0.65
2[(b)]	2.80	0.38
5[(b)]	5.07	-0.48
15	7.48	2.00
17	7.75	-1.55
21	8.71	1.04
23	9.16	-0.55
33	10.70	-0.40
35	10.87	0.52
40	11.38	0.53
54	13.34	-0.36
59	14.02	0.43
61	14.23	0.57
62	14.40	0.31
63	14.46	-0.87
64	14.56	0.28
78	16.68	0.30
83	17.36	-0.65
89	18.29	0.48
115	22.03	-0.29

(a) 187 modes were extracted, ranging from 2.80 Hz to 33.01 Hz. Shown above are the modes with the 20 highest participation factors. The cumulative modal mass of the 187 modes represents 94% of the total weight of the building.

(b) Modes 1, 2, and 5 represent significant modes for the superstructure and overhead crane.

(c) Note that the results in this table correspond to the seismic analysis performed for the operating license review (Reference 18) and may not reflect the latest as-built configuration of the turbine building.

TABLE 3.7-23C

TURBINE BUILDING MODEL
LOADED CRANE CASES
MAXIMUM ABSOLUTE ACCELERATIONS - HE ANALYSIS[b]

Elevation, ft	Location		Acceleration[a], g		
	Bent	Line	N-S	E-W	Vertical
104 & 107	1 to 5	A-G	--	1.16	1.54
	5 to 15	A-G	--	1.18	1.99
	15 to 17	A-G	--	1.06	2.49
	17 to 19	A-G	--	1.11	--
	1 to 19	A-G	1.22	--	--
119 & 123	1 to 5	A-G	--	1.84	2.16
	5 to 15	A-G	--	2.08	2.35
	15 to 17	A-G	--	1.83	2.43
	17 to 19	A-G	--	1.68	1.30
	1 to 19	A-G	2.20	--	--
140	1 to 5	A-G	--	1.37	1.68
	5 to 15	A-G	--	2.19	1.91
	15 to 17	A-G	--	1.24	1.29
	17 to 19	A-G	--	1.11	1.32
	1 to 19	A-G	1.91	--	--
159	1.9 to 4.8	G	--	1.29	0.73
	5.7 to 15	G	--	2.51	0.70
	16 to 19	G	--	1.51	0.59
	1.9 to 19	G	1.57	--	--
193	1.9 to 4.8	A, G	--	--	0.91
	5.7 to 15	A, G	--	--	0.70
	16 to 19	A, G	--	--	0.59
Roof	1 to 1.9	A-D	3.97	1.60	

(a) Acceleration values are zero period accelerations of floor response spectra. At and below elevation 140 feet, values are for the case of a single unloaded crane; values for the case of two cranes with one crane loaded are similar.

(b) Note that the results in this table correspond to the seismic analysis performed for the operating license review (Reference 18) and may not reflect the latest as-built configuration of the turbine building.

TABLE 3.7-23D

TURBINE BUILDING MODELS
LOADED CRANE CASES
MAXIMUM RELATIVE DISPLACEMENTS - HE ANALYSIS[b]

Elevation, ft	Location		Displacement[a], in.		
	Bent	Line	N-S	E-W	Vertical
104 & 107	1 to 5	A-G	0.08	0.07	0.25
	5 to 15	A-G	0.07	0.16	0.75
	15 to 17	A-G	0.04	0.13	1.31
	17 to 19	A-G	0.06	0.68	--
119, 123, & 125	1 to 5	A-G	0.25	0.23	0.52
	5 to 15	A-G	0.60	0.42	0.82
	15 to 17	A-G	0.80	0.35	0.62
	17 to 19	A-G	0.90	1.02	0.45
140	1 to 5	A-G	0.22	0.18	0.37
	5 to 15	A-G	0.20	0.58	0.03
	15 to 17	A-G	0.20	0.28	0.13
	17 to 19	A-G	0.20	0.83	0.42
159	1 to 5	G	0.42	2.06	0.39
	5 to 15	G	0.42	3.22	0.31
	15 to 17	G	0.41	2.97	0.06
	17 to 19	G	0.42	3.98	--
193	1 to 5	A-G	1.92	5.89	0.13
	5 to 15	A-G	1.09	8.76	0.05
	15 to 17	A-G	1.21	9.64	0.08
	17 to 19	A-G	1.32	11.46	--
Roof	1 to 1.9	A-D	2.60	3.64	--

(a) Displacement values are based on response spectrum analysis.

(b) Note that the results in this table correspond to the seismic analysis performed for the operating license review (Reference 18) and may not reflect the latest as-built configuration of the turbine building.

TABLE 3.7-23E

TURBINE PEDESTAL MODEL
FREQUENCIES OF VIBRATION[(a)] - HE ANALYSIS

Mode No.	Frequency (Hz)	Participation Factor N-S	Participation Factor E-W	Participation Factor Vertical
1	3.09	--	87.5	--
2	3.55	--	1.8	--
3	4.22	89.9	--	--
11	14.9	--	--	23.7
12	16.16	--	--	8.2
21	20.96	--	--	17.7
22	21.19	--	0.5	--
26	22.32	--	--	6.0
27	22.37	0.3	2.1	--
28	23.15	--	--	6.0
29	23.36	0.2	0.5	--
32	24.27	--	--	2.6
42	29.06	0.4	--	3.8
43	29.59	0.5	0.3	--
44	30.39	0.9	0.2	--
47	31.34	1.7	--	2.3

(a) 50 modes were extracted, ranging in frequency from 3.09 Hz to 32.26 Hz. Shown above are the modes with the five highest participation factors for each direction.

TABLE 3.7-23F

TURBINE PEDESTAL MODEL
MAXIMUM RELATIVE DISPLACEMENTS - HE ANALYSIS[(a)]

Nodal Point[(b)]	North-South Direction (in.)	East-West Direction (in.)
124	0.85 (N)	
36	0.82 (S)	
82		1.72 (E)
81		1.72 (W)

(a) Displacements are an envelope of the results for the Unit 1 and Unit 2 Pedestals.

(b) Refer to Figure 3.7-15G (Unit 2 model is a mirror image, reflected at frame 6)

TABLE 3.7-23G

SIGNIFICANT PERIODS OF VIBRATION AND PERCENT PARTICIPATION FACTORS
INTAKE STRUCTURE

	North-South Model		East-West Vertical Model		
Mode Number	Period (sec)	Percent Participation Factor	Period (sec)	East-West Percent Participation Factor	Vertical Percent Participation Factor
2	0.081	0.5	0.081	0.0	0.1
3	0.081	0.1	0.081	0.0	0.0
4	0.081	0.2	0.081	0.0	0.0
5	0.080	1.1	0.080	0.0	0.0
6	0.079	2.5	0.079	0.0	0.3
7	0.078	0.8	0.078	0.0	0.1
9	0.069	0.1	0.065	0.1	0.1
10	0.066	2.1	0.065	0.4	0.0
11	0.065	3.5	0.064	0.2	0.0
12	0.064	0.2	0.064	0.0	0.0
13	0.064	0.4	0.064	0.0	0.0
14	0.064	0.0	0.064	0.0	0.0
15	0.064	0.2	0.063	0.4	0.1
17	0.063	0.0	0.063	0.1	0.1
18	0.063	0.3	0.062	0.4	0.0
19	0.060	0.7	0.060	0.1	0.9
21	0.049	0.8	0.049	1.7	0.7
22	0.049	1.4	0.048	2.1	0.6
23	0.048	1.1	0.048	0.3	0.0
24	0.047	2.7	0.047	2.4	0.6
25	0.047	2.1	0.046	3.4	1.4
33	0.034	1.4	0.033		7.4
35	0.032	1.6	0.032	3.0	5.4
36	0.031	3.4	0.031	1.3	4.7

TABLE 3.7-23H

MAXIMUM RELATIVE DISPLACEMENTS AND
MAXIMUM ABSOLUTE ACCELERATIONS (HOSGRI)
INTAKE STRUCTURE

Nodal Point[a]	Elevation (ft)	North-South Displacement (in.)	East-West Displacement (in.)	Vertical Displacement (in.)	North-South Acceleration (g)	East-West Acceleration (g)	Vertical Acceleration (g)
330	+32.0	0.025	0.044	0.010	2.36	2.15	0.65
312	+24.4	0.016	0.029	0.008	1.50	1.55	0.65
71	+17.5	0.120	0.011	0.010	1.58	0.66	0.61
73	+17.5	0.065	0.011	0.007	0.94	0.66	0.54
74	+17.5	0.058	0.011	0.005	0.85	0.66	0.51
284	+17.5	0.009	0.019	0.007	0.64	1.00	0.62
363	+11.0	0.008	0.012	0.003	0.64	0.87	0.54
80	-2.1	0.133	0.007	0.010	1.71	0.70	0.59
83	-2.1	0.066	0.005	0.006	0.98	0.71	0.52
87	-16.8	0.251	0.003	0.005	3.14	0.72	0.52
89	-16.8	0.050	0.002	0.003	1.12	0.73	0.50

(a) See Figure 3.7-15F.

TABLE 3.7-23I

OUTDOOR WATER STORAGE TANKS
SUMMARY OF SIGNIFICANT PERIODS AND PERCENT PARTICIPATION FACTORS
REFUELING WATER STORAGE TANK

Mode No.	Period (sec)	Modal Participation Factor %
1	0.132	50.7
2	0.052	22.6

TABLE 3.7-23J

OUTDOOR WATER STORAGE TANKS
SUMMARY OF SIGNIFICANT PERIODS AND PERCENT PARTICIPATION FACTORS
FIREWATER AND TRANSFER TANK

	Load Case 1[a]		Load Case 2[b]	
Mode No.	Period (sec.)	Modal Participation Factor %	Period (sec.)	Modal Participation Factor %
1	0.12124	54.91	0.1234	48.4
2	0.04985	21.26	0.06318	6.21

(a) Load Case 1: Inner and outer tanks are filled with water up to design level
(b) Load Case 2: Inner tank is filled to design level, outer tank is empty

Table 3.7-24

FUNDAMENTAL MODE FREQUENCY RANGES
FOR RCL PRIMARY EQUIPMENT

	Frequency, Hz
Steam Generator	6.7 – 9.0
Reactor coolant pump	6.7 - 7.2
Reactor pressure vessel	16.8 - 17.0

TABLE 3.8-1

TESTING OF REINFORCING BARS FOR DESIGN CLASS I CONCRETE STRUCTURES
COMPARISON OF PROGRAM USED ON DIABLO CANYON POWER PLANT WITH REGULATORY GUIDE 1.15

Diablo Canyon Power Plan	Regulatory Guide 1.15
The number of test specimens required for acceptance is in accordance with ASTM A 615, Deformed Billet-Steel Bars for Concrete-Reinforcement, American Society for Testing and Materials. Additional samples were tested as part of the splice testing program. The requirements for acceptance testing are more stringent than ASTM A 615 in that all tests must be conducted using the full section of the bar.	At least one full-diameter specimen from each bar size should be tested for each 50 tons or fraction thereof of reinforcing bars that are produced from each heat and used in Category I structures.
Test procedures are in accordance with ASTM A 615-68.	The test procedures should be in accordance with ASTM A 370-68, Standard Methods and Definitions for Mechanical Testing of Steel Products, American Society for Testing and Materials.
Acceptance standards are in accordance with ASTM A 615-68 using full sections of the bars as rolled. Bend test requirements described in Item 3, Sheet 2 of 2, are more stringent than those in Supplemental Requirements (S-1) of ASTM A 615-72.	The acceptance standards should be in accordance with ASTM A 615-72, Standard Specification for Deformed Billet-Steel Bars for Concrete Reinforcement, American Society for Testing and Materials, including Supplemental Requirement (S-1)[a] using full sections of the bars as rolled.

TABLE 3.8-1

Diablo Canyon Power Plan	Regulatory Guide 1.15
In addition to the requirements of ASTM A 615, the Company specification requires the following: 1. Grade 60 bars be limited in carbon and manganese content to a maximum of 0.45% and 1.3% respectively. 2. Performance of a check analysis, which is listed as an option in ASTM A 615. 3. No. 14 and No. 18 bars be subjected to a 90° bend test using a pin having a diameter eight times the diameter of the bar.	Where any material property such as yield strength to tensile strength ratio, ductility, weldability or other similar property is relied upon by the designer or constructor, then the reinforcing bar chemistry should be controlled to the extent required to achieve the desired material property, and confirmatory testing should be performed.
Deformations were inspected during production to ensure conformance with ASTM A 615.	Deformations of the reinforcing bars should be inspected to assure their compliance with ASTM A 615-72 and with the licensee's specifications pertinent to bonding and other purposes which are dependent on the deformation characteristics.
Adequacy of deformations for splicing was demonstrated by the tensile tests of the Cadweld splices. See Table 3.8-2.	Adequacy of deformations for splicing will be demonstrated by the tensile tests of the splices. mechanical splice. See Safety Guide 10, "Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures."

(a) Supplemental Requirement (S-1) is for a 90° bend test, using a pin diameter 10 times the bar diameter, on No. 14 and No. 18 bars.

TABLE 3.8-2

MECHANICAL (CADWELD) SPLICES IN REINFORCING BARS OF CONCRETE CONTAINMENTS
COMPARISON OF PROGRAM USED ON DIABLO CANYON POWER PLANT WITH SAFETY GUIDE 10

Diablo Canyon Power Plant	Safety Guide 10
Prior to production splicing, each operator was instructed by a representative of the manufacturer. Each operator (a crew consisted of an operator and a helper) prepared one qualification splice for each of the splice positions for which he was qualified. The qualification splice was made using the same materials as those used in the structures. The completed qualification splices had to pass visual inspection and develop the minimum tensile strength of the reinforcing steel. A manufacturer's representative was present for at least the first 20 production splices for each crew to verify that proper procedures were being used and quality splices obtained.	1. Crew Qualification - Each member of splicing crew (or each crew if the members work as a crew) should prepare two qualification splices for each of the splice positions (e.g., horizontal, vertical, diagonal) to be used. The qualification splices should be made using the same materials (e.g., bar, sleeve, powder) as those to be used in the structure. The completed qualification splices should meet the requirements specified by the designer of the containment structure and approved by the licensee, pass visual inspection as provided by Paragraph 2 below, and meet the tensile tests as provided by Paragraph 3 below.
All completed splices were visually inspected in accordance with the recommendations of the Erico Co. inspection manual RB-5M 768, Inspection of the Cadweld Rebar Splice. This visual inspection included both ends of the sleeve, the tap hole, and measurement of void area. In addition, at least twice daily for each Cadweld crew, an inspector observed the entire splicing operation including cleaning of rebar ends, spacing of rebar, centering of rebar ends, loading the crucible,	2. Visual Inspection - All completed mechanical splices should be inspected at both ends of the splice sleeve and at the tap hole in the center of the splice sleeve in accordance with the requirements specified by the designer of the containment structure and approved by the licensee. Among the items should be included in these specifications are longitudinal centering of sleeve on the spliced ends, allowable voids in filler metal, extent of leaking of filler metal,

TABLE 3.8-2

Diablo Canyon Power Plant	Safety Guide 10
and firing the charge. The Cadweld procedure specified for the DCPP includes placing a mark 12 inches ± 1/4-inch back from the end of the bar. This line was used as a reference to determine if the bar ends are centered in the sleeve.	permissible gap between rebar ends, cartridge size, gas blowout, amount of packing and slag at the tap hole. Splices that fail to pass visual inspection should be discarded and replaced, and should not be used as tensile test samples.
Acceptance criteria for splice tensile tests is as follows:	3. Tensile Testing - Splice samples may be production splices (i.e., those cut directly from in place reinforcing) or sister splices (i.e., those removable splices made in-place next to production splices and under the same conditions).
No splice in the test series may have a tensile value below 125% of the specified yield point stress, and no more than 5 % of the splices tested may have an ultimate tensile strength less than 85% of that specified. The average tensile strength of all splices in the test series must equal or exceed the ASTM specified minimum ultimate strength.	Splice samples should be subjected to tensile tests in accordance with the sampling frequency specified in Paragraph 4a or Paragraph 4b below, to determine conformance with the following acceptance standards:
	a. The tensile strength of each sample tested should be equal or exceed 125 percent of the minimum yield strength specified in the ASTM standard appropriate for the grade of reinforcing bar using loading rates set forth in ASTM Specification A 370 dated August 15, 1968.
	b. The average tensile strength of each group of 15 consecutive samples should equal or exceed the guaranteed ultimate tensile strength specified for the reinforcing bar.

TABLE 3.8-2

Diablo Canyon Power Plant	Safety Guide 10
	If any sample tested fails to meet the provisions of Paragraph 3a above, the procedure of Paragraph 5a below should be followed. If the average tensile strength of the 15 samples tested fails to meet the provisions of Paragraph 3b above, the procedure of Paragraph 5b below should be followed.
Testing frequency for each crew, position, and grade of bar was as follows: One out of the first 10 splices. This splice must be a production splice for No. 18, Grade 60 bars and a sister splice for other sizes and grades of bar. Three out of the next 90 splices for No. 18, Grade 60 bars and one out of the next 90 splices for all other sizes and grades of bar. Three out of second and subsequent 100 splice units for No. 18, Grade 60 bars and one out of second and subsequent 100 splice units for all other sizes and grades of bar. At least 25% of the total number of No. 18, Grade 60 test splices must be made by cutting out	4. Tensile Test Frequency - Separate test cycles should be established for mechanical splices in horizontal, vertical, and diagonal bars, for each bar size, and for each splicing crew as follows: a. Test Frequency for Production Splice Test Samples. If only production splices are tested, the sample frequency should be: 1 of the first 10 splices 1 of the next 90 splices 2 of the next and subsequent units of 100 splices b. Test Frequency for Combinations of Production and Sister Splices. If production and sister splices are tested, the sample frequency should be:

TABLE 3.8-2

Diablo Canyon Power Plant	Safety Guide 10
production splices on a random basis. The remaining test splices may be made by having test bars tie wired alongside the production bars and spliced in sequence with those bars. The minimum length of the spliced bars is 3 feet.	1 production splice of the first 10 production splices 1 production and 3 sister splices, for the next 90 production splices 3 splices, either production or sister splices, for the next and subsequent units of 100 splices. At least 1/4 of the total number of splices tested should be production splices.
In the event a splice should fail the tensile test criteria, the specimen was to be examined by a testing laboratory. Based on the results of this investigation, additional splices by the crew responsible, as directed by the Engineer, were to be taken from the structure to ensure that there are no other defective splices. The procedures of the crew responsible for making the failed splice were to be reviewed, and if necessary, the crew retrained and requalified.	5. Procedure for Substandard Tensile Test Results a. If any production or sister splice tested fails to meet the tensile test specification of Paragraph 3a and the observed rate of splices that fail the tensile test at that time does not exceed 1 for each 15 consecutive test samples, the sampling procedure should be started anew. If any production or sister splice used for testing fails to meet the tensile test specification in Paragraph 3a, and the observed rate of splices that fail the tensile test exceeds 1 for each 15 consecutive test samples, mechanical splicing should be stopped. In addition, the adjacent production splices on each side of the last failed splice and 4 other splices distributed uniformly throughout the balance of the 100 production splices under investigation should be tested,

TABLE 3.8-2

Diablo Canyon Power Plant	Safety Guide 10
	and an independent laboratory analysis should be made to identify the cause of all failures. The results of these tests should be evaluated by the designer of the containment structure and the licensee to determine the required corrective action. The designer and the licensee should specify the extent of repairs necessary and the actions required to prevent further failures from the identified causes. b. If two or more splices from any of these 6 additional splice samples fail to meet the tensile test specification of Paragraph 3a, the balance of the 100 production splices under investigation should be rejected and replaced. When mechanical splicing is resumed, the sampling procedure should be started anew. If the average tensile strength of the 15 consecutive samples fails to meet the provisions of paragraph 3b above, the designer of the containment structure and the licensee should evaluate and assess the acceptability of the reduced average tensile strength with respect to the required strength at the location from which the samples were taken.

TABLE 3.8-3

NONDESTRUCTIVE EXAMINATION OF PRIMARY CONTAINMENT LINER WELDS
COMPARISON OF PROGRAM USED ON DIABLO CANYON POWER PLANT WITH SAFETY GUIDE 19

Diablo Canyon Power Plant	Safety Guide 19
	1. Nondestructive Examination of Liner Seam Welds
For each welder and welding position, the first 10 feet of weld was examined radiographically. Thereafter, a minimum of 10% of the welding (to at least include all intersections of joints) was progressively examined radiographically as welding was performed. This was done on a random basis with the location specified in such a manner that an approximately equal number of radiographs were taken from the work of each welder. The techniques of radiographic examination of welds were in accordance with Paragraph UW-51 of Section VIII, ASME Boiler and Pressure Vessel Code (ASME B&PV Code). See Notes 1 and 2.	a. For each welder and welding position (flat, horizontal, and overhead), the first 10 feet of weld, and one spot (not less than 12 inches in length) in each additional 50 foot increment of weld (weld test unit) or fraction thereof should be examined radiographically in accordance with the techniques prescribed in Section V, "Nondestructive Examination," of the ASME Boiler and Pressure Vessel Code (ASME B&PV Code). In any case, a minimum of 2 percent of all liner seam welds should be examined by radiography.
Where radiographic examination of liner seal welds was not feasible, a minimum 10% of the welding (to at least include all locations where there are welded backing strip splices and intersections) was examined by magnetic particle or liquid penetrant testing. Magnetic particle testing was in accordance with Appendix VI of Section VIII, ASME B&PV Code. Liquid penetrant testing was in accordance with Appendix VIII of Section VIII, ASME B&PV Code. See Notes 1 and 2.	b. Where radiographic examination of liner seam welds is not feasible or where the weld is located in areas which will not be accessible after construction, the entire length of weld should be examined by the magnetic particle method or by the ultrasonic method in accordance with the techniques prescribed in Section V of the ASME BP&V Code for such examination methods.

TABLE 3.8-3

Diablo Canyon Power Plant	Safety Guide 19
All liner seam welds were tested for leaktightness in accordance with the following method:	c. All liner seam welds should be tested for leaktightness in accordance with the following method (or other methods of equivalent sensitivity):
Immediately preceding the test, a soap solution is applied to the weld. The application of the soap solution must not precede the vacuum box by more than 3 minutes. The vacuum box, which contains a viewing window, is placed over the area to be tested and evacuated to a 5 psi differential with the atmospheric pressure.	Immediately preceding the test, a soap solution (or other appropriate solution) should be applied to the weld. A vacuum box containing a viewing window should be placed over the area to be tested and evacuated to produce at least 5 psi differential with the atmospheric pressure. Leaks in welds, if present, should be detected by formation of bubbles. The solution used for the test should have bubble formation properties adequate for identification of leaks. The test solution should be checked every hour, with a suitable test leak to verify the bubble formation property of the solution used.
Leak chase channels are installed over the liner welds. Upon completion of one zone of leak chase channels, the zone was tested at the containment structure design pressure of 47 psi. The acceptance criteria is that there be no loss of pressure within 2 hours as indicated by a pressure gauge.	d. Where leak chase system channels are installed over liner welds, channel-to-liner plate welds should be tested for leak- tightness by pressurizing the channels to containment design pressure. If any indicated loss of channel test pressure occurs within 2 hours, as evidenced by a test gauge, the channel-to-liner welds should be soap bubble tested in accordance with the above procedure.

TABLE 3.8-3

Diablo Canyon Power Plant	Safety Guide 19
	2. Nondestructive Examination of Penetration, Airlock, and Access Opening Welds
All welds in penetration, airlocks, and access openings that are not backed by concrete were fully examined in accordance with Class B requirements of Section III, ASME B&PV Code. See Notes 1, 2, 3, and 4. All welds between flued heads and pipelines were fully examined in accordance with the Class II requirements of ANSI B 31.7, Nuclear Power Piping.	a. All welds in penetration, airlocks, and access openings that are not backed by concrete, such as welds between penetrations and flued fittings and pipelines, should be fully examined in accordance with examination methods of NE-5120 of Section III of the ASME B&PV Code employing the techniques prescribed in Section V of that code.
Welds backed by concrete in the vicinity of penetrations were examined as follows: 1. Welds between the penetration sleeve and insert plate were fully examined in accordance with the Class B requirements of Section III, ASME B&PV Code. See Notes 1, 2, and 4. 2. Welds between the insert plate and the liner were examined under the same criteria as liner seam welds. All welds backed by concrete in the containment structure are carbon steel.	b. All welds in the vicinity of penetrations and access openings that are backed by concrete, such as welds between penetration and reinforcing plate,[a] penetration and liner, reinforcing plate and liner, liner insert and liner, reinforcing plate and frames for airlocks and access openings, and liners and frames for airlocks and access openings, should be fully examined (1) in accordance with Paragraph 2a above or (2) by magnetic particle, or liquid penetrant when a nonmagnetic weld is used, in accordance with the techniques prescribed in Section V of the ASME B&PV Code.

(a) Thickened liner insert which provides local reinforcement.

TABLE 3.8-3

Diablo Canyon Power Plant	Safety Guide 19
Examination of welds in penetrant assemblies and in the vicinity of penetrations is described in the preceding paragraphs.	c. All welds in bellow type expansion joints provided in penetration assemblies or appurtenances to the containment vessel should be magnetic-particle or liquid-penetrant tested when a nonmagnetic weld is used, in accordance with the techniques prescribed in Section V of ASME B&PV Code for such examination methods.
	3. <u>Qualification of Welders and Welding Procedures</u>
The qualification of welders, welding machine operators, and welding procedures was in accordance with Section IX, "Welding Qualifications," of the ASME B&PV Code. See Note 2.	The qualification of welders, welding machine operators, and welding procedures should be in accordance with Section IX, "Welding Qualifications," of the ASME B&PV Code.
	4. <u>Qualification of Nondestructive Examination Personnel</u>
Nondestructive examinations were performed by personnel qualified in accordance with the appropriate parts of the ASME B&PV Code. See Notes 1 and 2.	Nondestructive examination should be performed by personnel designated by the licensee or his agent and qualified in accordance with the provisions of Section V of the ASME B&PV Code.
	5. <u>Selection of Spots for Radiographic Examination</u>
The spots of liner seam welds to be radiographically examined were selected on a random basis with the locations selected such that all intersections	The spots of liner seam welds to be radiographically examined should be randomly selected, but no two spots in adjacent weld test units

Diablo Canyon Power Plant	Safety Guide 19
of joints were examined, and an approximately equal number of radiographs were taken from the work of each welder. The location covered by each radiograph was recorded.	should be closer than 10 feet and their locations should be recorded.
	6. <u>Time of Examination</u>
Nondestructive examinations were done progressively as welding was performed.	All examinations should be performed as soon as practicable after the linear increment of weld to be examined is completed.
	7. <u>Acceptance Standards</u>
	a. <u>Containment Liner Seam Welds Examined by Radiography</u>
Where a spot in the seam weld is judged acceptable in accordance with Paragraph UW-51 of Section VIII, ASME B&PV Code, the entire weld test unit represented by this spot radiograph is considered acceptable. See Notes 2 and 3.	Where a spot in the seam weld is judged acceptable in accordance with the referenced standards of NE-5120 of Section III of the ASME B&PV Code, the entire weld test unit represented by this spot radiograph is considered acceptable.
	b. <u>Containment Liner Seam Welds Examined by Ultrasonic or Magnetic Particle</u>
Where a spot in the seam weld examined by magnetic particle or liquid penetrant method is judged	Seam welds examined by ultrasonic or magnetic particle methods are considered acceptable

TABLE 3.8-3

Diablo Canyon Power Plant	Safety Guide 19
acceptable, in accordance with the acceptance criteria referenced in Section VIII, ASME B&PV Code, the entire weld seam represented by the examination is considered acceptable. See Notes 2 and 3.	provided the examinations meet the acceptance standards referenced for such examination methods in NE-5120 of Section III of the ASME B&PV Code.
	c. <u>Soap Bubble Leak Tests of Containment Liner Welds</u>
The acceptance criterion for the vacuum box test is that no leaks be detected.	Liner welds are considered acceptable provided no leakage is detected by soap bubble tests (or by other methods of equivalent sensitivity).
	d. <u>Penetration, Airlock, and Access Opening Welds</u>
Penetration, airlock, and access opening welds that are not backed by concrete are considered acceptable provided the examinations meet the acceptance standards referenced for Class B vessels in Section III, ASME B&PV Code. See Notes 2, 3, and 4. Welds between flued heads and pipelines are considered acceptable provided the examinations meet the acceptance standards referenced for Class II piping in ANSI B31.7, Nuclear Power Piping.	Penetration, airlock, and access opening welds are considered acceptable provided the examinations meet the acceptance standards referenced in NE-5120 of Section III of the ASME B&PV Code. Welds in bellows type expansion joints are considered acceptable if the examinations meet the acceptance standards referenced in magnetic particle and liquid penetrant methods in NE-5120 of Section III.

Diablo Canyon Power Plant	Safety Guide 19
Welds between the penetration sleeve and insert plate are considered acceptable provided the examinations meet the acceptance standards referenced for Class B vessels in Section III, ASME B&PV Code. See Note 2.	
	8. Repair and Reexamination
	a. Containment Liner Seam Welds Examined by Radiography
If a radiographed spot failed to meet the specified acceptance standards, two additional spots of the same length were radiographically examined in the same weld seam at locations away from the original spot, but in welds performed by the same welder or welder operator. The locations of these additional spots were determined as provided for the original spot examination.	When a radiographed spot fails to meet the specified acceptance standards, two additional spots should be radiographically examined in the same weld test unit at locations at least one foot removed (on each side) from the original spot. The locations of these additional spots should be determined by the examiner using the same procedure followed in the selection of the original spot for examination and the examination results should determine the following corrective actions:
If the two additional spots examined showed welding that meets the specified acceptance standards, the entire weld represented by the three radiographs is judged acceptable. The defective welding disclosed by the first of the three radiographs was removed and repaired.	(1) If the two additional spots examined meet the specified acceptance standards, the entire weld unit represented by the three spot radiographs is considered acceptable. However, the defective welding disclosed by the first of the three radiographs should be repaired by welding.

TABLE 3.8-3

Diablo Canyon Power Plant	Safety Guide 19
If either of the two additional spots examined showed welding that does not comply with the specified acceptance standards, the entire portion of the seam represented was considered unacceptable or, optionally, the entire weld represented was completely radiographed and defective welding corrected to meet the specific acceptance standards.	(2)If either of the two additional spots examined fails to meet the specified acceptance standards, the entire weld test unit is considered unacceptable. The entire weld should be removed and the joint should be rewelded or, optionally, the entire weld unit may be completely radiographed and defective welding only need be repaired.
Repair welding was performed using a qualified procedure. The rewelded joints or weld repaired areas were completely reradiographed and meet the specified acceptance standards.	(3)Repair welding should be performed using a procedure as specified under regulatory position 3. above. The weld repaired areas in each weld test unit should be spot radiographed at one selected location to meet the acceptance criteria specified in regulatory position 7.a. or 8.a. (1). above.
	b.<u>Containment Liner Seam Welds Examined by Ultrasonic or Magnetic Particle</u>
If a weld that had been examined did not comply with the specified acceptance standards, additional examination was performed to the same extent as required for radiography. The weld was repaired and reexamined in accordance with the provisions of Section VIII of the ASME B&PV Code. See Notes 2 and 3.	When a weld which has been examined does not comply with the specified acceptance standards, the weld should be repaired and reexamined in accordance with the provisions of Section III of the ASME B&PV Code.

TABLE 3.8-3

Diablo Canyon Power Plant	Safety Guide 19
	c. Soap Bubble Tests of Containment Liner Welds
If a weld was judged unacceptable because leakage is repaired. Repair welding was performed using a procedure qualified as specified for production welds. The weld repaired areas were reexamined by soap bubble leakage retesting.	Welds judged unacceptable because leakage is detected by the soap bubble test (see regulatory position 7.c. above) should be repaired. Repair welding should be performed using a qualified procedure as specified under regulatory position 3. above. The weld repaired areas should be reexamined by soap bubble leakage retesting.
	d. Penetration, Airlock and Access Opening Welds
If a weld was judged unacceptable on a penetration sleeve airlock, or access opening, the weld was repaired and reexamined in accordance with the provisions for Class B vessels of Section III of the ASME B&PV Code. See Notes 2 and 3.	Welds judged acceptable in accordance with regulatory position 7.d. should be repaired and reexamined in accordance with the provisions of Section III of the ASME B&PV Code.
	9. Records
Retention of records is discussed in Chapter 17.	Records of radiographs and other nondestructive examinations including those for repaired defective welds should be retained by the licensee in compliance with the provisions of Section XVII, "Quality Assurance Records," of Appendix B to 10 CFR Part 50, "Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants."

TABLE 3.8-3

Notes:

1. Section V, ASME B&PV Code, which provides techniques for nondestructive examination applicable to all sections of the ASME B&PV Code, was first published in July 1971. Although it may eventually replace the corresponding parts of other sections of the ASME B&PV Code, the individual sections still contain techniques for nondestructive examinations.

2. References in the table to ASME B&PV Code for the Diablo Canyon plant refer to 1968 Edition, including addenda through Summer 1968.

3. NE-5120, Section III, ASME B&PV Code, requires examination technique and acceptance criteria in accordance with Section VIII, ASME B&PV Code (Paragraph UW-51 for radiography).

4. Class B requirements of Section III, ASME B&PV Code, specify radiographic examination and acceptance criteria in accordance with Paragraph UW-51, Section VIII, ASME B&PV Code.

TABLE 3.8-4

CONTAINMENT BUILDING
BASE SLAB STRESS RATIOS[a]

	Operating		Accident		Accident With Hosgri		
Beam No.[b]	Moment Demand (ft-k)	Moment Capacity (ft-k)	Moment Demand (ft-k)	Moment Capacity (ft-k)	Moment Demand (ft-k)	Moment Capacity (ft-k)	Stress Ratio
62	15,293 --	37,640 -50,600	12,827 -37,772	36,925 -156,900	20,134 -44,967	41,250 -175,875	2.05
225	59,630 --	88,835 -18,275	75,492 --	279,275 -54,375	109,844 --	314,000 -61,083	1.49
43	12,126 --	51,880 -18,275	56,623 --	167,925 -54,375	69,444 --	188,124 -61,083	2.71
24	14,483 --	25,940 -9,138	64,157 --	83,963 -27,188	68,263 --	94,062 -30,541	1.31
129	23,425 --	88,835 -18,275	76,480 --	279,275 -54,375	93,238 --	314,000 -61,083	3.65
216	23,013 --	37,640 -50,600	19,780 -21,785	36,925 -156,900	26,007 -67,471	41,250 -175,875	1.59
171	1,898 -6,884	6,675 -30,360	-- -37,497	22,155 -94,140	5,831 -55,206	24,750 -105,525	1.91

(a) Stress Ratio = $\frac{\text{Capacity}}{\text{Demand}}$

(b) See Figure 3.8-38: + = Tension on bottom - = Tension on top

TABLE 3.8-5

CONTAINMENT BUILDING INTERNAL STRUCTURE
STRESS RATIOS IN SELECTED ELEMENTS

Description of Member	Location of Member	Load Combination[a]	Demand	Capacity	Stress Ratio[b]
Rebar in 3-ft concrete wall	Crane wall:				
	1. Vertical bar	D + L + DDE + CP + R + J + M	58 ksi	60 ksi	1.03
	2. Hoop bar		54 ksi	60 ksi	1.11
4-ft concrete wall	Fuel transfer canal:				
	1. Wall @ N & S from el 113 ft-1 1/2 in. to el 140 ft	D + L + DDE + CP + R + J + M	343 k-ft	381 k-ft	1.11
	2. Wall @ W from el 113 ft-1 1/2 in. to el 140 ft	D + L + DDE + CP + R + J + M	162 k-ft	216 k-ft	1.33
Rebar in 2-ft concrete wall	Fuel transfer canal wall @ W from el 88 ft to el 113 ft-1 1/2 in.	D + L + DDE + CP + R + J + M	22 ksi	60 ksi	2.72
3-ft concrete slab	Fuel transfer canal floor @ el 113 ft-1 1/2 in.	D + L + DDE + CP + R + J + M	258 k-ft	269 k-ft	1.04
4-ft Concrete Slab	Fuel transfer canal floor @ el 104 ft	D + L + DDE + CP + R + J + M	306 k-ft	381 k-ft	1.24
Rebar in 6-ft concrete wall	Reactor cavity wall:				
	1. Vertical bar	D+L+DDE+CP+R+J+M	15 ksi	60 ksi	4.0
	2. Hoop bar	D + L + DDE + CP + R + J + M	26 ksi	60 ksi	2.3
3-ft concrete slab	Floor @ el 140 ft	D + L + DE + T	46 k-ft	93 k-ft	2.02

(a) Load combinations with Hosgri do not govern.
(b) Stress Ratio $= \frac{\text{Capacity}}{\text{Demand}}$

TABLE 3.8-5

Description of Member	Location of Member	Load Combination[a]	Demand	Capacity	Stress Ratio[b]
4 ft 6 in. concrete slab	Floor @ el 140 ft	D + L + DE + T	156 k-ft	229 k-ft	1.46
5 ft concrete slab	Floor @ el 140 ft	D + L + DDE + CP + R + J + M	328 k-ft	500 k-ft	1.52
10 in. concrete slab	Annulus platform @ el 130 ft	D + L + DE + T	10 k-ft	39 k-ft	3.90
1 ft 6 in. concrete slab	Annulus platform @ el 140 ft	D + L + DE + T	35 k-ft	57 k-ft	1.62
W21x73	Annulus platform @ el 130 ft	D + L + DE + T + TH + FV + RVOT	22 ksi	24 ksi	1.09
W21x62	"	"	8 ksi	22 ksi	2.75
W12x40	"	"	9 ksi	22 ksi	2.44
W12x65	Annulus platform column	"	124 k (Unit 1)	268 ksi	2.16
W12x65	"	"	205 k (Unit 2)	268 ksi	1.31
W12x99	"	"	212 k (Unit 1)	366 k	1.73
W21x55	Annulus platform @ el 140 ft	"	24 ksi (Unit 1)	27 ksi	1.12
W21x82	"	"	11 ksi (Unit 1)	22 ksi	2.00
W21x68	"	"	17 ksi (Unit 1)	22 ksi	1.29
W21x96	"	"	23 ksi (Unit 1)	27 ksi	1.17

TABLE 3.8-5

Description of Member	Location of Member	Load Combination[a]	Demand	Capacity	Stress Ratio[b]
W24x100	Annulus platform @ el 140 ft	D + DDE + THA + FV + RVOT	28 (Unit 2)	37.4	1.34
W21x96	"	"	18 (Unit 2)	37.4	2.08
W21x68	"	"	21 (Unit 2)	37.4	1.78
W21x55	"	"	16 (Unit 2)	37.4	2.33
W24x100	"	D + HE	21 (Unit 2)	44.8	2.13
W21x96	"	"	20 (Unit 2)	44.8	2.21
W21x68	"	"	17 (Unit 2)	44.8	2.64
W21x55	"	"	13 (Unit 2)	44.8	3.45
W12x65	Annulus platform column	D + DDE + THA + FV + RVOT	260 (Unit 2)	45.6	1.75
W12x99	"	"	270 (Unit 2)	45.6	1.69
W12x65	"	D + HE	357 (Unit 2)	55.7	1.56
W12x99	"	D + HE	365 (Unit 2)	55.7	1.53

TABLE 3.8-5A

CONTAINMENT BUILDING PIPEWAY STRUCTURE STRESS RATIOS IN SELECTED MEMBERS

Description of Member	Location of Member		Load Combination	Stress Ratio[a]
	Unit	Elevation		
W8 x 40	1	114	D + DE	1.10
W8 x 40	1	114	D + DE	1.07
W8 x 40	1	114	D + DE	1.07
W14 x 111	2	109	D + DE	1.72
W14 x 111	2	109	D + DE	1.79
W14 x 111	2	109	D + DE	1.59
W8 x 40	1	114	D + DDE	1.11
W8 x 40	1	114	D + DDE	1.20
W8 x 40	1	114	D + DDE	1.20
W14 x 111	2	109	D + DDE	2.78
W14 x 111	2	109	D + DDE	1.89
W14 x 111	2	109	D + DDE	1.92
W8 x 40	1	114	D + DDE + Yr	1.06
W14 x 111	1	119	D + DDE + Yr	1.27
W14 x 202	1	119	D + DDE + Yr	1.32
W14 x 202	2	114	D + DDE + Yr	1.52
W8 X 17	2	109	D + DDE + Yr	1.11
W14 x 111	2	109	D + DDE + Yr	1.25
W8 x 40	1	114	D + HE	1.05
W8 x 40	1	114	D + HE	1.09
W8 x 40	1	114	D + HE	1.05
W14 x 111	2	109	D + HE	1.03
W10 x 31	2	109	D + HE	1.06
W12 x 106	2	138	D + HE	1.11

(a) Stress ratio $= \frac{\text{Capacity}}{\text{Demand}}$

TABLE 3.8-5B

CONTAINMENT AND AUXILIARY BUILDINGS COMPARISON OF DISPLACEMENTS AND SEPARATIONS

Elevation (ft)	Maximum Relative Seismic Displacement[a]		Minimum Separation (in.)[c]	Minimum Factor of Safety Against Contact[e]
	DDE	HE		
188	6.76	9.59	22	2.06
140	0.44	0.37	8	5.29
115	0.27	0.23	2[d]	3.05
100	0.17	0.11	1.25[d]	4.12

(a) Maximum relative seismic displacements are calculated as sum of maximum containment and auxiliary building displacements.

(b) Not Used.

(c) Minimum separation is measured at normal ambient temperature and pressure.

(d) Except for a few localized areas, the minimum separation is 4 inches at elevations 100 ft and 115 ft.

(e) The factor of safety is determined from the relative seismic displacements after the thermal and pressure effects are conservatively accounted for.

TABLE 3.8-6

VERIFICATION OF COMPUTER PROGRAMS

Program Name	General Function	Verification Measure
AISCBM/CE401	Analysis, design, and investigation of structural steel framing system in accordance with AISC specification	Bechtel Verification Manual
ANSR	Linear/nonlinear static and dynamic analysis finite-element program	URS/Blume QA Manual
AXIDYN	Static and dynamic analysis of axisymmetric structures	URS/Blume QA Manual
BLUME SAP IV	General-purpose linear elastic finite-element static and dynamic analysis	URS/Blume QA Manual
BSAP/CE800	General-purpose linear elastic finite-element static and dynamic analysis	Bechtel Verification Manual
BSAP-POST/ CE201&CE217	Postprocessing for BSAP computer program	Bechtel Verification Manual
CECAP/CE987	Computes stress in rebars and liner plate by considering cracking in concrete	Bechtel Verification Manual
Drain-2D	Nonlinear 2-D static and dynamic analysis	URS/Blume QA Manual
FINEL/CE801	Performs finite element static analysis by considering cracking and yielding	Bechtel Verification Manual
LOCAL STRESS/ME210	Calculates local stress in cylindrical shells due to external loading	Bechtel Verification Manual
SMIS	Matrix manipulation program	URS/Blume QA Manual
SPECTRA/CE802	Computes response spectra from acceleration time-histories	Bechtel Verification Manual
STAND/ME425	Design and evaluation of pipe support base plate with concrete anchor bolt assemblies	Bechtel Verification Manual

TABLE 3.8-6

Program Name	General Function	Verification Measure
THERMAL STRESS/ ME643	Performs thermal and stress analysis for 2-D plane or axisymmetric structures	Bechtel Verification Manual
BECHTEL ANSYS/ CE798	Large general-purpose linear/nonlinear static and dynamic analysis	Bechtel Verification Manual
BECHTEL STRUDL/ CE901	Finite element static/dynamic analysis, and design of structures	Bechtel Verification Manual
EASE2/E2SPEC	Linear elastic finite-element static and dynamic analysis computer program	Verification by Control Data Corporation
PG&E STRUDL	General purpose static and dynamic structural analysis	Partial verification of program as originally received. Complete verification performed on a case-by-case basis for each application.
GTSTRUDL/CE701	General-purpose static and dynamic finite element code	Verification by Control Data Corporation
PIPERUP	Performs nonlinear elastic/plastic analysis of 3-D piping system subject to static/dynamic time-history forcing functions	Nuclear Services Corporation Verification Manual
STARDYNE/CE991	General-purpose finite-element static and dynamic analysis	Bechtel Verification Verification
RAP	Pipe whip restraint design program	Nuclear Services Corporation Verification Manual
WECAN	Modal superposition time history analysis and static analysis of structure	Westinghouse Verification Manual
ADDA	Postprocessor to sum the time history responses for two different sets of modes	Westinghouse Verification Manual

TABLE 3.8-6

Program Name	General Function	Verification Measure
TAPES	To reformat the time history response tapes from ADDA for input to GENSPC	Westinghouse Verification Manual
GENSPC2	To calculate the modal response spectra	Westinghouse Verification Manual
COMBSPC	To combine the response spectra by SRSS	Westinghouse Verification Manual
SPREAD	To transform the scale of the spectra from frequency to period and plot the combined response spectra	Westinghouse Verification Manual
MARG3	Qualification analysis of structure	Westinghouse Verification Manual
SAP90	General-purpose linear elastic finite-element static and dynamic analysis	Computers and Structures, Inc. Verification Manual
SAP2000	General-purpose linear and non-linear finite-element program for static and dynamic analysis	Computers and Structures, Inc. Verification Manual
PC-SPECTRA	Pre- and post-processor for response spectra data	PG&E Nuclear Computer Program Acceptance Report
PC-ANSR	Linear/Nonlinear static and dynamic analysis finite-element program	PG&E Calculation No. 2252C-1 (Reference 40)

TABLE 3.8-6A

AVERAGE CONCRETE STRENGTH

CONTAINMENT AND INTERIOR STRUCTURE

Component	Average f_c' (test value) (psi)	E_c[a] (psi)
Base slab to elevation 87 ft	6330	4.53×10^6
Skin pour at elevation 89 ft	6330	4.53×10^6
Interior	6330	4.53×10^6
Skin pour	3850	3.54×10^6
Soldier beams	3850	3.54×10^6
Exterior walls	3850	3.54×10^6
Dome	3850	3.54×10^6

(a) $E_c = 57{,}000\ (f_c')^{1/2}$ per AC1 318-71, in psi

TABLE 3.8-6B

STEEL STRENGTH DATA

CONTAINMENT AND INTERIOR STRUCTURE

Structure	Designation of Steel	Yield (psi) Minimum	Yield (psi) Average	Ultimate (psi) Minimum	Ultimate (psi) Average
		Reinforcing Steel			
Containment (1 and 2) Exterior #18s	ASTM 615, Grade 60	61,750	66,854	93,750	105,992
Containment (1 and 2) Interior #11s	Grade 60	62,820	68,079	96,795	105,556
		Structural Steel			
ASTM	A36	36,100	43,950	58,200	68,040
ASTM	A441	42,100	51,620	67,200	75,910
ASTM	A516	45,800	51,040	72,200	79,170

TABLE 3.8-7

AUXILIARY BUILDING
FUEL HANDLING CRANE SUPPORT STRUCTURE
STRUCTURAL MEMBERS AND ANCHORAGES STRESS RATIOS

	Description Of Members And Their Functions	Stress Ratios[(a) (b)] Hosgri	DDE	DE
BRACES	Top Chord Roof	1.7	1.5	1.9
	Bottom Chord Roof	1.1	1.2	1.3
	East-West Elevation Diagonals	1.1	1.1	1.4
	N-S Truss Diagonals, West and East Exterior/Interior	1.9	≥ 1.6	≥ 2.0
	East-West Truss Diagonals	1.1	≥ 1.3	≥ 1.1
	East-West Truss Knee	1.1	≥ 1.3	≥ 1.5
CHORDS	North South Trusses, Top	3.4	4.0	4.2
	North South Trusses, Bottom	3.2	4.0	4.3
	East-West Trusses, Top	1.1	1.6	1.9
	East-West Trusses, Bottom	1.1	1.4	1.8
LATERAL & VERTICAL	Frame Horizontals, West and East Sides	4.5	4.8	5.9
	Vertical Columns	1.0	1.1	1.0
	East-West Truss Struts	1.2	1.4	1.6
BASE ANCHORAGE	Axial Tensions	1.0	1.1	1.6
	Axial Compressions	1.3	1.4	1.7
	Lateral Shears	1.9	2.2	2.3

(a) Stress Ratio = $\frac{\text{Capacity}}{\text{Demand}}$

(b) Refer to Reference 39 for stress ratios.

TABLE 3.8-8

AUXILIARY BUILDING SLABS STRESS RATIOS[a][b]
OUT-OF-PLANE LOADS (DE)

Slab Location	Member	Shear, psi Demand	Shear, psi Capacity	Moment, kips-ft Demand	Moment, kips-ft Capacity	Stress Ratio
El 73 ft Area bounded by column[c] lines H, U, 15.7, 16.8	Slab	49	60	24	30	1.2
	Beam	48	60	100	140	1.2
El 85 ft Area bounded by column lines H, U, 16.8, 19.2	Slab	64	78	82	120	1.2
El 100 ft & 115 ft Area bounded by column[c] lines H, T, 10.7, 15.7	Slab	48	78	75	88	1.2
	Beam	70	120	340	470	1.4
El 115 ft Area bounded by column lines U, V, 15.7, 20.3	Slab	45.0	78.0	27.0	32.00	1.2
El 115 ft Area bounded by column lines H, L.5, 15.7, 20.3	Slab	59	78	56	67	1.2
	Beam	170	210	2,000	2,400	1.2
El 140 ft Area bounded by column[c] lines H, T, 10.7, 15.7	Slab	120	130	2,050	2,390	1.1
	Beam	130	140	1450	1640	1.1
El 140 ft Area bounded by column[c] lines R, V, 15.7, 17.4	Slab	55	78	59	77	1.3
	Beam	89	97	2,010	2,240	1.1
El 140 ft Area bounded by column lines H, L, 15.7, 20.3	Slab	57.0	78.0	382.0	455.00	1.2
El 154-1/2 ft Area bounded by column lines L, R, 15.7, 17.4	Slab	30	78	14	17	1.2
	Beam	60	78	76	100	1.3
El 163-1/3 ft Area bounded by column lines H, L, 15.7, 20.3	Composite[d] Beam	9,300	14,000	2,260	2,300	1.02

(a) Stress ratio $= \frac{\text{Capacity}}{\text{Demand}}$ for shear or moment, whichever is smaller.

(b) This does not include the effects of pipe break loads which are evaluated locally in accordance with provisions of Reference 6.

(c) Counterpart in Unit 2 is similar.

(d) These values are for structural steel beams embedded in the slab.

TABLE 3.8-9

AUXILIARY BUILDING SLABS STRESS RATIOS[(a)b)]
OUT-OF-PLANE LOADS (DDE)

Slab Location	Member	Shear, psi Demand	Shear, psi Capacity	Moment, kips-ft Demand	Moment, kips-ft Capacity	Stress Ratio
El 73 ft Area bounded by column[(c)] lines H, U, 15.7, 16.8	Slab	53	91	27	56	1.7
	Beam	53	93	110	260	1.8
El 85 ft Area bounded by column lines H, U, 16.7, 19.2	Slab	72	120	93	220	1.7
El 100 ft & 115 ft Area bounded by column[(c)] lines H, T, 10.7, 15.7	Slab	51	120	81	167	2.1
	Beam	130	200	360	900	1.5
El 115 ft Area bounded by column lines U, V, 15.7, 20.3	Slab	49	120	30	62	2.1
El 115 ft Area bounded by column lines H, L.5, 15.7, 20.3	Slab	66	120	63	130	1.8
	Beam	190	330	2,200	4,600	1.7
El 140 ft Area bounded by column[(c)] lines H, T, 10.7, 15.7	Slab	120	190	2,110	4,310	1.6
	Beam	130	200	1,500	3,140	1.5
El 140 ft Area bounded by column[(c)] lines R, V, 15.7, 17.4	Slab	56	120	60	150	2.1
	Beam	93	150	2,110	4,030	1.9
El 140 ft Area bounded by column lines H, L, 15.7, 20.3	Slab	61	120	411	858	2.0
El 154-1/2 ft Area bounded by column lines L, R, 15.7, 17.4	Slab	32	120	15	33	2.2
	Beam	64	120	81	190	1.9
El 163-1/3 ft Area bounded by column lines H, L, 15.7, 20.3	Composite[(d)] Beam	11,000	20,000	2,200	3,100	1.4

(a) Stress ratio = $\frac{\text{Capacity}}{\text{Demand}}$ for shear or moment, whichever is smaller.

(b) This does not include the effects of pipe break loads which are evaluated locally in accordance with provisions of Reference 6.

(c) Counterpart in Unit 2 is similar.

(d) These values are for structural steel beams embedded in the slab.

TABLE 3.8-10

AUXILIARY BUILDING SLABS STRESS RATIOS[(a)(b)]
OUT-OF-PLANE LOADS (HE)

Slab Location	Member	Shear, psi Demand	Shear, psi Capacity	Moment, kips-ft Demand	Moment, kips-ft Capacity	Stress Ratio
El 73 ft						
Area bounded by column[(c)] lines H, U, 15.7, 16.8	Slab	65	110	32	56	1.7
	Beam	63	110	130	260	1.7
El 85 ft						
Area bounded by column lines H, U, 16.8, 19.2	Slab	91	130	120	270	1.4
El 100 ft & 115 ft						
Area bounded by column[(c)] lines H, T, 10.7, 15.7	Slab	68	130	110	200	1.8
	Beam	170	220	480	1,100	1.3
El 115 ft						
Area bounded by column lines U, V, 15.7, 20.3	Slab	60.0	130.00	36.0	74.00	2.0
El 115 ft						
Area bounded by column lines H, L.5, 15.7, 20.3	Slab	100	130	120	150	1.2
	Beam	300	360	4,300	5,500	1.2
El 140 ft						
Area bounded by column[(c)] lines H, T, 10.7, 15.7	Slab	160	200	2,720	5,220	1.3
	Beam	170	200	2,140	3,770	1.2
El 140 ft						
Area bounded by column[(c)] lines R, V, 15.7, 17.4	Slab	72	130	79	180	1.8
	Beam	145	160	3,280	4,870	1.1
El 140 ft						
Area bounded by column lines H, L, 15.7, 20.3	Slab	105.0	130.00	710.0	1,062.00	1.2
El 154-1/2 ft						
Area bounded by column[(c)] lines L, R, 15.7, 17.4	Slab	65	130	29	41	1.4
	Beam	92	130	150	230	1.4
El 163-1/3 ft						
Area bounded by column lines H, L, 15.7, 20.3	Composite[(d)] Beam	16,000	24,000	3,900	4,200	1.1

(a) Stress ratio = $\frac{\text{Capacity}}{\text{Demand}}$ for shear or moment, whichever is smaller.

(b) This does not include the effects of pipe break loads which are evaluated locally in accordance with provisions of Reference 6.

(c) Counterpart in Unit 2 is similar.

(d) These values are for structural steel beams embedded in the slab.

TABLE 3.8-11

AUXILIARY BUILDING SLABS STRESS RATIOS[f]
IN-PLANE LOADS (DE)

Elevation, ft	Section[e][g] Number	Shear[c], kip		Moment[d], k-ft		Stress Ratio
		Demand	Capacity[a]	Demand	Capacity[b]	
100	1-a	60	2,200	2,650	25,300	9.5
100	1-b	170	2,300	5,700	21,600	3.8
100	2-a	320	2,880	9,500	91,900	9.0
100	2-b	110	1,060	700	6,350	9.1
100	2-c	10	220	50	600	>10.0
115	1	300	2,240	7,350	23,200	3.2
115	2-a	240	4,350	200	50,900	>10.0
115	2-b	120	490	2,100	4,100	2.0
115	3-a	260	1,260	2,650	7,700	2.9
115	3-b	280	1,280	2,150	18,100	4.6
115	4	1,460	8,100	29,700	289,000	5.5
140	1	380	2,240	2,500	4,800	1.9
140	2-a	320	4,130	5,100	64,000	>10.0
140	2-b	270	2,100	3,050	13,800	4.5
140	3-a	160	430	1,950	4,950	2.5
140	3-b	390	1,530	5,900	16,300	2.8
140	4	1,840	8,580	39,500	443,000	4.7

(a) Shear capacity is calculated for the section subjected to demand moment and axial force.
(b) Moment capacity is calculated for the section subjected to demand axial force.
(c) Shear capacities and demands are rounded to the nearest 10 kips or 3 significant digits.
(d) Moment capacities and demands are rounded to the nearest 50 ft-kips or 3 significant digits.
(e) Section location is shown on Figures 3.8-60 through 3.8-62.
(f) Stress ratio = $\frac{\text{Capacity}}{\text{Demand}}$ for shear or moment, whichever is smaller
(g) Component in Unit 2 is similar.

TABLE 3.8-12

AUXILIARY BUILDING SLABS STRESS RATIOS[f]
IN-PLANE LOADS (DDE)

Elevation, ft	Section[e][g] Number	Shear[c], Kip		Moment[d], k-ft		Stress Ratio
		Demand	Capacity[a]	Demand	Capacity[b]	
100	1-a	110	3,710	3,400	73,300	>10.0
100	1-b	310	3,770	9,850	63,100	6.4
100	2-a	610	4,800	14,900	303,000	7.9
100	2-b	200	1,770	1,250	17,000	8.9
100	2-c	10	370	50	1,100	>10.0
115	1	500	3,730	15,400	88,700	5.8
115	2-a	430	7,240	3,550	167,000	>10.0
115	2-b	240	1,130	4,500	16,500	3.7
115	3-a	330	2,090	2,900	12,000	4.1
115	3-b	450	2,140	2,350	42,600	4.8
115	4	2,920	13,500	60,000	1,000,000	4.6
140	1	750	3,730	4,250	43,300	5.0
140	2-a	840	6,890	6,150	200,000	8.2
140	2-b	510	3,500	5,550	36,200	6.5
140	3-a	240	720	2,550	10,300	3.0
140	3-b	590	2,550	8,900	46,400	4.3
140	4	3,700	14,300	79,000	1,110,000	3.9

(a) Shear capacity is calculated for the section subjected to demand moment and axial force.
(b) Moment capacity is calculated for the section subjected to demand axial force.
(c) Shear capacities and demands are rounded to the nearest 10 kips or 3 significant digits.
(d) Moment capacities and demands are rounded to the nearest 50 ft-kips or 3 significant digits.
(e) Section location is shown on Figures 3.8-60 through 3.8-62.
(f) Stress ratio = $\frac{\text{Capacity}}{\text{Demand}}$ for shear or moment, whichever is smaller
(g) Component in Unit 2 is similar.

TABLE 3.8-13

AUXILIARY BUILDING SLABS STRESS RATIOS[f]
IN-PLANE LOADS (HE)

Elevation, ft	Section[e][g] Number	Shear[c], Kip		Moment[d], k-ft		Stress Ratio
		Demand	Capacity[a]	Demand	Capacity[b]	
100	1-a	170	4,350	3,650	86,100	>10.0
100	1-b	510	4,500	13,000	65,000	5.0
100	2-a	930	5,580	18,500	180,000	6.0
100	2-b	290	1,830	1,500	13,300	6.3
100	2-c	20	360	100	1,250	>10.0
115	1	770	3,730	25,000	88,500	3.5
115	2-a	710	7,700	11,300	231,000	>10.0
115	2-b	400	1,120	8,050	16,400	2.0
115	3-a	480	2,330	3,100	14,500	4.7
115	3-b	820	2,500	3,200	48,900	3.0
115	4	3,940	15,500	117,000	1,170,000	3.9
140	1	1,120	3,870	9,200	38,000	3.5
140	2-a	1,170	8,010	7,300	240,000	6.8
140	2-b	900	4,150	7,750	43,100	4.6
140	3-a	450	830	3,250	13,800	1.8
140	3-b	970	3,010	11,300	50,700	3.1
140	4	5,210	14,500	157,000	1,160,000	2.8

(a) Shear capacity is calculated for the section subjected to demand moment and axial force
(b) Moment capacity is calculated for the section subjected to demand axial force
(c) Shear capacities and demands are rounded to the nearest 10 kips or 3 significant digits
(d) Moment capacities and demands are rounded to the nearest 50 ft-kips or 3 significant digits
(e) Section location is shown on Figures 3.8-60 through 3.8-62
(f) Stress ratio = $\frac{\text{Capacity}}{\text{Demand}}$ for shear or moment, whichever is smaller
(g) Component in Unit 2 is similar

TABLE 3.8-14

AUXILIARY BUILDING CONCRETE WALLS
STRESS RATIOS (DE)[a]

Wall Location	Elev., ft	Shear, psi Demand	Shear, psi Capacity[b]	Moment 10^3, k-ft Demand	Moment 10^3, k-ft Capacity[b]	Stress Ratio
On line H (15.7-20.3)	100	120	230	57	188	1.9
	85	160	330	9	173	1.9
On line J (11.7-15.7)[c]	100	130	270	35	126	2.0
	85	150	280	47	126	1.8
On line T(6.4-15.7)[c]	100	75	200	141	343	2.4
	85	85	220	164	664	2.6
On line T (16.8-19.2)	100[d]	55	210	15	65	3.8
On line U.5 (10.3-12.9)[c]	100[d]	45	155	20	113	3.4
On line V (15.7-20.3)	100	50	100	32	250	2.0
	85	50	90	67	245	1.8
On line V (6.4-15.7)[c]	100[d]	70	360	82	200	2.4
On line 6.4 (V-S)[c]	100	80	250	85	158	1.9
	85	100	340	20	31	1.6
On line 10.3 (T-V)[c]	100[d]	80	250	54	146	2.7
On line 12.9 (T-V)[c]	100[d]	80	240	54	145	2.7
On line 15.7 (H-T.6)[c]	100	110	250	382	782	2.1
	85	110	220	481	818	1.7
On line 15.7 (H-T.6)[c]	100	110	260	7	19	2.3
	85	60	170	12	19	1.6

(a) Stress ratio = $\frac{\text{Capacity}}{\text{Demand}}$ for shear or moment, whichever is smaller
(b) Axial demand effect is included in the capacities
(c) Counterpart in Unit 2 is similar
(d) Wall does not extend below elevation 100 ft

TABLE 3.8-15

AUXILIARY BUILDING CONCRETE WALLS
STRESS RATIOS (DDE)[a]

Wall Location	Elev., ft	Shear, psi Demand	Shear, psi Capacity[b]	Moment 10^3, k-ft Demand	Moment 10^3, k-ft Capacity[b]	Stress Ratio
On line H (15.7-20.3)	100	240	390	114	407	1.6
	85	310	550	178	556	1.8
On line J (11.7-15.7)[c]	100	250	450	62	253	1.8
	85	290	460	90	253	1.6
On line T (6.4-15.7)[c]	100	140	340	273	987	2.4
	85	170	370	316	1086	2.2
On line T (16.8-19.2)	100[d]	110	350	31	147	3.2
On line U.5(10.3-12.9)[c]	100[d]	85	260	40	244	3.0
On line V (15.7-20.3)	100	90	170	63	394	1.9
	85	100	150	133	327	1.5
On line V (6.4-15.7)[c]	100[d]	140	600	133	400	3.0
On line 6.4 (V-S)[c]	100	160	420	141	333	2.4
	85	190	570	34	92	2.7
On line 10.3 (T-V)[c]	100[d]	160	420	105	280	2.6
On line 12.9 (T-V)[c]	100[d]	160	410	106	268	2.5
On line 15.7 (H-T.6)[c]	100	220	420	553	1205	1.9
	85	220	370	740	1155	1.6
On line 15.7 (U-V)[c]	100	220	430	13	32	1.9
	85	115	290	23	30	1.3

(a) Stress ratio = $\frac{\text{Capacity}}{\text{Demand}}$ for shear or moment, whichever is smaller
(b) Axial demand effect is included in the capacities
(c) Counterpart in Unit 2 is similar
(d) Wall does not extend below elevation 100 ft

TABLE 3.8-16

AUXILIARY BUILDING CONCRETE WALLS
STRESS RATIOS (HE)[a]

Wall Location	Elev., ft	Shear, psi Demand	Shear, psi Capacity[b]	Moment 10^3, k-ft Demand	Moment 10^3, k-ft Capacity[b]	Stress Ratio
On line H (15.7-20.3)	100	400	480	176	504	1.2
	85	510	630	270	682	1.2
On line J (11.7-15.7)[c]	100	390	500	107	313	1.3
	85	440	580	145	313	1.3
On line T (6.4-15.7) [c]	100	190	310	438	860	1.6
	85	230	320	493	979	1.4
On line T (16.8-19.2)	100[d]	150	350	42	168	2.3
On line U.5 (10.3-12.9) [c]	100[d]	130	190	60	197	1.5
On line V (15.7-20.3)	100	140	200	90	489	1.4
	85	150	170	182	405	1.15
On line V (6.4-15.7) [c]	100[d]	220	640	291	500	1.7
On line 6.4 (V-S) [c]	100	380	460	301	434	1.2
	85	390	450	63	84	1.15
On line 10.3 (T-V) [c]	100[d]	330	480	220	339	1.4
On line 12.9 (T-V) [c]	100[d]	270	460	199	324	1.6
On line 15.7 (H-T.6) [c]	100	330	520	666	1494	1.6
	85	320	450	859	1425	1.4
On line 15.7 (U-V) [c]	100	340	480	19	36	1.4
	85	160	300	30.5	32.6	1.07

(a) Stress ratio = $\frac{\text{Capacity}}{\text{Demand}}$ for shear or moment, whichever is smaller
(b) Axial demand effect is included in the capacities
(c) Counterpart in Unit 2 is similar
(d) Wall does not extend below elevation 100 ft

TABLE 3.8-17

AUXILIARY BUILDING COLUMNS STRESS RATIOS[a]

Column Location[b]	Stress Ratio		
	DE	DDE	HE
14 - J.7	2.9	3.7	2.6
15 – N	5.3	7.7	4.8
15 - R.8	3.6	3.3	2.5
15 - J.7	1.04	1.8	1.3
15 – S	1.9	2.9	2.3

(a) Stress ratio = $\frac{\text{Capacity}}{\text{Demand}}$

(b) Counterpart in Unit 2 is similar

TABLE 3.8-18

AUXILIARY BUILDING SHEAR DISSIPATION TO FOUNDATION (DE NORTH-SOUTH)

Building Portion Location[a]	Shear Demand at Elevation 85 ft. — Shear from Upper Level: Direct Shear, kips	Shear from Upper Level: Torsional Shear, kips[b]	Shear from Upper Level: Torsional + Direct Shear, kips	Inertial Load at El 85 ft, kips	Total, kips	Remarks
Central Portion	23,600	1,800	25,400	4,400	29,800	Capacity is 57,900 kips
North Wing	2,300 900 200 4,700 600	500 100	2,800 1,000 200 4,700 600	600 600 400	10,900	Load is directly dissipated to the foundation
South Wing	2,300 900 200 4,700 600	500 100	2,800 1,000 200 4,700 600	600 600 400	10,900	Load is directly dissipated to the foundation

Shear Capacity at Elevation 85 ft (Central Portion)

Capacity of diaphragm to dissipate shear force to the foundation at elevation 85 ft (kips)	by bearing by rebar tension	4,700 + 18,000 = 22,700 1,100 + 1,100 = 2,200
Shear capacity of walls below elevation 85 ft (kips)		33,000
Total shear capacity at and below elevation 85 ft (kips)		57,900

(a) For building portion location only, see Figures 3.8-63 and 3.8-64
(b) Torsional shears on only one side of the center of rigidity are considered

TABLE 3.8-19

AUXILIARY BUILDING SHEAR DISSIPATION TO FOUNDATION (DE EAST-WEST)

Building Portion Location[(a)]	Shear Demand at Elevation 85 ft. — Shear from Upper Level: Direct Shear, kips	Torsional Shear, kips	Torsional + Direct Shear, kips	Inertial Load at El 85 ft, kips	Total, kips	Remarks
Central Portion	35,500	-0-	35,500	4,400	39,900	Capacity is 76,200 kips
North Wing	900 1,100 400 400	-0-	900 1,100 400 400	600 600 400	4,400	Load is directly dissipated to the foundation
South Wing	900 1,100 400 400	-0-	900 1,100 400 400	600 600 400	4,400	Load is directly dissipated to the foundation
<u>Shear Capacity at Elevation 85 ft (Central Portion)</u>						
Capacity of diaphragm to dissipate shear force to the foundation at elevation 85 ft (kips)					4,100 4,100	
Shear capacity of walls below elevation 85 ft (kips)					68,000	
Total shear capacity at and below elevation 85 ft (kips)					76,200	

(a) For building portion location only, see Figures 3.8-63 and 3.8-64

TABLE 3.8-20

AUXILIARY BUILDING SHEAR DISSIPATION TO FOUNDATION (DDE NORTH-SOUTH)

Building Portion Location[a]	Shear Demand at Elevation 85 ft. — Shear from Upper Level: Direct Shear, kips	Shear from Upper Level: Torsional Shear, kips[b]	Torsional + Direct Shear, kips	Inertial Load at El 85 ft, kips	Total, kips	Remarks
Central Portion	46,500	3,700	50,200	9,200	59,400	Capacity is 115,700 kips
North Wing	4,300 1,600 400 9,100 1,200	1,000 300 100	5,300 1,900 500 9,100 1,200	1,100 1,100 800	21,000	Load is directly dissipated to the foundation
South Wing	4,300 1,600 400 9,100 1,200	1,000 300 100	5,300 1,900 500 9,100 1,200	1,100 1,100 800	21,000	Load is directly dissipated to the foundation

Shear Capacity at Elevation 85 ft (Central Portion)

Capacity of diaphragm to dissipate shear force to the foundation at elevation 85 ft (kips)	by bearing by rebar tension	11,000 + 42,000 = 53,000 1,000 + 1,900 = 3,800
Shear capacity of walls below elevation 85 ft (kips)		58,000
Total shear capacity at and below elevation 85 ft (kips)		115,700

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(a) For building portion location only see Figures 3.8-63 and 3.8-64
(b) Torsional shears on only one side of the center of rigidity are considered

TABLE 3.8-21

AUXILIARY BUILDING SHEAR DISSIPATION TO FOUNDATION (DDE EAST-WEST)

	Shear Demand at Elevation 85 ft.					
	Shear from Upper Level					
Building Portion Location[a]	Direct Shear, kips	Torsional Shear, kips	Torsional + Direct Shear, kips	Inertial Load at El 85 ft, kips	Total, kips	Remarks
Central Portion	68,700	-0-	68,700	9,200	77,900	Capacity is 128,000 kips
North Wing	1,600 2,100 900 700	-0-	1,600 2,100 900 700	1,100 1,100 800	8,300	Load is directly dissipated to the foundation
South Wing	1,600 2,100 900 700	-0-	1,600 2,100 900 700	1,100 1,100 800	8,300	Load is directly dissipated to the foundation
<u>Shear Capacity at Elevation 85 ft (Central Portion)</u>						
Capacity of diaphragm to dissipate shear force to the foundation at elevation 85 ft (kips)					7,000 7,000	
Shear capacity of walls below elevation 85 ft (kips)					114,000	
Total shear capacity at and below elevation 85 ft (kips)					128,000	

(a) For building portion location only see Figures 3.8-63 and 3.8-64

TABLE 3.8-22

AUXILIARY BUILDING SHEAR DISSIPATION TO FOUNDATION (HE NORTH-SOUTH)[(a)]

Building Portion Location[(a)]	Shear Demand at Elevation 85 ft. — Shear from Upper Level: Direct Shear, kips	Shear from Upper Level: Torsional Shear, kips[(b)]	Shear from Upper Level: Torsional + Direct Shear, kips	Inertial Load at El 85 ft, kips	Total, kips	Remarks
Central Portion	63,000	9,000	72,000	12,500	84,500	Capacity is 136,800 kips
North Wing	5,700 2,100 500 12,200 1,700	2,300 500 300	8,000 2,600 800 12,200 1,700	1,600 1,600 1,000	29,500	Load is directly dissipated to the foundation
South Wing	5,700 2,100 500 12,200 1,700	2,300 500 300	8,000 3,600 800 12,200 1,700	1,600 1,600 1,000	29,500	Load is directly dissipated to the foundation

<u>Shear Capacity at Elevation 85 ft (Central Portion)</u>

Capacity of diaphragm to dissipate shear force to the foundation at elevation 85 ft (kips)	by bearing by rebar tension	14,600 + 48,400 = 63,000 2,940 + 2,400 = 4,800
Shear capacity of walls below elevation 85 ft (kips)		69,000
Total shear capacity at and below elevation 85 ft (kips)		136,800

(a) For illustration, see Figures 3.8-63 and 3.8-64.
(b) Torsional shears on only one side of the center of rigidity are considered.

TABLE 3.8-23

AUXILIARY BUILDING SHEAR DISSIPATION TO FOUNDATION (HE EAST-WEST)[a]

Building Portion Location[a]	Shear Demand at Elevation 85 ft.: Shear from Upper Level: Direct Shear, kips	Shear from Upper Level: Torsional Shear, kips[b]	Shear from Upper Level: Torsional + Direct Shear, kips	Inertial Load at El 85 ft, kips	Total, kips	Remarks
Central Portion	84,700	5,300	90,000	12,500	102,500	Capacity is 154,200 kips
North Wing	2,000 2,500 1,100 900	1,600 1,900 600 100	3,600 4,400 1,700 1,000	1,600 1,600 1,000	14,900	Load is directly dissipated to the foundation
South Wing	2,000 2,500 1,100 900		2,000 2,500 1,100 900	1,600 1,600 1,000	10,700	Load is directly dissipated to the foundation
Shear Capacity at Elevation 85 ft (Central Portion)						
Capacity of Diaphragm to Dissipate Shear Force to the Foundation at Elevation 85 ft (kips)					8,600 8,600	
Shear Capacity of Walls Below Elevation 85 ft (kips)					137,000	
Total Shear Capacity at and Below Elevation 85 ft (kips)					154,200	

(a) For illustration, see Figures 3.8-63 and 3.8-64.
(b) Torsional shears on only one side of the center of rigidity are considered.

TABLE 3.8-23A

AUXILIARY BUILDING AND TURBINE BUILDING
COMPARISON OF DISPLACEMENTS AND SEPARATIONS

Elevation (ft)	Maximum Total Displacement (in.)[a] DDE	HE	Separation (in.)	Minimum Factor of Safety Against Contact
163	2.7	4.5	8.0	1.78
140	0.9	0.9	8.0	8.89
115	0.7	1.1	8.0	7.27
100	0.5	0.7	3.0	4.29

(a) Displacements are calculated as sum of maximum auxiliary and turbine building displacements.

TABLE 3.8-24

DUCTILITY[a]

Structure	Blume Ductility	Newmark Ductility	
Containment	1.3[b]	1.0[b]	
Auxiliary Building	1.3[b]	1.0[b]	
		Class I	Class II
Turbine Building	[c]	1.0[b]	[c]
Intake	[c]	1.0[b]	[c, d]

(a) Ductilities are on story basis; however, floor response spectra were, in general, computed on an elastic analysis basis.

(b) Under normal conditions Newmark ductility is 1.0 maximum; higher ductility may be considered for special cases where supporting evidence justifies its use. Blume ductility for Class I structures is 1.3, and may be used only in specific situations.

(c) Concrete 1.3; steel 3, with up to 6 locally.

(d) Or as may be required to demonstrate that function of Design Class I equipment will not be adversely affected.

TABLE 3.8-25

REFUELING WATER STORAGE TANK
STRESS RATIO[(a)]

		DL + HS + DE + R_A			DL + HS + DDE + R_A			DL + HS + HE + R_A		
Force Component	Material	Demand	Capacity	Stress Ratio	Demand	Capacity	Stress Ratio	Demand	Capacity	Stress Ratio
Longitudinal force, Kips/ft	Concrete	63.5	96.0	1.51	121.2	216.0	1.78	133.3	216.0	1.62
Circumferential force, kips/ft	Concrete	54.4	96.0	1.76	71.5	216.0	3.02	79.6	216.0	2.71
In plane shear force, kips/ft	Concrete	26.1	45.5	1.74	52.1	77.4	1.49	58.9	77.4	1.31
Longitudinal moment, Kips-ft/ft	Concrete	40.4	78.2	1.94	56.3	179.7	3.19	58.6	179.7	3.07
Stress intensity outside vault opening area, Kips/in^2	Steel	6.4	16.7	2.61	11.2	22.5	2.01	15.4	40.6	2.64
Stress intensity within vault opening area, Kips/in^2	Steel	12.1	16.7	1.38	20.5	22.5	1.10	34.0	40.6	1.19

(a) Stress Ratio = $\frac{\text{Capacity}}{\text{Demand}}$

TABLE 3.8-26

TURBINE BUILDING
STRUCTURAL STEEL MEMBERS
STRESS RATIOS (HE)

Member Description	Stress Ratios[a]
Exterior columns, lines A and G	[b]
East-west roof trusses – chords	1.2[c]
East-west roof trusses – diagonals	1.0[c]
North-south walls diagonal tension bracing	1.1[c]
North-south walls diagonal compression bracing	1.0[c]
Crane runway girder	[b]
Floor beams	[b]

(a) Stress Ratio = $\frac{\text{Capacity}}{\text{Demand}}$

(b) Inelastic deformation occurs, ductility meets limits of Table 3.8-24.

(c) Effects of force redistribution are included.

TABLE 3.8-27

TURBINE BUILDING
CONCRETE MEMBERS
STRESS RATIOS (HE)[a]

Member	Location	Stress Ratios[b]
Wall, line A	Elev. 85 ft (20-30.3)	1.9
Wall, line G	Elev. 85 ft (20-30.3)	1.3
Wall, line 19	Elev. 123 ft (A-G)	1.1
Buttress, line 27	Elev. 85 ft	1.5
Floor slab	Elev. 140 ft line 21 (A-C), line C (19-21)	1.2 1.5[c]
Turbine Pedestal	Frame 6 (See Figure 3.7-15G)	1.0[d]

(a) Stress Ratio = $\frac{\text{Capacity}}{\text{Demand}}$ for shear or moment, whichever is smaller.

(b) Axial demand effect is included in the capacity.

(c) Effects of concrete cracking are considered.

(d) Effects of force distribution are included.

TABLE 3.8-27A

TURBINE BUILDING AND TURBINE PEDESTAL (UNITS 1 & 2)[a]
COMPARISON OF DISPLACEMENTS AND SEPARATIONS AT EL 140 FT
HE ANALYSIS

Side of Pedestal Location	Maximum Calculated Displacement, in.[b]			Minimum Separation, in.[c]	Factor of Safety Against Contact
	Building	Pedestal	Total		
East	0.53	1.67	2.20	2.88	1.31
West	0.58	1.67	2.25	3.00	1.33
North	0.20	0.73	0.93	1.25	1.34
South	0.21	0.76	0.97	1.31	1.35

(a) Values shown are for Unit 1 or Unit 2, whichever has the lowest factor of safety.

(b) Displacements are an envelope of maximum displacements calculated using Newmark-Hosgri design response spectra.

(c) Separations are an envelope of minimum as-built separations.

TABLE 3.8-28

INTAKE STRUCTURE
CAPACITIES AND DUCTILITIES OF FLOW STRAIGHTENERS (OR PIERS)[1]

Pier	P (Tension)	M	M_{allow}	U (Ductility)
1	129.0	10,600	15,600	N/A
2	81.5	14,300	15,900	N/A
3	44.6	15,700	16,100	N/A
4	24.1	16,400	16,200	1.24
5	39.1	17,300	16,100	1.33
6	141.0	17,700	15,500	1.44
7	146.0	16,300	15,400	1.32

Notes:

(1) These values are due to the Newmark earthquake, which governs for all piers:

P = Axial tension, kips

M = Moment, in.-kips (value of M based on linear analysis)

M_{allow} = Factored (0=0.90) ultimate moment capacity including tension effects

u = Ductility of tensile steel (ratio between calculated strain to yield strain)

TABLE 3.9-1

LOAD COMBINATIONS AND ACCEPTANCE CRITERIA FOR PRESSURIZER SAFETY AND RELIEF VALVE PIPING

Combination	Plant/System Operation Condition	Load Combination	Piping Allowable Stress Intensity
Upstream of Valves			
1	Normal	N	1.0 S_h
2	Upset	N + DE + SOT_U	1.2 S_h
3	Emergency	N + SOT_E	1.8 S_h
4	Faulted	N + MAX(DDE, HOSGRI) + SOT_F	2.4 S_h
5	Faulted	N + LOCA + MAX(DDE, HOSGRI) + SOT_F	2.4 S_h
Downstream of Valves(Note 1)			
1	Normal	N	1.0 S_h
2	Upset	N + SOT_U	1.2 S_h
3	Upset	N + DE + SOT_U	1.8 S_h
4	Emergency	N + SOT_E	1.8 S_h
5	Faulted	N + MAX(DDE, HOSGRI) + SOT_F	2.4 S_h
6	Faulted	N + LOCA + MAX(DDE, HOSGRI) + SOT_F	2.4 S_h

NOTES:

(1) This table is applicable to the seismically designed portion of downstream PG&E Design Class II and III piping necessary to isolate the response, and to assure acceptable valve loading on the discharge nozzle.

(2) Refer to SOT definitions and other load abbreviations.

(3) The bounding number of valves (and discharge sequence if setpoints are significantly different) for the applicable system operating transient defined on this sheet should be used.

(4) Use SRSS for combining dynamic load responses.

Abbreviations

N	=	Sustained loads during normal plant operation
SOT	=	System Operating Transient
SOT_U	=	Relief Valve Discharge Transient
SOT_E	=	Safety Valve Discharge Transient
SOT_F	=	Max (SOT_U; SOT_E)
DE	=	Design Earthquake
DDE	=	Double Design Earthquake
HOSGRI	=	Hosgri earthquake
LOCA	=	Loss-of-coolant accident
S_h	=	Basic material allowable stress at maximum (hot) temperature

TABLE 3.9-2

HOSGRI AND DDE SEISMIC LOADING COMBINATIONS AND STRUCTURAL CRITERIA MECHANICAL EQUIPMENT(1)

COMPONENT (2, 3)	LOADING COMBINATIONS (4)	CRITERIA (7, 8, 9, 10, 11, 12, 13, 14)	
Tanks, Heat Exchangers, Filters, Demineralizers	Deadweight + Pressure + HOSGRI/DDE + Nozzle/Piping Loads	σ_m (σ_m or σ_L) + σ_b	$\leq$ 2.0S $\leq$ 2.4S
Active Pumps	Deadweight + Pressure + HOSGRI/DDE + Nozzle/Piping Loads + Operating Loads	σ_m (σ_m or σ_L) + σ_b	$\leq$ 1.2S $\leq$ 1.8S
Inactive Pumps	Deadweight + Pressure + HOSGRI/DDE + Nozzle/Piping Loads + Operating Loads	σ_m (σ_m or σ_L) + σ_m	$\leq$ 2.0S $\leq$ 2.4S
Active Valves	Deadweight + Pressure + HOSGRI/DDE + Nozzle/Piping Loads + Operating Loads(12)	Extended Structure: Pressure Boundary: Valve Nozzles: Bolting:	$\sigma_m \leq$ 1.2S (σ_m or σ_L) + $\sigma_b \leq$ 1.8S or S_y (higher of) ANSI B16.5 or MSS-SP-66 (5) $\sigma_m \leq$ 2.0S
Inactive Valves	Deadweight + Pressure + HOSGRI/DDE + Nozzle/Piping Loads + Operating Loads(12)	Extended Structure: Pressure Boundary: Valve Nozzles: Bolting:	$\sigma_m \leq$ 2.0S (σ_m or σ_L) + $\sigma_b \leq$ 2.4S ANSI B16.5 or MSS-SP-66 (6) $\sigma_m \leq$ 2.0S

TABLE 3.9-2

COMPONENT (2, 3)	LOADING COMBINATIONS (4)	CRITERIA (7, 8, 9, 10, 11, 12, 13, 14)	
Inactive Cast Iron Pressure Retaining Components	Deadweight + Pressure + HOSGRI/DDE + Nozzle/Piping Loads + Operating Loads(12)	σ_p $(\sigma_m$ or $\sigma_L) + \sigma_b$	$0.1\ S_u$ $\leq 2.4 \times 0.1\ S_u$
Inactive Cast Iron Non-pressure Retaining Components	Deadweight + Pressure + HOSGRI/DDE + Nozzle/Piping Loads + Operating Loads(12)	$(\sigma_m$ or $\sigma_L) + \sigma_b$	$\leq 2.0 \times 0.2\ S_u$

Notes:

(1) Refer to Chapter 5 Table 5.2-8 for structural components.

(2) Active: Mechanical equipment which is needed to go from normal full power operation to safe shutdown following the earthquake and which must perform mechanical motions during the course of accomplishing its design function. For DCPP, safe shutdown is defined as Mode 5 following a Hosgri earthquake.

(3) Inactive: Mechanical Equipment which is not required to perform mechanical motions in taking the plant from normal full power operation to safe shutdown following the earthquake. For DCPP, safe shutdown is defined as Mode 5 following a Hosgri earthquake.

(4) Nozzle loads shall include piping loads transmitted to the component during the HOSGRI/DDE earthquake.

(5) Piping loads at piping/active-valve interfaces shall be limited such that maximum fiber stresses in the piping at the interface are less than the piping yield strength at temperature (S_y).

(6) Valves, being stronger than the attached piping and having a proven history without any gross failures of pressure boundaries, can safely transmit piping loads without compromising their pressure retaining integrity. Therefore piping integrity assures valve integrity.

(7) σ_m = General membrane stress. This stress is equal to the average stress across the solid section under consideration, excludes discontinuities and concentrations and is produced only by mechanical loads.

TABLE 3.9-2

Notes: (Cont'd)

(8) σ_L = Local membrane stress. This stress is equal to the same as σ_m except that it includes the effect of discontinuities.

(9) σ_b = Bending stress. This stress is equal to the linear varying portion of the stress across the solid section under consideration, excludes discontinuities and concentrations, and is produced only by mechanical loads.

(10) S = 1971 or 1974 ASME Code allowable stress. The allowable stress shall correspond to the highest metal temperature at the section under consideration during the condition consideration.

(11) S_y = 1971 or 1974 ASME Code minimum yield stress. The yield stress shall correspond to the highest metal temperature at the section under consideration during the condition consideration.

(12) Except racked-out valves.

(13) σ_p = Local membrane stress. This stress is equal to the average stress across the solid section under consideration. It excludes discontinuities and concentrations and is produced only by pressure.

(14) S_u = Material minimum tensile strength listed in either the code the component was purchased and manufactured under, or ASME Code Section III.

TABLE 3.9-3

HOSGRI AND DDE SEISMIC LOADING COMBINATIONS AND STRUCTURAL CRITERIA MECHANICAL EQUIPMENT SUPPORTS AND STRUCTURAL COMPONENT[1]

ELEMENT	LOADING COMBINATIONS (4, 5)	CRITERIA (6, 7, 8, 9, 10, 11, 12, 13)	
Linear[3]	Deadweight + HOSGRI/DDE + Nozzle/Piping Loads	1974 ASME Code Appendix XVII, Subsection NF, and Appendix F or AISC Manual, 7th Edition[11] (Stresses not to exceed S_y for active components supports)	
Plate and shell[2] (active components)	Deadweight + HOSGRI/DDE + Nozzle/Piping Loads	σ_m $(\sigma_m + \sigma_b)$	$\leq 1.2S$ $\leq 1.8S$ or S_y
Plate and shell (inactive components)	Deadweight + HOSGRI/DDE + Nozzle/Piping Loads	σ_m $(\sigma_m + \sigma_b)$	$\leq 2.0S$ $\leq 2.4S$
Bolts	Deadweight + HOSGRI/DDE + Nozzle/Piping Loads	1974 ASME Code Section III, Appendix XVII, Code Case 1644-6 and Appendix F, or AISC Manual, 7th Edition	

Notes:

(1) Includes reactor cavity manipulator crane, spent fuel pit bridge crane, flux mapping transfer devices and RCS seal table and parts. Qualification of reactor cavity manipulator crane, spent fuel pit bridge crane, and flux mapping transfer device, not required for DDE (required for HOSGRI only in order to insure structural integrity and preclude seismic interaction).

(2) Plate and shell type supports: Plate and shell type components supports are supports such as vessel skirts and saddles which are fabricated from plate and shell elements and are normally subjected to a biaxial stress field.

TABLE 3.9-3

Notes (Continued):

(3) Linear type support: A linear type component support is defined as acting under essentially as single components of direct stress. Such elements may also be subjected to shear stresses. Examples of such structural elements are: tension and compression struts, beams and columns subjected to bending, trusses, frames, rings, arches, and cables.

(4) Nozzle loads shall be those nozzle loads acting on the supported components during the HOSGRI/DDE earthquake.

(5) Plus operating loads, as applicable.

(6) σ_m = General membrane stress. This stress is equal to the average stress across the solid section under consideration, excludes discontinuities and concentrations and is produced only by mechanical loads.

(7) σ_b = Bending stress. This stress is equal to the linear varying portion of the stress across the solid section under consideration, excludes discontinuities and concentrations, and is produced only by mechanical loads.

(8) S = 1971 or 1974 ASME Code allowable stress value. The allowable stress shall correspond to the highest metal temperature at the section under consideration during the condition under consideration.

(9) S_y = 1971 or 1974 ASME Code minimum yield stress. The yield stress shall correspond to the highest metal temperature at the section under consideration during the condition under consideration.

(10) For the reactor cavity manipulator crane, the spent fuel pit bridge crane, and the flux mapping transfer device, the stress limits for the above loading combinations are obtained by increasing the normal condition allowable stresses by a factor of 1.7.

(11) The reference, "AISC Manual, 7th Edition," where used in this section, refers to the AISC Code, Part 5, "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," 1969 version.

(12) σ_p = Local membrane stress. This is equal to the average stress across the solid section under consideration. It excludes discontinuities and concentrations and is produced only by pressure.

(13) S_u = Material minimum tensile strength listed in either the code the component was purchased and manufactured under, or ASME Code Section III.

TABLE 3.9-4

DE SEISMIC LOADING COMBINATIONS AND STRUCTURAL CRITERIA MECHANICAL EQUIPMENT(1)

COMPONENT (2, 3)	LOADING COMBINATIONS (4)	CRITERIA (7, 8, 9, 10, 11, 12, 13, 14)	
Tanks, heat-exchangers filters, demineralizers	Deadweight + Pressure + DE + Nozzle/Piping Loads.	σ_m (σ_m or σ_L) + σ_b	$\leq$ 1.0S(13) $\leq$ 1.65S
Active pumps	Deadweight + Pressure + DE + Nozzle/Piping Loads + Operating Loads.	σ_m (σ_m or σ_L) + σ_b	$\leq$ 1.1S $\leq$ 1.65S
Inactive pumps	Deadweight + Pressure + DE + Nozzle/Piping Loads + Operating Loads.	σ_m (σ_m or σ_L) + σ_b	$\leq$ 1.1S $\leq$ 1.65S
Active valves	Deadweight + Pressure + DE + Nozzle/Piping Loads + Operating Loads.(11)	Extended structure:	$\sigma_m \leq$ 1.1S (σ_m or σ_L) + $\sigma_b \leq 1.0S_y$
		Pressure boundary: valve nozzles: bolting:	ANSI B16.5 or MSS-SP-66 (6) $\sigma_b \leq$ 2.0S
Inactive valves	Deadweight + Pressure + DE + Nozzle/Piping Loads + Operating Loads.(11)	Extended structure:	$\sigma_b \leq$ 1.1S (σ_m or σ_L) + $\sigma_b \leq 1.0S_y$
		Pressure boundary: valve nozzles: bolting:	ANSI B16.5 or MSS-SP-66 (6) $\sigma_m \leq$ 2.0S

TABLE 3.9-4

COMPONENT (2, 3)	LOADING COMBINATIONS (4)	CRITERIA (7, 8, 9, 10, 11, 12, 13, 14)	
Inactive cast iron, pressure retaining components	Deadweight + Pressure + DE + Nozzle/Piping Loads + Operating Loads(11)	σ_p $(\sigma_m$ or $\sigma_L) + \sigma_b$	$\leq 0.1\ S_u$ $\leq 1.5 \times 0.1\ S_u$
Inactive cast iron non-pressure retaining Components	Deadweight + Pressure + DE + Nozzle/Piping Loads + Operating Loads(11)	$(\sigma_m$ or $\sigma_L) + \sigma_b$	$\leq 1.0 \times 0.2\ S_u$

Notes:

(1) Refer to Chapter 5, Table 5.2.8 for structural components.

(2) Active: Mechanical equipment which is needed to go from normal full power operation to safe shutdown following the earthquake and which must perform mechanical motions during the course of accomplishing its design function. For DCPP, safe shutdown is defined as Mode 5 following a Hosgri earthquake.

(3) Inactive: Mechanical equipment which is not required to perform mechanical motions in taking the plant from normal full power operations to safe shutdown following the earthquake. For DCPP, safe shutdown is defined as Mode 5 following a Hosgri earthquake.

(4) Nozzle loads shall include piping loads transmitted to the component during the DE earthquake.

(5) Deleted.

(6) Valves, being stronger than the attached piping and having a proven history without any gross failures of pressure boundaries, can safely transmit piping loads without compromising their pressure retaining integrity. Therefore, piping integrity assures valve integrity.

(7) σ_m = General membrane stress. This stress is equal to the average stress across the solid section under consideration, excludes discontinuities and concentrations and is produced only by mechanical loads.

(8) σ_L = Local membrane stress. This stress is the same as σ_m except that it includes the effect of discontinuities.

TABLE 3.9-4

Notes (Continued):

(9) σ_b = Bending stress. The stress is equal to the linear varying portion of the stress across the solid section under consideration, excludes discontinuities and concentrations, and is produced only by mechanical loads.

(10) S = 1971 or 1974 ASME code allowable stress value. The allowable stress shall correspond to the highest metal temperature at the section under consideration during the condition under consideration.

(11) Except racked-out valves.

(12) The primary membrane stress limit for pressure vessels under DE loading is conservatively selected to be lower than the level permitted by the present ASME Code, in order to insure that it is also conservative with respect to earlier editions of the code of which these components were designed.

(13) σ_p = Local membrane stress. This is equal to the average stress across the solid section under consideration. It excludes discontinuities and concentrations and is produced only by pressure.

(14) S_u = Material minimum tensile strength listed in either the code the component was purchased and manufactured under, or ASME Code Section III.

(15) S_y = 1971 or 1974 ASME Code minimum yield stress. The yield stress shall correspond to the highest metal temperature at the section under consideration during the condition under consideration.

TABLE 3.9-5

DE SEISMIC LOADING COMBINATION
STRUCTURAL CRITERIA MECHANICAL EQUIPMENT SUPPORTS
AND STRUCTURAL COMPONENTS[1]

ELEMENT (2, 3)	LOADING COMBINATIONS (4,5)	CRITERIA (6,7, 8)	
Linear[3]	Deadweight + DE + Nozzle/Piping Loads	1974 ASME Code Section III Appendix XVII, Subsection NF or AISC Manual, 7th Edition.	
Plate and shell[2] (active components)	Deadweight + DE + Nozzle/Piping Loads	σ_m $(\sigma_m + \sigma_b)$	$\leq 1.0S$ $\leq 1.5S$
Plate and shell (inactive components)	Deadweight + DE + Nozzle/Piping Loads	σ_m $(\sigma_m + \sigma_b)$	$\leq 1.0S$ $\leq 1.5S$
Bolts	Deadweight + DE + Nozzle/Piping Loads	1974 ASME Code Section III Appendix XVII, Code Case 1644-6 or AISC Manual, 7th Edition.	

TABLE 3.9-5

Notes:

(1) Includes RCS seal table and parts. Qualification of reactor cavity manipulator crane and spent fuel pit bridge crane and flux mapping transfer device not required for DE. Structural integrity insured by HOSGRI qualification.

(2) Plate and shell type supports: Plate and shell type component supports are supports such as vessel skirts and saddles which are fabricated from plate and shell elements and are normally subjected to a biaxial stress field.

(3) Linear type support: A linear type component support is defined as acting under essentially a single component of direct stress. Such elements may also be subjected to shear stresses. Examples of such structural elements are: tension and compression struts, beams and columns subjected to bending, trusses, frames, rings, arches, and cables.

(4) Nozzle loads shall be those nozzle loads acting on the supported component during the DE earthquake.

(5) Plus Operating Loads, as applicable.

(6) σ_m = General membrane stress. This stress is equal to the average stress across the solid section under consideration, excludes discontinuities and concentrations and is produced only by mechanical loads.

(7) σ_b = Bending stress. This stress is equal to the linear varying portion of the stress across the solid section under consideration, excludes discontinuities and concentrations, and is produced only by mechanical loads.

(8) S = 1971 or 1974 ASME code allowable stress value. The allowable stress shall correspond to the highest metal temperature at the section under consideration during the condition under consideration.

TABLE 3.9-6

NORMAL CONDITIONS LOADING COMBINATIONS AND STRUCTURAL CRITERIA MECHANICAL EQUIPMENT(1)

COMPONENT (2, 3)	LOADING COMBINATIONS (4)	CRITERIA (7, 8, 9, 10, 11, 12, 13)	
Tanks, heat-exchangers filters, demineralizers	Deadweight + Pressure + Nozzle/ Piping Loads.	σ_m $(\sigma_m$ or $\sigma_L) + \sigma_b$	$\leq 1.0S$ $\leq 1.5S$
Active pumps	Deadweight + Pressure + Nozzle/ Piping Loads + Operating Loads.	σ_m $(\sigma_m$ or $\sigma_L) + \sigma_b$	$\leq 1.0S$ $\leq 1.5S$
Inactive pumps	Deadweight + Pressure + Nozzle/ Piping Loads + Operating Loads.	σ_m $(\sigma_m$ or $\sigma_L) + \sigma_b$	$\leq 1.0S$ $\leq 1.5S$
Active valves	Deadweight + Pressure + Nozzle/ Piping Loads + Operating Loads.	Extended Structure:	$\sigma_m \leq 1.0S$ $(\sigma_m$ or $\sigma_L) +$ $\sigma_b \leq 1.5S$
		Pressure Boundary: Valve Nozzles: Bolting:	ANSI B16.5 or MSS-SP-66 (6) $\sigma_m \leq 2.0S$
Inactive valves	Deadweight + Pressure + Nozzle/ Piping Loads + Operating Loads.	Extended Structure:	$\sigma_m \leq 1.1S$ $(\sigma_m$ or $\sigma_L)$ $+ \sigma_b \leq 1.5S$
		Pressure Boundary: Valve Nozzles: Bolting:	ANSI B16.5 or MSS-SP-66 (6) $\sigma_m \leq 2.0S$

TABLE 3.9-6

COMPONENT (2, 3)	LOADING COMBINATIONS (4)	CRITERIA (7, 8, 9, 10, 11, 12, 13)	
Inactive cast iron, pressure retaining Components	Deadweight + Pressure + Nozzle/Piping Loads + Operating Loads(12)	σ_p $(\sigma_m$ or $\sigma_L) + \sigma_b$	$\leq 0.1\ S_u$ $\leq 1.5 \times 0.1\ S_u$
Inactive cost iron non-pressure retaining Components	Deadweight + Pressure + Nozzle/Piping Loads + Operating Loads(12)	$(\sigma_m$ or $\sigma_L) + \sigma_b$	$\leq 1.0 \times 0.2\ S_u$

Notes:

(1) Refer to Chapter 5, Table 5.2.8 for structural components.

(2) Active: Mechanical equipment which is needed to go from normal full power operation to safe shutdown following the earthquake and which must perform mechanical motions during the course of accomplishing its design function. For DCPP, safe shutdown is defined as Mode 5 following a Hosgri earthquake.

(3) Inactive: Mechanical equipment which is not required to perform mechanical motions in taking the plant from normal full power operations to safe shutdown following the earthquake. For DCPP, safe shutdown is defined as Mode 5 following a Hosgri earthquake.

(4) Nozzle loads shall include piping loads transmitted to the component during the normal conditions.

(5) Deleted.

(6) Valves, being stronger than the attached piping and having a proven history without any gross failures of pressure boundaries, can safely transmit piping loads without compromising their pressure retaining integrity. Therefore, piping integrity assures valve integrity.

TABLE 3.9-6

Notes (Continued):

(7) σ_m = General membrane stress. This stress is equal to the average stress across the solid section under consideration, excludes discontinuities and concentrations and is produced only by mechanical loads.

(8) σ_L = Local membrane stress. This stress is the same as σ_m except that it includes the effect of discontinuities.

(9) σ_b = Bending stress. The stress is equal to the linear varying portion of the stress across the solid section under consideration, excludes discontinuities and concentrations, and is produced only by mechanical loads.

(10) S = 1971 or 1974 ASME code allowable stress value. The allowable stress shall correspond to the highest metal temperature at the section under consideration during the condition under consideration.

(11) S_y = 1971 or 1974 ASME code yield stress value. The yield stress shall correspond to the highest metal temperature at the section under consideration during the condition under consideration.

(12) S_p = Local membrane stress. This is equal to the average stress across the solid section under consideration. It excludes discontinuities and concentrations and is produced only by pressure.

(13) S_u = Material minimum tensile strength listed in either the code the component was purchased and manufactured under, or ASME Code Section III.

TABLE 3.9-7

NORMAL CONDITIONS LOADING COMBINATIONS AND STRUCTURAL CRITERIA MECHANICAL EQUIPMENT SUPPORTS AND STRUCTURAL COMPONENTS[1]

ELEMENT	LOADING COMBINATIONS (4, 5)	CRITERIA (6, 7, 8)	
Linear[3]	Deadweight + Nozzle/Piping Loads	1974 ASME Code Appendix XVII, Subsection NF or AISC Manual, 7th Edition	
Plate and shell[2] (active components)	Deadweight + Nozzle/Piping Loads	σ_m $(\sigma_m + \sigma_b)$	≤ 1.0S (and/or 1974 ASME Code, ≤ 1.5S Subsection NF)
Plate and shell (inactive components)	Deadweight + Nozzle/Piping Loads	σ_m $(\sigma_m + \sigma_b)$	≤ 1.0S (and/or 1974 ASME Codes, ≤ 1.5S Subsection NF)
Bolts	Deadweight + Nozzle/Piping Loads	1974 ASME Code Appendix XVII, Code Case 1644-6 or AISC Manual, 7th Edition	

TABLE 3.9-7

Notes:

(1) Includes RCS seal table and parts. Qualification of reactor cavity manipulator crane and spent fuel pit bridge crane and flux mapping transfer device not required for DE. Structural integrity insured by HOSGRI qualification.

(2) Plate and shell type supports: Plate and shell type component supports are supports such as vessel skirts and saddles which are fabricated from plate and shell elements and are normally subjected to a biaxial stress field.

(3) Linear type support: A linear type component support is defined as acting under essentially a single component of direct stress. Such elements may also be subjected to shear stresses. Examples of such structural elements are: tension and compression struts, beams and columns subjected to bending, trusses, frames, rings, arches, and cables.

(4) Nozzle loads shall be those nozzle loads acting on the supported component during the normal conditions.

(5) Plus Operating Loads, as applicable.

(6) σ_m = General membrane stress. This stress is equal to the average stress across the solid section under consideration, excludes discontinuities and concentrations and is produced only by mechanical loads.

(7) σ_b = B ending stress. This stress is equal to the linear varying portion of the stress across the solid section under consideration, excludes discontinuities and concentrations, and is produced only by mechanical loads.

(8) S = 1971 or 1974 ASME code allowable stress value. The allowable stress shall correspond to the highest metal temperature at the section under consideration during the condition under consideration.

TABLE 3.9-8

TANK DESIGN

Storage Function	Design Code	Tank Plate Material	ASME Code Allowable Design Stress, (psi)	Code
Boric acid	ASME Sec. VIII, Div. 1 (no code stamp)	ASTM A240 T304	16,000	ASME Sec. VIII, Div. 1
Liquid holdup	ASME Sec. III, Class C	ASTM A240 T304	16,000	ASME Sec. VIII, Div. 1
Component cooling water surge	ASME Sec. VIII,	ASTM A285 Gr. C	13,750	ASME Sec. VIII, Div. 1
Waste gas decay	ASME Sec. III, Class C	ASTM A285 Gr. C	13,750	ASME Sec. VIII, Div. 1
Diesel fuel oil storage (underground)	UL 58	ASTM A36	12,650	ASME Sec. VIII, Div. 1
Volume control	ASME Sec. III, Class C	ASTM A240 T304	16,000	ASME Sec. VIII, Div. 1

TABLE 3.9-8

Storage Function	Design Code	Tank Plate Material	ASME Code Allowable Design Stress, (psi)	Code
Accumulator	ASME Sec. III, Class C	ASTM A516 Gr. 70 W/ A240 T304 Cladding	17,500	ASME Sec. VIII, Div. 1
Spray additive	ASME Sec. VIII, Div. 1 (no code stamp)	ASTM A240 T304	16,000	ASME Sec. VIII, Div. 1
Transfer storage & firewater	AWWA D100	ASTM A516 Gr. 60	19,500	ASME Sec. VIII, Div. 2
Reactor coolant drain tank	ASME Sec. III, Class C	ASTM A240 T304	16,000	ASME Sec. VIII, Div. 1
Waste concentrates holding	ASME Sec. III, Class C	ASTM A240 T304	16,000	ASME Sec. VIII, Div. 1

TABLE 3.9-8

Storage Function	Design Code	Tank Plate Material	ASME Code Allowable Design Stress, (psi)	Code
Spent Resin Storage	ASME Sec. III, Class C	ASTM A240 T304	16,000	ASME Sec. VIII, Div. 1
Equipment Drain Receiver	ASME Sec. VIII, Div. 1 (no code stamp)	ASTM A240 T304	16,000	ASME Sec. VIII, Div. 1

1. ASME Section III - American Society of Mechanical Engineers, Boiler and Pressure Vessel, Section III (1968, 1971).
2. ASME Section VIII - American Society of Mechanical Engineers, Boiler and Pressure Vessel Code, Section VIII (1968, 1971) Div. 1.
3. UL-58 - Underwriters Standards, Steel Underground Tanks for Flammable and Combustible Liquids.
4. AWWAD100 - American Waterworks Association, Standard for Steel Tanks, Standpipes Reservoirs and Elevated Tanks for Water Storage.
5. Waste concentrates holding tank abandoned in place.

TABLE 3.9-9

LIST OF ACTIVE VALVES

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[(a)]	Failure Analysis Comments
AUX FW PUMP 1 TURBINE GOV	FCV-15	3.2-4	Globe	-	Speed Governor	NA	Open	11
MSIV BYPASS – LEAD 4	FCV-22	3.2-4	Globe	3	Air	Closed	Closed	
MSIV BYPASS – LEAD 3	FCV-23	3.2-4	Globe	3	Air	Closed	Closed	
MSIV BYPASS – LEAD 2	FCV-24	3.2-4	Globe	3	Air	Closed	Closed	
MSIV BYPASS – LEAD 1	FCV-25	3.2-4	Globe	3	Air	Closed	Closed	
MAIN STM LEAD 2 TO AUX FW PUMP 1 TURBINE	FCV-37	3.2-4	Gate	4	Motor	FAI	Operable	10, 25
MAIN STM LEAD 3 TO AUX FW PUMP 1 TURBINE	FCV-38	3.2-4	Gate	4	Motor	FAI	Operable	10, 25
MAIN STEAM ISOL LEAD 1	FCV-41	3.2-4	Swing Check	28	Air	Refer to Note 7	Closed	7
MAIN STEAM ISOL LEAD 2	FCV-42	3.2-4	Swing Check	28	Air	Refer to Note 7	Closed	7
MAIN STEAM ISOL LEAD 3	FCV-43	3.2-4	Swing Check	28	Air	Refer to Note 7	Closed	7

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[(a)]	Failure Analysis Comments
MAIN STEAM ISOL LEAD 4	FCV-44	3.2-4	Swing Check	28	Air	Refer to Note 7	Closed	7
MN STM TO AUX FW PUMP 1 TURBINE	FCV-95	3.2-4	Gate	4	Motor	FA	Open	11
BORIC ACID BLENDER INLET	FCV-110A	3.2-8	Globe	2	Air	Open	Open	23
STEAM GEN NO. 1 TO BLOWDN TANK	FCV-151	3.2-8	Globe	3	Air	Closed	Closed	
AUX FP TURB 1 STM INLET	FCV-152	3.2-4	Globe	4	Manual	NA	Open	11
STEAM GEN NO. 2 TO BLOWDN TANK	FCV-154	3.2-4	Globe	3	Air	Closed	Closed	
STEAM GEN NO. 3 TO BLOWDN TANK	FCV-157	3.2-4	Globe	3	Air	Closed	Closed	
STEAM GEN NO. 4 TO BLOWDN TANK	FCV-160	3.2-4	Globe	3	Air	Closed	Closed	
CTMT H_2 SAMPLE SUPPLY IN CTMT	FCV-235	3.2-23	Globe	3/8	Solenoid	Closed	Operable	
CTMT H_2 SAMPLE SUPPLY OUT CTMT	FCV-236	3.2-23	Globe	3/8	Solenoid	Closed	Operable	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
CTMT H_2 SAMPLE RETURN OUT CTMT	FCV-237	3.2-23	Globe	3/8	Solenoid	Closed	Operable	
CTMT H_2 SAMPLE SUPPLY IN CTMT	FCV-238	3.2-23	Globe	3/8	Solenoid	Closed	Operable	
CTMT H_2 SAMPLE SUPPLY OUT CTMT	FCV-239	3.2-23	Globe	3/8	Solenoid	Closed	Operable	
CTMT H_2 SAMPLE RETURN OUT CTMT	FCV-240	3.2-23	Globe	3/8	Solenoid	Closed	Operable	
STM GEN 4 BD SAMPLE OS CNTMT	FCV-244	3.2-4	Globe	¾	Air	Closed	Closed	
STM GEN 3 BD SAMPLE OS CNTMT	FCV-246	3.2-4	Globe	¾	Air	Closed	Closed	
STM GEN 2 BD SAMPLE OS CNTMT	FCV-248	3.2-4	Globe	¾	Air	Closed	Closed	
STM GEN 1 BD SAMPLE OS CNTMT	FCV-250	3.2-4	Globe	¾	Air	Closed	Closed	
RC DRN PPS DISCH IN CNTMT	FCV-253	3.2-19	Ball	2-1/2	Air	Closed	Closed	
RC DRN PPS DISCH OUT CNTMT	FCV-254	3.2-19	Ball	2-1/2	Air	Closed	Closed	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[(a)]	Failure Analysis Comments
RC DRN TANK VENT HEADER IN CONTAINMENT	FCV-255	3.2-19	Ball	3/4	Air	Closed	Closed	
RC DRN TANK VENT HEADER OUT CONTAINMENT	FCV-256	3.2-19	Ball	3/4	Air	Closed	Closed	
RC DRN TANK GAS ANALYZER OUT CONTAINMENT	FCV-257	3.2-19	Ball	1/2	Air	Closed	Closed	
RC DRN TANK GAS ANALYZER IN CONTAINMENT	FCV-258	3.2-19	Ball	1/2	Air	Closed	Closed	
RC DRN TANK N2 SUPPLY OUT CONTAINMENT	FCV-260	3.2-19	Ball	3/4	Air	Closed	Closed	
CCW SUPPLY HEADER C	FCV-355	3.2-14	B'fly	20	Motor	FAI	Closed	
CCW TO RC PUMPS	FCV-356	3.2-14	B'fly	10	Motor	FAI	Closed	
RCP THERMAL BARRIER CCW RETURN	FCV-357	3.2-14	Globe	6	Motor	FAI	Closed	
EXCESS LETDOWN HT EXCH CCW RETURN	FCV-361	3.2-14	B'fly	4	Air	Closed	Closed	
RCP OIL COOLER CCW RETURN	FCV-363	3.2-14	B'fly	6	Motor	FAI	Closed	
RHR HT EXCHANGER 2 CCW RETURN	FCV-364	3.2-14	B'fly	12	Air	21	Functional	10

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
RHR HT EXCHANGER 3 CCW RETURN	FCV-365	3.2-14	B'fly	12	Air	21	Functional	10
CCW SUPPLY HEADER A	FCV-430	3.2-14	B'fly	30	Motor	FAI	Open	
CCW SUPPLY HEADER B	FCV-431	3.2-14	B'fly	30	Motor	FAI	Open	
RAW WATER STG RES AUX FEED PUMP 1	FCV-436	3.2-3	B'fly	8	Manual	FAI	Functional	
RAW WATER STG RES AUX FEED PUMPS 2 & 3	FCV-437	3.2-3	B'fly	8	Manual	FAI	Functional	
MAIN FEEDWATER ISOLATION LEAD 1	FCV-438	3.2-3	Gate	16	Motor	FAI	Closed	5
MAIN FEEDWATER ISOLATION LEAD 2	FCV-439	3.2-3	Gate	16	Motor	FAI	Closed	5
MAIN FEEDWATER ISOLATION LEAD 3	FCV-440	3.2-3	Gate	16	Motor	FAI	Closed	5
MAIN FEEDWATER ISOLATION LEAD 4	FCV-441	3.2-3	Gate	16	Motor	FAI	Closed	5
AUX SALTWATER PUMPS CROSS	FCV-495	3.2-17	B'fly	24	Motor	FAI	Operable	
AUX. SALTWATER PUMPS CROSS	FCV-496	3.2-17	B'fly	24	Motor	FAI	Operable	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
CNT BLDG SUMP PP DISCHARGE IN CNTMT	FCV-500	3.2-19	Ball	2	Air	Closed	Closed	
CNT BLDG SUMP PP DISCHARGE OUT CNTMT	FCV-501	3.2-19	Ball	2	Air	Closed	Closed	
STEAM GEN 1 MAIN FW SUPPLY	FCV-510	3.2-3	Globe	16	Air	Closed	Closed	
STEAM GEN 2 MAIN FW SUPPLY	FCV-520	3.2-3	Globe	16	Air	Closed	Closed	
STEAM GEN 3 MAIN FW SUPPLY	FCV-530	3.2-3	Globe	16	Air	Closed	Closed	
STEAM GEN 4 MAIN FW SUPPLY	FCV-540	3.2-3	Globe	16	Air	Closed	Closed	
CNT WEST INSTRUMENT AIR	FCV-584	3.2-25	Ball	2	Air	Closed	Closed	
AUX SW TO CCW HT EXCH NO. 1	FCV-602	3.2-17	B'fly	24	Air	Open	21, 22	
AUX SW TO CCW HT EXCH NO. 2	FCV-603	3.2-17	B'fly	24	Air	Open	21, 22	
CNTMT FIRE WATER ISOLATION	FCV-633	3.2-18	Globe	3	Air	Closed	Closed 21	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[(a)]	Failure Analysis Comments
RHR PUMP 1 RECIRC	FCV-641A	3.2-10	Globe	2	Motor	FAI	Functional	
RHR PUMP 2 RECIRC	FCV-641B	3.2-10	Globe	2	Motor	FAI	Functional	
CNTMT ISO CHPS EXHAUST	FCV-658	3.2-23	Gate	4	Motor	FAI	Functional	
CNTMT ISO CHPS EXHAUST	FCV-659	3.2-23	Gate	4	Motor	FAI	Functional	
CONT PURGE SUPPLY IC	FCV-660	3.2-23	B'fly	48	Air	Closed	Closed	
CONT PURGE SUPPLY OC	FCV-661	3.2-23	B'fly	48	Air	Closed	Closed	
CONT VAC/PRESS RELIEF 1C	FCV-662	3.2-23	B'fly	12	Air	Closed	Closed	
CONT PRESSURE RELIEF OC	FCV-663	3.2-23	B'fly	12	Air	Closed	Closed	
CONT VACUUM RELIEF OC	FCV-664	3.2-23	B'fly	12	Air	Closed	Closed	
CNTMT ISO CHPS EXHAUST	FCV-668	3.2-23	Gate	4	Motor	FAI	Functional	
CNTMT ISO CHPS EXHAUST	FCV-669	3.2-23	Gate	4	Motor	FAI	Functional	
CNT AIR SAMPLE (INSIDE CNT)	FCV-678	3.2-23	Ball	1	Air	Closed	Closed	5
CNT AIR SAMPLE (OUTSIDE CNT)	FCV-679	3.2-23	Ball	1	Air	Closed	Closed	5

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
CNT AIR SAMPLE (OUTSIDE CNT)	FCV-681	3.2-23	Ball	1	Air	Closed	Closed	5
POST-LOCA SAMPLING SYST	FCV-696	3.2-19	Globe	3/8	Solenoid	Closed	Closed	5
POST-LOCA SAMPLING SYST	FCV-697	3.2-19	Globe	3/8	Solenoid	Closed	Closed	5
POST-LOCA SAMPLING SYST	FCV-698	3.2-23	Globe	3/8	Solenoid	Closed	Closed	5
POST-LOCA SAMPLING SYST	FCV-699	3.2-23	Globe	3/8	Solenoid	Closed	Closed	5
POST-LOCA SAMPLING SYST	FCV-700	3.2-23	Globe	3/8	Solenoid	Closed	Closed	5
RCP OIL COOLER CCW RETURN	FCV-749	3.2-14	B'fly	6	Motor	FAI	Closed	
RCP THERMAL BARRIER CCW RETURN	FCV-750	3.2-14	Globe	6	Motor	FAI	Closed	
STEAM GEN NO. 1 BLOWDOWN AND SAMPLE	FCV-760	3.2-4	Globe	3	Air	Closed	Closed	
STEAM GEN NO. 2 BLOWDOWN AND SAMPLE	FCV-761	3.2-4	Globe	3	Air	Closed	Closed	
STEAM GEN NO. 3 BLOWDOWN AND SAMPLE	FCV-762	3.2-4	Globe	3	Air	Closed	Closed	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
STEAM GEN NO. 4 BLOWDOWN AND SAMPLE	FCV-763	3.2-4	Globe	3	Air	Closed	Closed	
STEAM GEN NO. 1 MAIN FW SUPPLY BY-PASS	FCV-1510	3.2-3	Globe	6	Air	Closed	Closed	
STEAM GEN NO. 2 MAIN FW SUPPLY BY-PASS	FCV-1520	3.2-3	Globe	6	Air	Closed	Closed	
STEAM GEN NO. 3 MAIN FW SUPPLY BY-PASS	FCV-1530	3.2-3	Globe	6	Air	Closed	Closed	
STEAM GEN NO. 4 MAIN FW SUPPLY BY-PASS	FCV-1540	3.2-3	Globe	6	Air	Closed	Closed	
CHG PUMPS DISCH TO REGEN HT EXCH	HCV-142	3.2-4	Globe	3	Air	Closed	21	23
RHR TO COLD LEGS 3 & 4	HCV-637	3.2-10	Ball	8	Air	Open	Open	
RHR TO COLD LEGS 1 & 2	HCV-638	3.2-10	Ball	8	Air	Open	Open	
DSL FO DAY TK 1-2 HEADER B	LCV-85	3.2-21	Ball	1-1/2	Air	Closed	Functional	
DSL FO DAY TK 2-1 HEADER B	LCV-86	3.2-21	Ball	1-1/2	Air	Closed	Functional	
DSL FO DAY TK 1-3 HEADER B	LCV-87	3.2-21	Ball	1-1/2	Air	Closed	Functional	
DSL FO DAY TK 1-2 HEADER A	LCV-88	3.2-21	Ball	1-1/2	Air	Closed	Functional	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
DSL FO DAY TK 2-1 HEADER A	LCV-89	3.2-21	Ball	1-1/2	Air	Closed	Functional	
DSL FO DAY TK 1-3 HEADER A	LCV-90	3.2-21	Ball	1-1/2	Air	Closed	Functional	
AUX FEEDWATER FROM TURB AFW PP TO SG 1	LCV-106	3.2-3	Globe	2	Motor	FAI	Operable	4
AUX FEEDWATER FROM TURB AFW PP TO SG 2	LCV-107	3.2-3	Globe	2	Motor	FAI	Operable	4
AUX FEEDWATER FROM TURB AFW PP TO SG 3	LCV-108	3.2-3	Globe	2	Motor	FAI	Operable	4
AUX FEEDWATER FROM TURB AFW PP TO SG 4	LCV-109	3.2-3	Globe	2	Motor	FAI	Operable	4
AUX FEEDWATER FROM MOTOR AFW PP TO SG 1	LCV-110	3.2-3	Globe	2	Electro Hydraulic	Open	Operable	4
AUX FEEDWATER FROM MOTOR AFW PP TO SG 2	LCV-111	3.2-3	Globe	2	Electro Hydraulic	Open	Operable	4
VOLUME CONTROL TANK TO CHARG PUMPS	LCV-112B	3.2-8	Gate	4	Motor	FAI	Closed	5
VOLUME CONTROL TANK TO CHARG PUMPS	LCV-112C	3.2-8	Gate	4	Motor	FAI	Closed	5
AUX FEEDWATER FROM MOTOR AFW PP TO SG 4	LCV-113	3.2-3	Globe	2	Electro Hydraulic	Open	Operable	4

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
AUX FEEDWATER FROM MOTOR AFW PP TO SG 3	LCV-115	3.2-3	Globe	2	Electro Hydraulic	Open	Operable	4
STEAM GEN 1 10% ATM STM DUMP	PCV-19	3.2-4	Globe	8	Air	Closed	Functional 21	
STEAM GEN 2 10% ATM STM DUMP	PCV-20	3.2-4	Globe	8	Air	Closed	Functional 21	
STEAM GEN 3 10% ATM STM DUMP	PCV-21	3.2-4	Globe	8	Air	Closed	Functional 21	
STEAM GEN 4 10% ATM STM DUMP	PCV-22	3.2-4	Globe	8	Air	Closed	Functional 21	
PRESSURIZER POWER-OPERATED RELIEF	PCV-455C	3.2-7	Globe	2	Air	Closed	Functional 21	8,12
PRESSURIZER POWER-OPERATED RELIEF	PCV-456	3.2-7	Globe	2	Air	Closed	Functional 21	8,12
CONT PURGE EXHAUST 1C	RCV-11	3.2-23	B'fly	48	Air	Closed	Closed	
CONT PURGE EXHAUST OC	RCV-12	3.2-23	B'fly	48	Air	Closed	Closed	
MAIN STM SAFETY LEAD 1	RV-3	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 1	RV-4	3.2-4	Relief	6	Spring	NA	Operable	9

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[(a)]	Failure Analysis Comments
MAIN STM SAFETY LEAD 1	RV-5	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 1	RV-6	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 2	RV-7	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 2	RV-8	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 2	RV-9	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 2	RV-10	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 3	RV-11	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 3	RV-12	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 3	RV-13	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 3	RV-14	3.2-4	Relief	6	Spring	NA	Operable	9
CCW SURGE TK RV	RV-45	3.2-14	Relief	3	Spring	NA	Operable	
MAIN STM SAFETY LEAD 4	RV-58	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 4	RV-59	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 4	RV-60	3.2-4	Relief	6	Spring	NA	Operable	9

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
MAIN STM SAFETY LEAD 4	RV-61	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 1	RV-222	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 2	RV-223	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 3	RV-224	3.2-4	Relief	6	Spring	NA	Operable	9
MAIN STM SAFETY LEAD 4	RV-225	3.2-4	Relief	6	Spring	NA	Operable	9
FIRE WATER TANK CROSSTIE	FP-0-306	3.2-18	Gate	8	Manual	NA	Functional	
FIRE WATER TANK CROSSTIE BYPASS	FP-0-307	3.2-18	Gate	8	Manual	NA	Functional	
PRESSURIZER POWER RELIEF ISO	8000A	3.2-7	Gate	3	Motor	FAI	Operable	
PRESSURIZER POWER RELIEF ISO	8000B	3.2-7	Gate	3	Motor	FAI	Operable	
PRESSURIZER POWER RELIEF ISO	8000C	3.2-7	Gate	3	Motor	FAI	Operable	
PRESSURIZER SAFETY	8010A	3.2-7	Relief	6	Spring	NA	Functional	9
PRESSURIZER SAFETY	8010B	3.2-7	Relief	6	Spring	NA	Functional	9

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
PRESSURIZER SAFETY	8010C	3.2-7	Relief	6	Spring	NA	Functional	9
PRESSURIZER RELIEF TK PRIMARY WTR	8029	3.2-7	Ball	3	Air	Closed	Closed	
PRESSURIZER RELIEF TK GAS ANALYZER IC	8034A	3.2-7	Globe	3/8	Air	Closed	Closed	
PRESSURIZER RELIEF TK GAS ANALYZER OC	8034B	3.2-7	Globe	3/8	Air	Closed	Closed	
PRESSURIZER RELIEF TK N2 SUPPLY	8045	3.2-7	Dia-phragm	3/4	Air	Closed	Closed	
REACTOR VESSEL HEAD VENT SYS	8078A	3.2-7	Globe	1	Solenoid	Closed	Operable	
REACTOR VESSEL HEAD VENT SYS	8078B	3.2-7	Globe	1	Solenoid	Closed	Operable	
REACTOR VESSEL HEAD VENT SYS	8078C	3.2-7	Globe	1	Solenoid	Closed	Operable	
REACTOR VESSEL HEAD VENT SYS	8078D	3.2-7	Globe	1	Solenoid	Closed	Operable	
REACTOR COOL PPS SEAL WTR RET	8100	3.2-7	Gate	4	Motor	FAI	Closed	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
BORIC ACID TO CHARGING PPS	8104	3.2-8	Globe	2	Motor	FAI	Operable	
CENTRIFUGAL I CHG PPS RECRC	8105	3.2-8	Globe	2	Motor	FAI	Functional	5
CENTRIFUGAL CHG PPS RECRC	8106	3.2-8	Globe	2	Motor	FAI	Functional	5
CHG PPS DISCH TO LETDOWN HX	8107	3.2-8	Gate	3	Motor	FAI	Functional	
CHG PPS DISCH TO LETDOWN HX	8108	3.2-8	Gate	3	Motor	FAI	Functional	
REACT COOL PPS SEAL WTR RET	8112	3.2-8	Gate	4	Motor	FAI	Closed	
LETDOWN LINE RV	8117	3.2-8	Relief	2	Spring	Open	Closed	
RCP SEAL WTR RTN RV	8121	3.2-8	Relief	2	Spring	Open	Closed	
SEAL WTR HX INLET RV	8123	3.2-8	Relief	2	Spring	Open	Closed	
CHARGING PUMP SUCTION HEADER	8125	3.2-8	Relief	3/4	Spring	Open	Closed	
PRESSURIZER AUX SPRAY	8145	3.2-8	Globe	2	Air	Closed	Operable 21	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
CHG PUMP TO LOOP 4 COLD LEG	8146	3.2-8	Globe	3	Air	Open	Operable 21	
CHG PUMP TO LOOP 3 COLD LEG	8147	3.2-8	Globe	3	Air	Open	Operable 21	
RCS PRESSURIZER AUX SPRAY	8148	3.2-8	Globe	2	Air	Closed	Operable 21	
LETDOWN LINE ISOL	8149A	3.2-8	Globe	2	Air	Closed	Closed	
LETDOWN LINE ISOL	8149B	3.2-8	Globe	2	Air	Closed	Closed	
LETDOWN LINE ISOL	8149C	3.2-8	Globe	2	Air	Closed	Closed	
LETDOWN LINE ISOL	8152	3.2-8	Globe	2	Air	Closed	Closed	
CCP 1 FCV-128 MANUAL BYPASS	8387B	3.2-8	Globe	3	Manual	NA	Functional	23
CCP 2 FCV-128 MANUAL BYPASS	8387C	3.2-8	Globe	3	Manual	NA	Functional	23
HCV-142 MANUAL BYPASS	8403	3.2-8	Globe	3	Manual	NA	Functional	23
MANUAL EMERGENCY BORATE VALVE	8471	3.2-08	Diaphragm	2	Manual	NA	Functional	23

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[(a)]	Failure Analysis Comments
BA TRANSFER PUMP SUCTION CROSSTIE	8476	3.2-8	Diaphragm	2	Manual	NA	Functional	23
RHR PP 1 SUCT	8700A	3.2-10	Gate	14	Motor	FAI	Operable	
RHR PP 2 SUCT	8700B	3.2-10	Gate	14	Motor	FAI	Operable	
RHR SUCTION FROM LOOP 4 HOT LEG	8701	3.2-10	Gate	14	Motor	FAI	Operable	23
RHR SUCTION FROM LOOP 4 HOT LEG	8702	3.2-10	Gate	14	Motor	FAI	Operable	23
RHR DISCHARGE TO HOT LEGS 1 & 2	8703	3.2-10	Gate	12	Motor	FAI	Functional	10, 19
RHR SUCTION PIPING RV	8707	3.2-10	Relief	3	Spring	Open	Closed	
RHR COOLDOWN LINE RV	8708	3.2-10	Relief	3/4	Spring	Open	Closed	
RHR HT EXCH 1 TO RCS HOT LEGS 1 & 2	8716A	3.2-10	Gate	8	Motor	FAI	Operable	
RHR HT EXCH 2 TO RCS HOT LEGS 1 & 2	8716B	3.2-10	Gate	8	Motor	FAI	Operable	
CHARGING INJECT LINE DISCHARGE	8801A	3.2-9	Gate	4	Motor	FAI	Open	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
CHARGING INJECT LINE DISCHARGE	8801B	3.2-9	Gate	4	Motor	FAI	Open	
SAFETY INJECT PUMP 1 DISCH TO HOT LEGS 1 & 2	8802A	3.2-9	Gate	4	Motor	FAI	Operable	19
SAFETY INJECT PUMP 2 DISCH TO HOT LEGS 3 & 4	8802B	3.2-9	Gate	4	Motor	FAI	Operable	19
CHARG PUMPS TO CHARGING INJECT LINE	8803A	3.2-9	Gate	4	Motor	FAI	Open	10
CHARG PUMPS TO CHARGING INJECT LINE	8803B	3.2-9	Gate	4	Motor	FAI	Open	10
RHR HT EXCH 1 TO CHG PPS SUCT	8804A	3.2-9	Gate	8	Motor	FAI	Functional	13
RHR HT EXCH 2 TO CHG PPS SUCT	8804B	3.2-9	Gate	8	Motor	FAI	Functional	13
RWST TO CHARG PUMP SUCT	8805A	3.2-9	Gate	8	Motor	FAI	Open	10
RWST TO CHARG PUMP SUCT	8805B	3.2-9	Gate	8	Motor	FAI	Open	10
CHARGING PPS SIS PPS SUC CROSSTIE	8807A	3.2-9	Gate	4	Motor	FAI	Functional	10, 13

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[(a)]	Failure Analysis Comments
CHARGING PPS SIS PPS SUC CROSSTIE	8807B	3.2-9	Gate	4	Motor	FAI	Functional	10, 13
RHR HT EXCH 1 TO COLD LEGS 1 & 2	8809A	3.2-9	Gate	8	Motor	FAI	Functional	20
RHR HT EXCH 2 TO COLD LEGS 3 & 4	8809B	3.2-9	Gate	8	Motor	FAI	Functional	20
SIS PUMP 1 DISCH TO COLD LEGS	8821A	3.2-9	Gate	4	Motor	FAI	Functional	
SIS PUMP 2 DISCH TO COLD LEGS	8821B	3.2-9	Gate	4	Motor	FAI	Functional	
SIS PUMP DISCH TO COLD LEGS	8835	3.2-9	Gate	4	Motor	FAI	Open	14
SI PUMP DISCHARGE RV	8851	3.2-9	Relief	¾	Spring	Open	Closed	
SI PUMP DISCHARGE RV	8853A	3.2-9	Relief	¾	Spring	Open	Closed	
SI PUMP DISCHARGE RV	8853B	3.2-9	Relief	¾	Spring	Open	Closed	
ACCUM RV	8855A	3.2-9	Relief	1	Spring	Open	Closed	
ACCUM RV	8855B	3.2-9	Relief	1	Spring	Open	Closed	
ACCUM RV	8855C	3.2-9	Relief	1	Spring	Open	Closed	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[(a)]	Failure Analysis Comments
ACCUM RV	8855D	3.2-9	Relief	1	Spring	Open	Closed	
RHR HT EXCH OUTLET RELIEF RV	8856A	3.2-10	Relief	2	Spring	Open	Closed	
RHR HT EXCH OUTLET RELIEF	8856B	3.2-10	Relief	2	Spring	Open	Closed	
SI PUMP SUCT HEADER RV	8858	3.2-9	Relief	3/4	Spring	Open	Closed	
ACCUM TEST IN CTMT	8871	3.2-9	Globe	3/4	Air	Closed	Closed	
ACCUM N2 SUPPLY HEADER	8880	3.2-9	Globe	1	Air	Closed	Closed	
SAFETY INJECTION TEST LINE	8883	3.2-9	Globe	3/4	Air	Closed	Closed	5
SAFETY INJECT PUMP NO. 1 SUCT	8923A	3.2-9	Gate	6	Motor	FAI	Operable	
SAFETY INJECT PUMP NO. 2 SUCT	8923B	3.2-9	Gate	6	Motor	FAI	Operable	
ACCUM TEST OUTSIDE CONTAINMENT	8961	3.2-9	Globe	3/4	Air	Closed	Closed	
SAFETY INJECT PUMP MIN. RECIRC. VALVES	8974A	3.2-9	Globe	2	Motor	FAI	Operable	
SAFETY INJECT PUMP MIN. RECIRC. VALVES	8974B	3.2-9	Globe	2	Motor	FAI	Operable	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
RWST TO SAFETY INJECTION PUMP SUCT	8976	3.2-9	Gate	8	Motor	FAI	Open	14
RWST TO RHR PUMP SUCTION	8980	3.2-9	Gate	12	Motor	FAI	Operable	14
CNTMT SUMP TO RHR PP1 SUCT	8982A	3.2-10	Gate	14	Motor	FAI	Operable	
CNTMT SUMP TO RHR PP2 SUCT	8982B	3.2-10	Gate	14	Motor	FAI	Operable	
SPRAY ADDITIVE SYSTEM	8987	3.2-12	Relief	3/4	Spring	Open	Close	
SPRAY ADD TK OUT ISOL	8994A	3.2-12	Gate	3	Motor	FAI	Open	10
SPRAY ADD TK OUT ISOL	8994B	3.2-12	Gate	3	Motor	FAI	Open	10
CNTMT SPRAY PP 1 DISCHG	9001A	3.2-12	Gate	8	Motor	FAI	Open	10
CNTMT SPRAY PP 2 DISCHG	9001B	3.2-12	Gate	8	Motor	FAI	Open	10
RHR HT EXCH 1 TO CONT SPRAY	9003A	3.2-12	Gate	8	Motor	FAI	Operable	10
RHR HT EXCH 2 TO CONT SPRAY	9003B	3.2-12	Gate	8	Motor	FAI	Operable	10

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[(a)]	Failure Analysis Comments
RCS SAMPLE	9351A	3.2-11	Globe	3/8	N_2	Closed	Closed 21	23
RCS SAMPLE	9351B	3.2-11	Globe	3/8	N_2	Closed	Closed 21	23
PRESS STEAM SPACE IN CONTMT	9354A	3.2-7	Globe	3/8	Air	Closed	Closed	
PRESS STEAM SPACE OUT CONTMT	9354B	3.2-7	Globe	3/8	Air	Closed	Closed	
PRESS LIQUID SPACE IN CONTMT	9355A	3.2-7	Globe	3/8	Air	Closed	Closed	
PRESS LIQUID SPACE OUT CONTMT	9355B	3.2-7	Globe	3/8	Air	Closed	Closed	
HOT LEGS 1 & 4 IN CONTMT SAMPLE	9356A	3.2-7	Globe	3/8	N_2	Closed	Closed 21	
HOT LEGS 1 & 4 IN CONTMT SAMPLE	9356B	3.2-7	Globe	3/8	N_2	Closed	Closed 21	
ACCUM SAMPLE HDR IN CONTAINMENT	9357A	3.2-9	Globe	3/8	Air	Closed	Closed	
ACCUM SAMPLE HDR OUT CONTAINMENT	9357B	3.2-9	Globe	3/8	Air	Closed	Closed	

TABLE 3.9-9

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[a]	Failure Analysis Comments
CCW PUMP SUCT CROSSTIE VALVE (2 VALVES ON LINES 97 & 2285)	CCW-4 & -5	3.2-14	B'fly	20	Manual	NA	Operable	
CCW PUMP 2 DISCH ISOLATION (TO HDR B)	CCW-16	3.2-14	B'fly	20	Manual	NA	Operable	
CCW PUMP 3 DISCH ISOLATION (TO HDR B)	CCW-17	3.2-14	B'fly	20	Manual	NA	Operable	
CCW PUMP 1 DISCH ISOLATION (TO HDR A)	CCW-18	3.2-14	B'fly	20	Manual	NA	Operable	
CCW PUMP 2 DISCH ISOLATION (TO HDR A)	CCW-19	3.2-14	B'fly	20	Manual	NA	Operable	
CCW HDR C SUPPLY FROM HDR A	CCW-23	3.2-14	B'fly	24	Manual	NA	Operable	
CCW HDR C SUPPLY FROM HDR B	CCW-24	3.2-14	B'fly	24	Manual	NA	Operable	
MAIN STEAM LEAD ONE 10% STEAM DUMP ISOLATION	MS-1015	3.2-4	Gate	8	Manual	NA	Operable 24	
MAIN STEAM LEAD TWO 10% STEAM DUMP ISOLATION	MS-2015	3.2-4	Gate	8	Manual	NA	Operable 24	
MAIN STEAM LEAD THREE 10% STEAM DUMP ISOLATION	MS-3015	3.2-4	Gate	8	Manual	NA	Operable 24	

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[(a)]	Failure Analysis Comments
MAIN STEAM LEAD FOUR 10% STEAM DUMP ISOLATION	MS-4015	3.2-4	Gate	8	Manual	NA	Operable 24	

(a) The valves whose positions are listed in this column are those valves whose operability is relied on to perform an active function such as safe shutdown of the reactor or mitigation of the consequences of a Design Basis Accident coincidental with loss of offsite power. An entry of "functional" or equivalently "operable" means that the valve must be capable of being opened and/or closed to perform its active function. For DCPP, safe shutdown is defined as Mode 3 following an accident (SSER 7 and SSER 22), Mode 5 following a Hosgri earthquake, and Mode 3, followed by Mode 5 within 72 hours, following an Appendix R fire (10 CFR 50, Appendix R).

Failure Analysis Comment Notes:

1. Deleted in Revision 9.
2. Deleted in Revision 9.
3. Deleted in Revision 9.
4. Valve is provided for control. Failure, open or close, is remedied by redundant train and EOP RNO actions.
5. Valve provides isolation. Failure to close is remedied by valve in series.
6. Deleted in Revision 9.
7. Locally mounted air accumulators protected against compressed air system failure by check valves can hold open the main steam isolation valves for a short duration of time after the compressed air system is lost. In the event of loss of all air to the main steam isolation valves, the valves will fail closed.
8. These valves are provided for controlled steam release. Failure to open is remedied by redundant valves. Failure to close is remedied by closure of series valve or system shutdown.
9. These valves provide vessel protection. Failure to open is remedied by redundant valves in parallel. Valve size limits flow on failure to close.
10. Valve provides isolation. Failure to close (or stay closed) is remedied by a redundant valve in series. Failure to open (or stay open) is remedied by a redundant line (or system).
11. Valve opens to start device. Failure to open is remedied by use of redundant system.
12. Air-operated valve operation is not required for safe shutdown.
13. Used during recirculation mode.

Failure Analysis Comment Notes (continued)

14. Valve provides isolation. Failure to stay open could defeat system function. "Hot" short could close valve, but is not considered credible.
15. Deleted in Revision 9.
16. Deleted in Revision 9.
17. Deleted in Revision 9.
18. Deleted in Revision 9.
19. Valves operated (opened) during changeover from cold leg recirculation to hot leg injection. Failure to stay closed during cold leg injection or cold leg recirculation could defeat system function. "Hot" short could open valve but is not considered credible.
20. Valve 8809A operated (closed) during the changeover from cold leg injection to cold leg recirculation. Valve 8809B operated (closed) during the changeover from cold leg recirculation to hot leg recirculation. Failure of one valve to stay open during cold leg injection remedied by redundant system.
21. Air operated valves required to operate or maintain position after a loss of the compressed air system are supplied with compressed gas from the backup air/nitrogen supply system. Refer to Section 9.3.1.6 for details.
22. If one of the CCW heat exchangers is valved out-of-service, then backup air is supplied to the respective CCW heat exchanger saltwater inlet valve to maintain the valve closed. This ensures all ASW flow is directed to in-service CCW heat exchangers.
23. Valve does not have an active safety function to support accident mitigation or Mode 3 safe shutdown. Valve is active to support achieving post-Hosgri cold shutdown in the manner defined in the Hosgri Report. Valve needs to be seismically qualified for active function for Hosgri only.
24. Valve has an active safety function to support accident mitigation or Mode 3 safe shutdown. Valve is passive to support achieving post-Hosgri cold shutdown in the manner defined in the Hosgri Report.
25. Normal position for Safe Shutdown is Open. For Containment Isolation and the condition described in Section 6.5.3.9 (Rupture of a Steam Supply Line to the Turbine-Driven AFW Pump), valve must be Operable.

Abbreviations:

FCV	=	Flow control valve	RCP	=	Reactor coolant pump	B'fly	=	Butterfly
LCV	=	Level control valve	FAI	=	Fail as is	RC	=	Reactor coolant
PCV	=	Pressure control valve	PP & PPS	=	Pump(s)	CCW	=	Component cooling water
HCV	=	Hand control valve	CNT	=	Containment	RHR	=	Residual heat removal
RV	=	Relief valve	CHG	=	Charging	AFW	=	Auxiliary feedwater
TCV	=	Temperature control valve	DSL FO	=	Diesel fuel oil	NA	=	Not applicable
RCV	=	Radiation control valve	MS	=	Main steam			

TABLE 3.9-10

MAXIMUM DEFLECTIONS ALLOWED FOR REACTOR INTERNAL SUPPORT STRUCTURES FOR FAULTED CONDITIONS

Component	Allowable Deflection, Inches	No Loss of Function Deflection, Inches
Upper Barrel		
Radial inward	4.1	8.2
Radial outward	1.0	1.0
Upper Core Plate	0.100[a]	0.150
Rod Cluster Control Guide Tubes	1.0	1.75

(a) Only to ensure that the plate will not touch a guide tube.

TABLE 3.9-11

PRESSURIZED GAS CONTAINERS (Above 100 psig)

Vessel	PG&E Design Class	Design Code	Design Pressure	Vessel Operating Pressure	Vessel Volume	Type Relief Device[a]	Relief Set-point	Stored Energy ft-lb(ea)	Attached Piping: Vessel Location	Attached Piping: PG&E Design Class	Attached Piping: Largest Size	Deviations from OSHA 29 CFR Section 1910
CO_2 storage tanks (Cardox)	I	ASME B&PV Code Sec. VIII	363 psig	300 psig	7.5 ton	Relief valve Pop safety	341 psig 357 psig	N.A.[b]	Turbine building	II	6 in.	None
Diesel generator starting air receivers	I	ASME B&PV Code Sec. VIII	342 psig	250 psig	53 cu ft	Relief valve	260 psig	3×10^6	Turbine building	I	2 in.	None
Diesel generator turbocharger booster air receivers	I	ASME B&PV Code Sec. VIII	342 psig	250 psig	106 cu ft	Relief valve	260 psig	5.9×10^6	Turbine building	I	2 in.	None
Air plant receiver	II	ASME B&PV Code Sec. VIII-Div. I	120 psig	110 psig	650 cu ft	Relief valve	115 psig	12.6×10^6	Turbine building	III	4 in.	None
N_2 storage vessels	II	ASME B&PV Code Sec. VIII Case 1205	2450 psig	2200 psig	51 cu ft per vessel	Relief valve	2450 psig	34.5×10^6	Yard vault	III	3/4 in.	None
Instrument air receivers	II	ASME B&PV Code Sec. VIII	120 psig	105 psig per receiver 2 receivers	152 cu ft	Relief valve	110 psig	3×10^6	Auxiliary building & intake Structure	III	2 in.	None
H_2 storage vessels	II	ASME B&PV Code Sec VIII Case1205	2450 psig	2200 psig	51 cu. ft. per vessel 6 vessels	Relief Valve	2450 psig	34.5×10^6	Yard Vault	III	3/4 in.	None

TABLE 3.9-11

Vessel	PG&E Design Class	Design Code	Design Pressure	Vessel Operating Pressure	Vessel Volume	Type Relief Device[a]	Relief Set-point	Stored Energy ft-lb(ea)	Attached Piping: Vessel Location	Attached Piping: PG&E Design Class	Attached Piping: Largest Size	Deviations from OSHA 29 CFR Section 1910
H_2 bottles standard commercial bottles (rental)	II	ICC Std. 3A	3225 psig	2000 psig	6.7 cu. ft. bottle 4 bottles	Relief Valve	3200 psig	6,000	Turbine Building	II	3/4 in.	None
CO_2 bottles standard commer-cial bottles (rental)	II	ICC Std. 3A	3225 psig	2000 psig	75 lb $C0_2$ per vessel 11 vessels	Relief Valve	3200 psig	2.245×10^6 (Calc. M-634)	Intake Structure	III	1/2 in.	None
Compressed breath air storage vessels	II	ASME B&PV Code Sec. VIII	3873 psig	3500 psig	21 cu. ft. per vessel 9 vessels	Relief Valve	3870 psig	23.4×10^6	Unit 2 Turbine Building	III	3/4 in.	None
Carbon dioxide Storage bottles	II	ICC Std. 3AA-2265	2265 psig	2000 psig	1.5 cu. ft. per bottle 16 bottles	Relief Valve	2200 psig		Turbine Building Elev. 85'	II	2 in.	None
N_2 storage bottles	II	ICC Std. 3AA-3500	3500 psig	2800 psig	1.5 cu. ft. per bottle 4 bottles	Relief Valve	3000 psig		Aux Bldg	II	1 in.	None
Argon storage bottles	II	ICC Std. 3AA-2015	2015 psig	2000 psig	1.5 cu. ft. per bottle 5 bottles	Relief Valve	2015 psig		Penetration Area	II	1 in.	None

(a) Relief setpoint and capacity based on the applicable design code.

(b) Filled with liquid, which requires heat input to flash.

TABLE 3.9-12

MECHANICAL EQUIPMENT SEISMIC
QUALIFICATION RESULTS

Equipment	Location Building/ Elevation, ft	Qualification Method	Qualifying Spectra HE, DDE, DE	Damping Value Used
Feedwater System				
AFW Pump (Motor Driven)	Aux/100	A	DE DDE HE	R R R
AFW Pump Motor	Aux/100	A	DE DDE HE	R R R
AFW Pump (Turbine-driven)	Aux/100	A	DE DDE HE	R R R
AFW Pump Turbine	Aux/100	A	DE DDE HE	R R R
CVC System				
Boric Acid Tank and Heater	Aux/115	A	DE DDE HE	2 2 4
Safety Injection System				
SI Pump Lube Oil Filter Stand	Aux/85	A	DE DDE HE	R R R

TABLE 3.9-12

Equipment	Location Building/ Elevation, ft	Qualification Method	Qualifying Spectra HE, DDE, DE	Damping Value Used
Component Cooling System				
CCW Pump	Aux/73	A	DE	R
			DDE	R
			HE	R
CCW Pump Motor	Aux/73	A	DE	R
			DDE	R
			HE	R
CCW Heat Exchanger	Turb/85	A	DE	2
			DDE	2
			HE	4
CCW Surge Tank	Aux/163	A	DE	R
			DDE	R
			HE	R
CCW Pump Lube Oil Cooler	Aux/73	A	DE	R
			DDE	R
			HE	R
Makeup Water System				
Makeup Water Transfer Pump and Motor	Aux/100	A	DE	R
			DDE	R
			HE	R

TABLE 3.9-12

Equipment	Location Building/ Elevation, ft	Qualification Method	Qualifying Spectra HE, DDE, DE	Damping Value Used
Saltwater System				
ASW Pump and Motor	Intake/-2	A	DE	R
			DDE	R
			HE	4
Fire Protection System				
Fire Pump	Aux/115	A	DE	R
			DDE	R
			HE	R
Fire Pump Motor	Aux/115	A	DE	R
			DDE	R
			HE	R
Portable Fire Pump (diesel)	MSS/85	A, T	DE	R
			DDE	R
			HE	R
Diesel Generator System				
Diesel Generator	Turb/85	A, T	DE	2
			DDE	2
			HE	4

TABLE 3.9-12

Equipment	Location Building/ Elevation, ft	Qualification Method	Qualifying Spectra HE, DDE, DE	Damping Value Used
Diesel Transfer Pump	MSS/77	A	DE	R
			DDE	R
			HE	R
Diesel Transfer Pump Motor	MSS/77	A	DE	R
			DDE	R
			HE	R
Diesel Transfer Filter	MSS/77	A	DE	R
			DDE	R
			HE	R
Diesel Transfer Strainer	MSS/77	A	DE	R
			DDE	R
			HE	R
Priming Tank	Turb/85	A	DE	R
			DDE	R
			HE	R
Starting Air Receiver	Turb/85	A	DE	2
			DDE	2
			HE	4
Turbocharger Air Receiver	Turb/85	A	DE	R
			DDE	R
			HE	R

TABLE 3.9-12

Equipment	Location Building/ Elevation, ft	Qualification Method	Qualifying Spectra HE, DDE, DE	Damping Value Used
Ventilation System				
Containment H_2 Purge Supply Filters	Aux/100	A	DE DDE HE	R R R
Containment H_2 Purge Exhaust Filters	Aux/115	A	DE DDE HE	R R R
Containment Fan Cooler Box	Cont/140	A	DE DDE HE	R R R
Gaseous Radwaste System				
Waste Gas Compressor	Aux/60	A	DE[(a)]	R
Waste Gas Moisture Separator	Aux/60	A	DE[(a)]	R
Waste Gas Decay Tank	Aux/60	A	DE[(a)]	R

(a) Qualified for DE only per Table 3.2-2 Note (c).

(b) Legend:

A = Qualification by analysis (Qualification Method Column)
T = Qualification by testing
R = Rigid

TABLE 3.10-1

WESTINGHOUSE SUPPLIED CLASS IE INSTRUMENTATION AND ELECTRICAL EQUIPMENT SEISMIC CAPABILITIES

Equipment	Elev./Bldg.	Qualification Method	Qualifying Spectra HE, DDE, DE	FSAR Reference
Nuclear Instrumentation System Cabinet	140'/Aux.	T / A	HE, DDE, DE	3.10.2.1.1
Radiation Monitoring System Cabinets	140'/Aux.	T	HE, DDE, DE	3.10.2.1.1.1
Two-section Power Range Excore Neutron Detector	102'/Cont.	T	HE, DDE, DE	3.10.2.1.1
Solid State Protection System	140'/Aux.	T	HE, DDE, DE	3.10.2.1.2
Process Control and Protection System	128'/Aux.	T / A	HE, DDE, DE	3.10.2.1.3
Cont. Pressure Transmitter				
- Transmitter	109.67'/Cont. Exterior	T	HE, DDE, DE	3.10.2.1.5
- Sensor	109.67'/Cont.	T	HE, DDE, DE	3.10.2.1.5
Reactor Coolant Level Differential Pressure Transmitter	100'/Aux.	T	HE, DDE, DE	3.10.2.1.5
Reactor Trip Switchgear	115'/Aux.	T	HE, DDE, DE	3.10.2.1.6
Main Coolant Loop Resistance Temperature Detectors	117'/Cont.	T	HE, DDE, DE	3.10.2.1.7
Safeguards Test Cabinet	140'/Aux.	T	HE, DDE, DE	3.10.2.1.8
Aux. Safeguards Cabinet	128'/Aux.	T / A	HE, DDE, DE	3.10.2.1.9
Main Control Board	140'/Aux.	T / A	HE, DDE, DE	3.10.2.2
Electric Hydrogen Recombiner	140'/Cont.	T	HE, DDE, DE	3.10.2.31

TABLE 3.10-1

Equipment	Elev./Bldg.	Qualification Method	Qualifying Spectra HE, DDE, DE	FSAR Reference
Hydrogen Recombiner Control Panel and Power Supply	100'/Aux.	T	HE, DDE, DE	3.10.2.31
Reactor Vessel Level Instrumentation System/Incore Thermocouple Cabinets (PAMS 3 & 4)	140'/Aux.	T	HE, DDE, DE	3.10.2.32
Surface Mounted Resistance Temperature Detectors	140'/Cont.	T	HE, DDE, DE	3.10.2.32.2
High Volume Sensors	127'/Cont.	T	HE, DDE, DE	3.10.2.32.3
Hydraulic Isolators	89'/GW	T	HE, DDE, DE	3.10.2.32.4
Flux Mapping Transfer Device	127'/Cont.	A	HE	3.10.2.33
Incore Flux Mapping Cabinets	140'/Cont.	A	HE	3.10.2.33

A = Qualification by analysis (Qualification Method Column)
T = Qualification by testing
DE = Design Earthquake
DDE = Double Design Earthquake
HE = Hosgri Earthquake

TABLE 3.10-2

EQUIPMENT SEISMIC QUALIFICATION RESULTS:
ELECTRICAL, INSTRUMENTATION, AND CONTROLS

Equipment	Bldg./Elev.	Qualification Method	Qualifying Spectra HE, DDE, DE	FSAR Reference
Main annunciator	Aux/128	T / A	HE, DDE, DE	3.10.2.9
Battery chargers	Aux/115	T	HE, DDE, DE	3.10.2.8.3
Station battery	Aux/115	T / A	HE, DDE, DE	3.10.2.8.1
DC switchgear	Aux/115	T / A	HE, DDE, DE	3.10.2.8.4
Diesel generators				
a) Excitation cabinet	Turb/85	T / A	HE, DDE, DE	3.10.2.6
b) Engine control cabinet	Turb/85	T / A		
Electrical penetrations	Cont/Various	A	HE, DDE, DE	3.10.2.10
Emergency light packs	Various	T	HE, DDE, DE	--
Fire pump controller	Aux/115	T	HE, DDE, DE	3.10.2.13
Hot shutdown panel	Aux/100	T / A	HE, DDE, DE	3.10.2.3
Heat trace distribution panel	Aux/Various	A	HE, DDE, DE	N/A
Instrument power ac panelboards	Aux/115	T / A	HE, DDE, DE	3.10.2.7.7

TABLE 3.10-2

Equipment	Bldg./Elev.	Qualification Method	Qualifying Spectra HE, DDE, DE	FSAR Reference
Instrument panels PIA, PIB & PIC	Aux/128	A	HE, DDE, DE	3.10.2.5
Instrument (Panels A & B)	Aux/128	T / A		
Local instrument panels	Various	A	HE, DDE, DE	3.10.2.4
Local starters (LPF 36)	Turb/119	T	HE, DDE, DE	3.10.2.14
Local starters (LPS 96)	Turb/140	T	HE, DDE, DE	3.10.2.14
Local starters (LPG 66) E1 100/J	Aux/ Various	T / A	HE, DDE, DE	3.10.2.14
Local starter 125 Vdc (FCV 95)	Aux/100	T	HE, DDE, DE	3.10.2.8.5
Limit switches	Various	T	HE, DDE, DE	3.10.2.27
P&ΔP transmitters	Various	T	HE, DDE, DE	3.10.2.11
Safeguards relay board	Turb/119	T / A	HE, DDE, DE	3.10.2.7.3
Ventilation control				
a) Logic cabinet (POV1, POV2)	Aux/140	T / A	HE, DDE, DE	3.10.2.15
b) Relay cabinet (RCV1, RCV2)	Aux/128	T / A	HE, DDE, DE	3.10.2.15
c) Electro-mechanical devices	Aux/Various	T / A	HE, DDE, DE	3.10.2.15
Vital load center (480 Vac MCC)	Aux/100	T / A	HE, DDE, DE	3.10.2.7.4

TABLE 3.10-2

Equipment	Bldg./Elev.	Qualification Method	Qualifying Spectra HE, DDE, DE	FSAR Reference
Vital load center transformer (480 V)	Aux/100	T / A	HE, DDE, DE	3.10.2.7.5
Auxiliary relay panels (SPF, SPG, SPH)	Aux/100	T / A	HE, DDE, DE	3.10.2.7.6
Fan cooler starter	Aux/100	T / A		
Vital switchgear (4.16 kV)	Turb/119	T / A	HE, DDE, DE	3.10.2.7.1
Potential transformers	Turb/119	T / A	HE, DDE, DE	3.10.2.7.2
Air circuit breaker (pressurizer heaters)	Aux/100	A	HE, DDE, DE	--
Solenoid valves	Various	T	HE, DDE, DE	3.10.2.23
Postaccident monitor panels and instrument (PAMs 1 & 2)	Aux/140	T / A	HE, DDE, DE	3.10.2.22
Containment H2 monitors	GW/85 GE/100	T / A	HE, DDE, DE	3.10.2.25
Containment high-range radiation detector	F/145	T	HE, DDE, DE	3.10.2.28
Containment purge exhaust detectors and LRP	L/100	T / A	HE, DDE, DE	3.10.2.26
Instrument AC UPS	Aux/115	T	HE, DDE, DE	3.10.2.1.4

TABLE 3.10-2

Equipment	Bldg./Elev.	Qualification Method	Qualifying Spectra HE, DDE, DE	FSAR Reference
Control room air supply radiation monitors	H/158	T	HE, DDE, DE	3.10.2.20
Control room pressurization	Turb/145	T	HE, DDE, DE	3.10.2.20
Radiation monitors	H/140	T	HE, DDE, DE	3.10.2.20
Control room vent & press. control & power panels	Various	T / A	HE, DDE, DE	3.10.2.20
Pressurizer SRV	F/146	T	HE, DDE, DE	3.10.2.29
Position margin monitor	H/140			
Sub-cooled margin monitor (calculators)	H/140	T	HE, DDE, DE	3.10.2.21
Process solenoid valves	G/110 GW/110	T	HE, DDE, DE	3.10.2.24

A = Qualification by analysis (Qualification Method Column)
T = Qualification by testing
DE = Design Earthquake
DDE = Double Design Earthquake
HE = Hosgri Earthquake

TABLE 3.10-3

HVAC EQUIPMENT SEISMIC QUALIFICATION SPECTRA RESULTS

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Supply Fan 1 (S-1)	L/85	A	HE, DDE, DE	(b)
Supply Fan 1 (2S-1)	L/85	C	HE, DDE, DE	(b)
Supply Fan 2 (S-2)	L/85	A	HE, DDE, DE	(b)
Supply Fan 2 (2S-2)	L/85	C	HE, DDE, DE	(b)
Supply Fan 31 (S-31)	K/140	A	HE, DDE, DE	(b)
Supply Fan 32 (S-32)	K/140	A	HE, DDE, DE	(b)
Supply Fan 33 (S-33)	K/140	C	HE, DDE, DE	(b)
Supply Fan 34 (S-34)	K/140	C	HE, DDE, DE	(b)
Supply Fan 39 (S-39)	K/156	A	HE, DDE, DE	(b)
Supply Fan 40 (S-40)	K/156	A	HE, DDE, DE	(b)
Supply Fan 41 (S-41)	K/156	C	HE, DDE, DE	(b)
Supply Fan 42 (S-42)	K/156	C	HE, DDE, DE	(b)
Exhaust Fan 1 (E-1)	L/128	A	HE, DDE, DE	(b)
Exhaust Fan 1 (2E-1)	L/128	C	HE, DDE, DE	(b)
Exhaust Fan 2 (E-2)	L/128	A	HE, DDE, DE	(b)
Exhaust Fan 2 (2E-2)	L/128	C	HE, DDE, DE	(b)
Exhaust Fan 4 (E-4)	L/140	A	HE, DDE, DE	(b)
Exhaust Fan 4 (2E-4)	L/140	C	HE, DDE, DE	(b)
Exhaust Fan 5 (E-5)	L/140	A	HE, DDE, DE	(b)
Exhaust Fan 5 (2E-5)	L/140	C	HE, DDE, DE	(b)
Exhaust Fan 6 (E-6)	L/140	A	HE, DDE, DE	(b)
Exhaust Fan 6 (2E-6)	L/140	C	HE, DDE, DE	(b)
Exhaust Fan 101 (E-101)	ISA/15	A	HE, DDE, DE	(b)
Exhaust Fan 102 (E-102)	ISA/15	A	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Exhaust Fan 103 (E-103)	ISA/15	A	HE, DDE, DE	(b)
Exhaust Fan 104 (E-104)	ISA/15	A	HE, DDE, DE	(b)
Supply Fan 43 (S-43)	H/163	A	HE, DDE, DE	(b)
Supply Fan 44 (S-44)	H/163	A	HE, DDE, DE	(b)
Supply Fan 45 (S-45)	H/163	C	HE, DDE, DE	(b)
Supply Fan 46 (S-46)	H/163	C	HE, DDE, DE	(b)
Exhaust Fan 43 (E-43)	H/163	A	HE, DDE, DE	(b)
Exhaust Fan 44 (E-44)	H/163	A	HE, DDE, DE	(b)
Exhaust Fan 45 (E-45)	H/163	C	HE, DDE, DE	(b)
Exhaust Fan 46 (E-46)	H/163	C	HE, DDE, DE	(b)
Supply Fan 35 (S-35)	K/157	A	HE, DDE, DE	(b)
Supply Fan 36 (S-36)	K/157	A	HE, DDE, DE	(b)
Supply Fan 37 (S-37)	K/157	C	HE, DDE, DE	(b)
Supply Fan 38 (S-38)	K/157	C	HE, DDE, DE	(b)
Supply Fan 67 (S-67)	A/119	A	HE, DDE, DE	(b)
Supply Fan 67 (2S-67)	A/119	C	HE, DDE, DE	(b)
Supply Fan 68 (S-68)	A/119	A	HE, DDE, DE	(b)
Supply Fan 68 (2S-68)	A/119	C	HE, DDE, DE	(b)
Supply Fan 69 (S-69)	A/119	A	HE, DDE, DE	(b)
Supply Fan 69 (2S-69)	A/119	C	HE, DDE, DE	(b)
Compressor Unit 35 (CP-35)	K/156	A	HE, DDE, DE	(b)
Compressor Unit 36 (CP-36)	K/156	A	HE, DDE, DE	(b)
Compressor Unit 37 (CP-37)	K/156	C	HE, DDE, DE	(b)
Compressor Unit 38 (CP-38)	K/156	C	HE, DDE, DE	(b)
Condenser Unit 35 (CR-35)	K/157	A	HE, DDE, DE	(b)
Condenser Unit 36 (CR-36)	K/157	A	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Condenser Unit 37 (CR-37)	K/157	C	HE, DDE, DE	(b)
Condenser Unit 38 (CR-38)	K/157	C	HE, DDE, DE	(b)
Mode Damper 78 in. (1A)	L/128	A	HE, DDE, DE	(b)
Mode Damper 78 in. (2-1A)	L/128	C	HE, DDE, DE	(b)
Mode Damper 78 in. (1B)	L/128	A	HE, DDE, DE	(b)
Mode Damper 78 in. (2-1B)	L/128	C	HE, DDE, DE	(b)
Mode Damper 132x144 (3)	L/122	A	HE, DDE, DE	(b)
Mode Damper 132x144 (2-3)	L/122	C	HE, DDE, DE	(b)
Mode Damper 96x144 (5A)	L/107	A	HE, DDE, DE	(b)
Mode Damper 96x144 (2-5A)	L/107	C	HE, DDE, DE	(b)
Mode Damper 96x144 (5B)	L/107	A	HE, DDE, DE	(b)
Mode Damper 96x144 (2-5B)	L/107	C	HE, DDE, DE	(b)
Mode Damper 108x144 (6)	L/122	A	HE, DDE, DE	(b)
Mode Damper 108x144 (2-6)	L/122	C	HE, DDE, DE	(b)
Mode Damper 108x144 (9)	L/122	A	HE, DDE, DE	(b)
Mode Damper 108x144 (2-9)	L/122	C	HE, DDE, DE	(b)
Backdraft Damper for Supply Fan S-31 96x72	K/146	A	HE, DDE, DE	(b)
Backdraft Damper for Supply Fan S-32 96x72	K/146	A	HE, DDE, DE	(b)
Backdraft Damper for Supply Fan S-33 96x72	K/146	C	HE, DDE, DE	(b)
Backdraft Damper for Supply Fan S-34 96x72	K/146	C	HE, DDE, DE	(b)
Backdraft Damper for Exhaust Fan E-1 90x66	L/121	A	HE, DDE, DE	(b)
Backdraft Damper for Exhaust Fan 2E-1 90x66	L/121	C	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Backdraft Damper for Exhaust Fan E-2 90x66	L/121	A	HE, DDE, DE	(b)
Backdraft Damper for Exhaust Fan 2E-2 90x66	L/121	C	HE, DDE, DE	(b)
Backdraft Damper for Exhaust Fan E-4 56x44	L/143	A	HE, DDE, DE	(b)
Backdraft Damper for Exhaust Fan 2E-4 56x44	L/143	A	HE, DDE, DE	(b)
Backdraft Damper for Exhaust Fan E-5 56x44	L/154	A	HE, DDE, DE	(b)
Backdraft Damper for Exhaust Fan 2E-5 56x44	L/154	A	HE, DDE, DE	(b)
Backdraft Damper for Exhaust Fan E-6 56x44	L/154	A	HE, DDE, DE	(b)
Backdraft Damper for Exhaust Fan 2E-6 56x44	L/154	A	HE, DDE, DE	(b)
Forced Draft Shutter Damper 30x48	G/96	A	HE, DDE, DE	(b)
Forced Draft Shutter Damper 30x48	G/96	C	HE, DDE, DE	(b)
Fire Damper 46x14 (FD-4)	H/125	A	HE, DDE, DE	(b)
Fire Damper 46x14 (2FD-4)	H/125	C	HE, DDE, DE	(b)
Fire Damper 46x14 (FD-5)	H/125	A	HE, DDE, DE	(b)
Fire Damper 46x14 (2FD-5)	H/125	C	HE, DDE, DE	(b)
Fire Damper 46x14 (FD-6)	H/125	A	HE, DDE, DE	(b)
Fire Damper 46x14 (2FD-6)	H/125	C	HE, DDE, DE	(b)
Carbon Tray Filters (EFC-1)	L/115	A	HE, DDE, DE	(b)
Carbon Tray Filters (2EFC-1)	L/115	C	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Carbon Tray Filters (EFC-5)	L/148	A	HE, DDE, DE	(b)
Carbon Tray Filters (2EFC-5)	L/148	C	HE, DDE, DE	(b)
Carbon Tray Filters (EFC-6)	L/148	A	HE, DDE, DE	(b)
Carbon Tray Filters (2EFC-6)	L/148	C	HE, DDE, DE	(b)
Astrocel-Hepa Filters (EFH-1)	L/126	A	HE, DDE, DE	(b)
Astrocel-Hepa Filters (2EFH-1)	L/126	C	HE, DDE, DE	(b)
Astrocel-Hepa Filters (EFH-2a)	L/126	A	HE, DDE, DE	(b)
Astrocel-Hepa Filters (2EFH-2a)	L/126	C	HE, DDE, DE	(b)
Astrocel-Hepa Filters (EFH-2b)	L/105	A	HE, DDE, DE	(b)
Astrocel-Hepa Filters (2EFH-2b)	L/105	C	HE, DDE, DE	(b)
Astrocel-Hepa Filters (EFH-4)	L/150	A	HE, DDE, DE	(b)
Astrocel-Hepa Filters (2EFH-4)	L/150	C	HE, DDE, DE	(b)
Astrocel-Hepa Filters (EFH-5)	L/148	A	HE, DDE, DE	(b)
Astrocel-Hepa Filters (2EFH-5)	L/148	C	HE, DDE, DE	(b)
Astrocel-Hepa Filters (EFH-6)	L/148	A	HE, DDE, DE	(b)
Astrocel-Hepa Filters (2EFH-6)	L/148	C	HE, DDE, DE	(b)
Varicel-Roughing Filter (EFR-4)	L/154	A	HE, DDE, DE	(b)
Varicel-Roughing Filter (2EFR-4)	L/154	C	HE, DDE, DE	(b)
Varicel-Roughing Filter (EFR-5)	L/154	A	HE, DDE, DE	(b)
Varicel-Roughing Filter (2EFR-5)	L/154	C	HE, DDE, DE	(b)
Varicel-Roughing Filter (EFR-6)	L/154	A	HE, DDE, DE	(b)
Varicel-Roughing Filter (2EFR-6)	L/154	C	HE, DDE, DE	(b)
Filter Housing With Filters (FU-39)	K/155	A	HE, DDE, DE	(b)
Filter Housing With Filters (FU-41)	K/155	C	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Filter Box (FB-29)	H/163	A	HE, DDE, DE	(b)
Filter Box (2FB-29)	H/163	C	HE, DDE, DE	(b)
Electric Duct Heater (EH-30)	L/137	A	HE, DDE, DE	(b)
Chromalox Model TDH-54C (2EH-30)	J/138	C	HE, DDE, DE	(b)
Supply Fan 96 for CRPS (OS-96)	A/140	C	HE, DDE, DE	(b)
Supply Fan 97 for CRPS (OS-97)	A/140	C	HE, DDE, DE	(b)
Supply Fan 98 for CRPS (OS-98)	A/140	A	HE, DDE, DE	(b)
Supply Fan 99 for CRPS (OS-99)	A/140	A	HE, DDE, DE	(b)
Mode Damper 72 in. ϕ and Actuator (2A)	L/132	A	HE, DDE, DE	(b)
Mode Damper 72 in. ϕ and Actuator (2-2A)	L/132	C	HE, DDE, DE	(b)
Mode Damper 72 in. ϕ and Actuator (2B)	L/132	A	HE, DDE, DE	(b)
Mode Damper 72 in. ϕ and Actuator (2-2B)	L/132	C	HE, DDE, DE	(b)
Mode Damper 90x66 and Actuator (7)	L/102	A	HE, DDE, DE	(b)
Mode Damper 90x66 and Actuator (2-7)	L/102	C	HE, DDE, DE	(b)
Mode Damper 10 in. ϕ (13A)	K/97	A	HE, DDE, DE	(b)
Mode Damper 10 in. ϕ (2-13A)	K/97	C	HE, DDE, DE	(b)
Mode Damper 10 in. ϕ (13B)	K/97	A	HE, DDE, DE	(b)
Mode Damper 10 in. ϕ (2-13B)	K/97	C	HE, DDE, DE	(b)
Mode Damper 10 in. ϕ (14A)	GE/81	A	HE, DDE, DE	(b)
Mode Damper 10 in. ϕ (2-14A)	GE/81	C	HE, DDE, DE	(b)
Mode Damper 10 in. ϕ (14B)	GE/81	A	HE, DDE, DE	(b)
Mode Damper 10 in. ϕ (2-14B)	GE/81	C	HE, DDE, DE	(b)
Mode Damper 14 in. ϕ (15A)	GE/70	A	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Mode Damper 14 in. ϕ (2-15A)	GE/70	C	HE, DDE, DE	(b)
Mode Damper 14 in. ϕ (15B)	GE/70	A	HE, DDE, DE	(b)
Mode Damper 14 in. ϕ (2-15B)	GE/70	C	HE, DDE, DE	(b)
Mode Damper 48 in. ϕ (35)	L/107	A	HE, DDE, DE	(b)
Mode Damper 48 in. ϕ (2-35)	L/107	C	HE, DDE, DE	(b)
Mode Damper 54 in. ϕ and Actuator (29)	L/150	A	HE, DDE, DE	(b)
Mode Damper 54 in. ϕ and Actuator (2-29)	L/150	C	HE, DDE, DE	(b)
Mode Damper 54 in. ϕ and Actuator (30)	L/150	A	HE, DDE, DE	(b)
Mode Damper 54 in. ϕ and Actuator (2-30)	L/150	C	HE, DDE, DE	(b)
Mode Damper 54 in. ϕ and Actuator (31)	L/150	A	HE, DDE, DE	(b)
Mode Damper 54 in. ϕ and Actuator (2-31)	L/150	C	HE, DDE, DE	(b)
Mode Damper 72x100 (4A)	L/107	A	HE, DDE, DE	(b)
Mode Damper 72x100 (2-4A)	L/107	C	HE, DDE, DE	(b)
Mode Damper 72x100 (4B)	L/107	A	HE, DDE, DE	(b)
Mode Damper 72x100 (2-4B)	L/107	C	HE, DDE, DE	(b)
Mode Damper 72x75 (8A)	L/133	A	HE, DDE, DE	(b)
Mode Damper 72x75 (2-8A)	L/133	C	HE, DDE, DE	(b)
Mode Damper 72x75 (8B)	L/115	A	HE, DDE, DE	(b)
Mode Damper 72x75 (2-8B)	L/115	C	HE, DDE, DE	(b)
Mode Damper 90x66 (10)	L/110	A	HE, DDE, DE	(b)
Mode Damper 90x66 (2-10)	L/110	C	HE, DDE, DE	(b)
Mode Damper 40x84 (12)	K/90	A	HE, DDE, DE	(b)
Mode Damper 40x84 (2-12)	K/90	C	HE, DDE, DE	(b)
Mode Damper 46x40 (16A)	K/95	A	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Mode Damper 46x40 (2-16A)	K/95	C	HE, DDE, DE	(b)
Mode Damper 46x40 (16B)	K/95	A	HE, DDE, DE	(b)
Mode Damper 46x40 (2-16B)	K/95	C	HE, DDE, DE	(b)
Mode Damper 26x54 (17A)	K/94	A	HE, DDE, DE	(b)
Mode Damper 26x54 (2-17A)	K/94	C	HE, DDE, DE	(b)
Mode Damper 26x54 (17B)	K/94	A	HE, DDE, DE	(b)
Mode Damper 26x54 (2-17B)	K/94	C	HE, DDE, DE	(b)
Mode Damper 96x72 (20)	K/146	A	HE, DDE, DE	(b)
Mode Damper 96x72 (2-24)	K/146	C	HE, DDE, DE	(b)
Mode Damper 96x72 (21)	K/146	A	HE, DDE, DE	(b)
Mode Damper 96x72 (2-21)	K/146	C	HE, DDE, DE	(b)
Mode Damper 54x100 (22A)	K/141	A	HE, DDE, DE	(b)
Mode Damper 54x100 (2-22A)	K/141	C	HE, DDE, DE	(b)
Mode Damper 54x100 (22B)	K/132	A	HE, DDE, DE	(b)
Mode Damper 54x100 (2-22B)	K/132	C	HE, DDE, DE	(b)
Mode Damper 40x40 (23)	K/135	A	HE, DDE, DE	(b)
Mode Damper 40x40 (2-23)	K/135	C	HE, DDE, DE	(b)
Mode Damper 40x40 (23B)	K/135	A	HE, DDE, DE	(b)
Mode Damper 40x40 (2-23B)	K/135	C	HE, DDE, DE	(b)
Mode Damper 14x44 (24)	K/111	A	HE, DDE, DE	(b)
Mode Damper 14x44 (2-24)	K/111	C	HE, DDE, DE	(b)
Mode Damper 14x44 (24B)	K/111	A	HE, DDE, DE	(b)
Mode Damper 14x44 (2-24B)	K/111	C	HE, DDE, DE	(b)
Mode Damper 48x40 (26A)	K/87	A	HE, DDE, DE	(b)
Mode Damper 48x40 (2-26A)	K/87	C	HE, DDE, DE	(b)
Mode Damper 48x40 (26B)	K/87	A	HE, DDE, DE	(b)
Mode Damper 48x40 (2-26B)	K/87	C	HE, DDE, DE	(b)
Mode Damper 30x48 (25)	K/95	A	HE, DDE, DE	(b)
Mode Damper 30x48 (2-25)	K/95	C	HE, DDE, DE	(b)
Mode Damper 30x48 (25B)	K/95	A	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Mode Damper 30x48 (2-25B)	K/95	C	HE, DDE, DE	(b)
Mode Damper 42x64 (33)	L/108	A	HE, DDE, DE	(b)
Mode Damper 42x64 (2-33)	L/108	C	HE, DDE, DE	(b)
Mode Damper 42x64 (34)	L/105	A	HE, DDE, DE	(b)
Mode Damper 42x64 (2-34)	L/105	C	HE, DDE, DE	(b)
Backdraft Damper for Supply Fan S-1 64x42	L/95	A	HE, DDE, DE	(b)
Backdraft Damper for Supply Fan 2S-1 64x42	L/95	C	HE, DDE, DE	(b)
Backdraft Damper for Supply Fan S-2 64x42	L/95	A	HE, DDE, DE	(b)
Backdraft Damper for Supply Fan 2S-2 64x42	L/95	C	HE, DDE, DE	(b)
Backdraft Damper 14 in. ϕ (OBD-1)	A/149	A	HE, DDE, DE	(b)
Backdraft Damper 14 in. ϕ (OBD-2)	A/149	A	HE, DDE, DE	(b)
Backdraft Damper 14 in. ϕ (OBD-3)	A/149	C	HE, DDE, DE	(b)
Backdraft Damper 14 in. ϕ (OBD-4) A/149	A/149	C	HE, DDE, DE	(b)
Quadrant Damper (QD 10 in. ϕ)	K/110	A	HE, DDE, DE	(b)
Quadrant Damper (QD 20 in. ϕ)	K/82	A	HE, DDE, DE	(b)
Quadrant Damper (QD 14 in. ϕ)	K/82	A	HE, DDE, DE	(b)
Quadrant Damper (QD 14 in. ϕ)	K/111	A	HE, DDE, DE	(b)
Quadrant Damper (QD 16 in. ϕ)	K/111	A	HE, DDE, DE	(b)
Quadrant Damper (QD 12 in. ϕ)	K/112	A	HE, DDE, DE	(b)
Quadrant Damper (QD 12 in. ϕ)	K/130	A	HE, DDE, DE	(b) (e)
Quadrant Damper (QD 10 in. ϕ)	K/132	A	HE, DDE, DE	(b) (e)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Quadrant Damper (QD 14 in. ϕ)	K/132	A	HE, DDE, DE	(b) (e)
Quadrant Damper (QD 12 in. ϕ)	K/134	A	HE, DDE, DE	(b) (e)
Quadrant Damper (QD 16 in. ϕ)	K/134	A	HE, DDE, DE	(b) (e)
Quadrant Damper (QD 16 in. ϕ)	K/131	A	HE, DDE, DE	(b) (e)
Quadrant Damper (QD 20 in. ϕ)	K/134	A	HE, DDE, DE	(b) (e)
Quadrant Damper (QD 14 in. ϕ)	K/134	A	HE, DDE, DE	(b) (e)
Quadrant Damper (QD 14 in. ϕ)	K/70	A	HE, DDE, DE	(b)
Quadrant Damper (QD 14 in. ϕ)	GE/70	A	HE, DDE, DE	(b)
Quadrant Damper (QD 14 in. ϕ)	H/79	A	HE, DDE, DE	(b)
Quadrant Damper (QD 20 in. ϕ)	H/65	A	HE, DDE, DE	(b)
Quadrant Damper (QD 28 in. ϕ)	J/132	A	HE, DDE, DE	(b)
Quadrant Damper (QD 16 in. ϕ)	L/141	A	HE, DDE, DE	(b)
Quadrant Damper (QD 14 in. ϕ)	GE/70	A	HE, DDE, DE	(b)
Quadrant Damper (QD 18 in. ϕ)	K/132	C	HE, DDE, DE	(b)
Quadrant Damper #52 (QD 48x24)	J/111	A	HE, DDE, DE	(b)
Quadrant Damper #2-52 (QD 48x24)	J/111	A	HE, DDE, DE	(b)
Quadrant Damper #54 (QD 42x42)	J/134	A	HE, DDE, DE	(b)
Quadrant Damper #2-54 (QD 42x42)	J/134	A	HE, DDE, DE	(b)
Quadrant Damper #46 (QD 38x14)	J/123	A	HE, DDE, DE	(b)
Quadrant Damper #2-46 (QD 38x14)	J/123	A	HE, DDE, DE	(b)
Quadrant Damper #33 (QD 40x40)	K/135	A	HE, DDE, DE	(b)
Quadrant Damper #2-33 (QD 40x40)	K/135	A	HE, DDE, DE	(b)
Quadrant Damper #34 (QD 14x10)	K/136	A	HE, DDE, DE	(b)
Quadrant Damper #2-34 (QD 14x10)	K/136	A	HE, DDE, DE	(b)
Quadrant Damper #35 (QD 46x14)	K/134	A	HE, DDE, DE	(b)
Quadrant Damper #2-35 (QD 46x14)	K/134	A	HE, DDE, DE	(b)
Quadrant Damper #24 (QD 14x14)	K/109	A	HE, DDE, DE	(b)
Quadrant Damper #2-24 (QD 14x14)	K/109	A	HE, DDE, DE	(b)
Quadrant Damper #31 (QD 54x100)	K/132	A	HE, DDE, DE	(b)
Quadrant Damper #2-31 (QD 54x100)	K/132	C	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Quadrant Damper #25 (QD 14x30)	K/111	A	HE, DDE, DE	(b)
Quadrant Damper #2-25 (QD 14x30)	K/111	A	HE, DDE, DE	(b)
Quadrant Damper #66 (QD 24x44)	K/92	A	HE, DDE, DE	(b)
Quadrant Damper #2-67 (QD 24x44)	K/92	A	HE, DDE, DE	(b)
Quadrant Damper #5 (QD 58x32)	K/81	A	HE, DDE, DE	(b)
Quadrant Damper #2-5 (QD 58x32)	K/81	C	HE, DDE, DE	(b)
Quadrant Damper #64 (QD 44x32)	K/81	A	HE, DDE, DE	(b)
Quadrant Damper #2-64 (QD 44x32)	K/81	A	HE, DDE, DE	(b)
Quadrant Damper #6 (QD 24x32)	K/81	A	HE, DDE, DE	(b)
Quadrant Damper #2-6 (QD 24x32)	K/81	A	HE, DDE, DE	(b)
Quadrant Damper #4 (QD 44x26)	K/82	A	HE, DDE, DE	(b)
Quadrant Damper #2-4 (QD 44x26)	K/82	A	HE, DDE, DE	(b)
Quadrant Damper #53 (QD 72x100)	L/100	A	HE, DDE, DE	(b)
Quadrant Damper #2-53 (QD 72x100)	L/100	C	HE, DDE, DE	(b)
Volume Damper (VD 24x24)	H/123	A	HE, DDE, DE	(b)
Volume Damper (VD 24x24)	H/124	A	HE, DDE, DE	(b)
Volume Damper (VD 27x21)	H/124	A	HE, DDE, DE	(b)
Quadrant Damper #19 (QD 38x44)	K/96	A	HE, DDE, DE	(b)
Quadrant Damper #2-19 (QD 38x44)	K/96	A	HE, DDE, DE	(b)
Quadrant Damper #15 (QD 20x12)	K/97	A	HE, DDE, DE	(b)
Quadrant Damper #2-15 (QD 20x12)	K/97	A	HE, DDE, DE	(b)
Quadrant Damper #20 (QD 20x44)	K/97	A	HE, DDE, DE	(b)
Quadrant Damper #2-20 (QD 20x44)	K/97	A	HE, DDE, DE	(b)
Quadrant Damper #14 (QD 72x24)	K/96	A	HE, DDE, DE	(b)
Quadrant Damper #2-14 (QD 72x24)	K/96	A	HE, DDE, DE	(b)
Quadrant Damper #28 (QD 18x16)	K/113	A	HE, DDE, DE	(b)
Quadrant Damper #2-28 (QD 18x16)	K/113	A	HE, DDE, DE	(b)
Quadrant Damper #29 (QD 16x18)	K/112	A	HE, DDE, DE	(b)
Quadrant Damper #2-29 (QD 16x18)	K/112	A	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location(c) Building/ Elevation, ft	Qualification Method(c)	Qualifying Spectra(c)	Notes
Quadrant Damper #37 (QD 32x30)	K/132	A	HE, DDE, DE	(b)
Quadrant Damper #2-37 (QD 32x30)	K/132	A	HE, DDE, DE	(b)
Quadrant Damper #45 (QD 38x38)	J/120	A	HE, DDE, DE	(b)
Quadrant Damper #2-45 (QD 38x38)	J/120	A	HE, DDE, DE	(b)
Quadrant Damper (QD 24x24)	H/130	C	HE, DDE, DE	(b) (f)
Quadrant Damper (QD 18x24)	H/130	C	HE, DDE, DE	(b) (f)
Quadrant Damper #36 (QD 22x30)	K/131	A	HE, DDE, DE	(b)
Quadrant Damper #2-36 (QD 22x30)	K/131	A	HE, DDE, DE	(b)
Quadrant Damper #10 (QD 46x30)	K/78	A	HE, DDE, DE	(b)
Quadrant Damper #2-10 (QD 46x30)	K/78	A	HE, DDE, DE	(b)
Quadrant Damper #11 (QD 18x30)	K/79	A	HE, DDE, DE	(b)
Quadrant Damper #2-11 (QD 18x30)	K/79	A	HE, DDE, DE	(b)
Quadrant Damper #12 (QD 6x30)	K/79	A	HE, DDE, DE	(b)
Quadrant Damper #2-12 (QD 6x30)	K/79	A	HE, DDE, DE	(b)
Quadrant Damper #3 (QD 20x32)	K/69	A	HE, DDE, DE	(b)
Quadrant Damper #2-3 (QD 20x32)	K/69	A	HE, DDE, DE	(b)
Quadrant Damper #55 (QD 30x12)	H/61	A	HE, DDE, DE	(b)
Quadrant Damper #2-55 (QD 30x12)	H/61	A	HE, DDE, DE	(b)
Quadrant Damper #40 (QD 16x40)	J/152	A	HE, DDE, DE	(b)
Quadrant Damper #2-40 (QD 16x40)	J/152	A	HE, DDE, DE	(b)
Quadrant Damper #27 (QD 40x42)	K/110	A	HE, DDE, DE	(b)
Quadrant Damper #2-27 (QD 40x42)	K/110	A	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Quadrant Damper #26 (QD 36x42)	K/110	A	HE, DDE, DE	(b)
Quadrant Damper #2-26 (QD 36x42)	K/110	A	HE, DDE, DE	(b)
Quadrant Damper #18 (QD 40x26)	K/97			
Quadrant Damper #2-18 (QD 40x26)	K/97			
Quadrant Damper #2 (QD 30x66)	H/69	A	HE, DDE, DE	(b)
Quadrant Damper #2-2 (QD 30x66)	H/69	A	HE, DDE, DE	(b)
Quadrant Damper #1 (QD 24x72)	K/69	A	HE, DDE, DE	(b)
Quadrant Damper #2-1 (QD 24x72)	K/69	A	HE, DDE, DE	(b)
Quadrant Damper #43 (QD 86x48)	K/132	A	HE, DDE, DE	(b)
Quadrant Damper #2-43 (QD 86x48)	K/132	C	HE, DDE, DE	(b)
Quadrant Damper #42 (QD 36x18)	K/132	A	HE, DDE, DE	(b)
Quadrant Damper #2-42 (QD 36x18)	K/132	A	HE, DDE, DE	(b)
Quadrant Damper #8 (QD 36x18)	K/81	A	HE, DDE, DE	(b)
Quadrant Damper #2-8 (QD 36x18)	K/81	A	HE, DDE, DE	(b)
Quadrant Damper #9 (QD 36x18)	K/81	A	HE, DDE, DE	(b)
Quadrant Damper #2-9 (QD 36x18)	K/81	A	HE, DDE, DE	(b)
Quadrant Damper #41 (QD 56x50)	L/143	A	HE, DDE, DE	(b)
Quadrant Damper #2-41 (QD 56x50)	L/143	C	HE, DDE, DE	(b)
Quadrant Damper #65 (QD 60x30)	J/152	A	HE, DDE, DE	(b)
Quadrant Damper #2-65 (QD 60x30)	J/152	C	HE, DDE, DE	(b)
Quadrant Damper #49 (QD 38x30)	J/156	A	HE, DDE, DE	(b)
Quadrant Damper #2-49 (QD 38x30)	J/156	C	HE, DDE, DE	(b)
Quadrant Damper #7 (QD 84x72)	K/85	A	HE, DDE, DE	(b)
Quadrant Damper #2-7 (QD 84x72)	K/85	C	HE, DDE, DE	(b)
Quadrant Damper #13 (QD 54x48)	K/86	A	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Quadrant Damper #2-13 (QD 54x48)	K/86	C	HE, DDE, DE	(b)
Quadrant Damper #32 (QD 37x78)	K/131	A	HE, DDE, DE	(b)
Quadrant Damper #2-32 (QD 37x78)	K/131	A	HE, DDE, DE	(b)
Motorized Damper 24 in. ϕ (2)	H/156	A	HE, DDE, DE	(b)
Motorized Damper 24 in. ϕ (2-2)	H/156	A	HE, DDE, DE	(b)
Motorized Damper 24 in. ϕ (2A)	H/160	A	HE, DDE, DE	(b)
Motorized Damper 24 in. ϕ (2-2A)	H/160	A	HE, DDE, DE	(b)
Motorized Damper 18 in. ϕ (3)	H/156	A	HE, DDE, DE	(b)
Motorized Damper 18 in. ϕ (2-3)	H/156	A	HE, DDE, DE	(b)
Motorized Damper 18 in. ϕ (3A)	H/160	A	HE, DDE, DE	(b)
Motorized Damper 18 in. ϕ (2-3A)	H/160	A	HE, DDE, DE	(b)
Motorized Damper 24 in. ϕ (7)	H/161	A	HE, DDE, DE	(b)
Motorized Damper 24 in. ϕ (2-7)	H/161	A	HE, DDE, DE	(b)
Motorized Damper 24 in. ϕ (7A)	H/159	A	HE, DDE, DE	(b)
Motorized Damper 24 in. ϕ (2-7A)	H/159	A	HE, DDE, DE	(b)
Motorized Damper 18 in. ϕ (8)	H/163	A	HE, DDE, DE	(b)
Motorized Damper 18 in. ϕ (2-8)	H/163	A	HE, DDE, DE	(b)
Motorized Damper 18 in. ϕ (8A)	H/163	A	HE, DDE, DE	(b)
Motorized Damper 18 in. ϕ (2-8A)	H/163	A	HE, DDE, DE	(b)
Motorized Damper 14 in. ϕ for CRPS (1)	A/143	A	HE, DDE, DE	(b)
Motorized Damper 14 in. ϕ for CRPS (2-1)	A/143	C	HE, DDE, DE	(b)
Motorized Damper 14 in. ϕ for CRPS (1A)	A/149	A	HE, DDE, DE	(b)
Motorized Damper 14 in. ϕ for CRPS (2-1A)	A/149	C	HE, DDE, DE	(b)
Motorized Damper 14 in. ϕ for CRPS (1B)	A/143	A	HE, DDE, DE	(b)
Motorized Damper 14 in. ϕ for CRPS (2-1B)	A/143	C	HE, DDE, DE	(b)
Motorized Damper 14 in. ϕ for CRPS (1C)	A/149	A	HE, DDE, DE	(b)
Motorized Damper 14 in. ϕ for CRPS (2-1C)	A/149	C	HE, DDE, DE	(b)
Limitorque Actuator for	H/156	T	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location(c) Building/ Elevation, ft	Qualification Method(c)	Qualifying Spectra(c)	Notes
Motorized Damper (2)				
Limitorque Actuator for Motorized Damper (2-2)	H/156	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (2A)	H/160	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (2-2A)	H/160	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (3)	H/156	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (2-3)	H/156	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (3A)	H/160	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (2-3A)	H/161	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (7)	H/160	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (2-7)	H/161	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (7A)	H/159	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (2-7A)	H/159	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (8)	H/163	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (2-8)	H/163	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (8A)	H/163	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (2-8A)	H/163	T	HE, DDE, DE	(b)
Motorized Damper 18x16 (4)	H/163	A	HE, DDE, DE	(b)
Motorized Damper 18x16 (2-4)	H/163	C	HE, DDE, DE	(b)
Motorized Damper 36x24 (5)	H/163	A	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Motorized Damper 36x24 (2-5)	H/163	C	HE, DDE, DE	(b)
Motorized Damper 36x24 (6)	H/159	A	HE, DDE, DE	(b)
Motorized Damper 36x24 (2-6)	H/159	C	HE, DDE, DE	(b)
Motorized Damper 70x16 (9)	H/159	A	HE, DDE, DE	(b)
Motorized Damper 70x16 (2-9)	H/159	C	HE, DDE, DE	(b)
Motorized Damper 70x16 (9A)	H/159	A	HE, DDE, DE	(b)
Motorized Damper 70x16 (2-9A)	H/159	C	HE, DDE, DE	(b)
Motorized Damper 70x20 (10)	H/157	A	HE, DDE, DE	(b)
Motorized Damper 70x20 (2-10)	H/157	C	HE, DDE, DE	(b)
Motorized Damper 70x20 (10A)	H/157	A	HE, DDE, DE	(b)
Motorized Damper 70x20 (2-10A)	H/157	C	HE, DDE, DE	(b)
Motorized Damper 70x16 (11)	H/159	A	HE, DDE, DE	(b)
Motorized Damper 70x16 (2-11)	H/159	C	HE, DDE, DE	(b)
Motorized Damper 70x16 (11A)	H/159	A	HE, DDE, DE	(b)
Motorized Damper 70x16 (2-11A)	H/159	C	HE, DDE, DE	(b)
Motorized Damper 70x20 (12)	H/157	A	HE, DDE, DE	(b)
Motorized Damper 70x20 (2-12)	H/157	C	HE, DDE, DE	(b)
Motorized Damper 70x20 (12A)	H/157	A	HE, DDE, DE	(b)
Motorized Damper 70x20 (2-12A)	H/157	C	HE, DDE, DE	(b)
Motorized Damper 22x12 (13)	H/159	A	HE, DDE, DE	(b)
Motorized Damper 22x12 (2-13)	H/159	C	HE, DDE, DE	(b)
Motorized Damper 22x12 (14)	H/159	A	HE, DDE, DE	(b)
Motorized Damper 22x12 (2-14)	H/159	C	HE, DDE, DE	(b)
Balancing Damper 14 in. ϕ (1-15)	H/163	A	HE, DDE, DE	(b)
Balancing Damper 14 in. ϕ(2-15)	H/163	C	HE, DDE, DE	(b)
Balancing Damper 14 in. ϕ (1-16)	A/141	A	HE, DDE, DE	(b)
Balancing Damper 14 in. ϕ (2-16)	A/141	C	HE, DDE, DE	(b)
Shut-off Damper (48X48) (HD-43)	H/168	A	HE, DDE, DE	(b)
Shut-off Damper (48X48) (HD-44)	H/168	A	HE, DDE, DE	(b)
Shut-off Damper (48X48) (HD-45)	H/168	C	HE, DDE, DE	(b)
Shut-off Damper (48X48) (HD-46)	H/168	C	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Back Draft Damper (E-21)	GW/96	A	HE, DDE, DE	(b)
Back Draft Damper (48X48) (BDD-43)	H/168	A	HE, DDE, DE	(b)
Back Draft Damper (48X48) (BDD-44)	H/168	A	HE, DDE, DE	(b)
Back Draft Damper (48X48) (BDD-45)	H/168	C	HE, DDE, DE	(b)
Back Draft Damper (48X48) (BDD-46)	H/168	C	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper 4	K/166	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper 2-4	K/166	C	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper 5	K/166	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper 2-5	K/166	C	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper 6	K/166	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper 2-6	K/166	C	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (9)	K/159	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (2-9)	K/159	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (9A)	K/159	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (2-9A)	K/159	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (10)	H/157	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (2-10)	H/157	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (10A)	H/157	T	HE, DDE, DE	(b)

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Barber Colman Actuator for Motorized Damper (2-10A)	H/157	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (11)	K/159	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (2-11)	K/159	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (11A)	K/159	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (2-11A)	K/159	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (12)	H/157	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (2-12)	H/157	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (12A)	H/157	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (2-12A)	H/157	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (13)	K/159	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (2-13)	K/159	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (14)	K/159	T	HE, DDE, DE	(b)
Barber Colman Actuator for Motorized Damper (2-14)	K/159	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (1)	A/143	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (2-1)	A/143	C	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (1A)	A/149	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (2-1A)	A/149	C	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (1B)	A/143	T	HE, DDE, DE	(b)
Limitorque Actuator for	A/143	C	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Motorized Damper (2-1B)				
Limitorque Actuator for Motorized Damper (1C)	A/149	T	HE, DDE, DE	(b)
Limitorque Actuator for Motorized Damper (2-1C)	A/149	C	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan S-1	L/91	T	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan 2S-1	L/91	C	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan S-2	L/91	T	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan 2S-2	L/91	C	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan S-31	K/150	T	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan S-33	K/150	C	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan S-32	K/150	T	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan S-34	K/150	C	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan E-1	L/135	T	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan 2E-1	L/135	C	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan E-2	L/135	T	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan 2E-2	L/135	C	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan E-4	L/146	T	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan 2E-4	L/146	C	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Pneumatic Contromatics Operator for Fan E-5	L/146	T	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan 2E-5	L/146	C	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan E-6	L/146	T	HE, DDE, DE	(b)
Pneumatic Contromatics Operator for Fan 2E-6	L/146	C	HE, DDE, DE	(b)
Fire Damper 12x14 (FD-128)	K/73	A	HE, DDE, DE	(b)
Fire Damper 12x14 (2FD-128)	K/73	C	HE, DDE, DE	(b)
Motor for Supply Fan OS-96 (CRPS)	A/140'	A	HE, DDE, DE	(b)
Motor for Supply Fan OS-97 (CRPS)	A/140'	A	HE, DDE, DE	(b)
Motor for Supply Fan OS-98 (CRPS)	A/140'	A	HE, DDE, DE	(b)
Motor for Supply Fan OS-99 (CRPS)	A/140'	A	HE, DDE, DE	(b)
Fire Damper 46x14 (FD-1)	H/111	A	HE, DDE, DE	(b)
Fire Damper 46x14 (2FD-1)	H/111	C	HE, DDE, DE	(b)
Fire Damper 46x14 (FD-2)	H/111	A	HE, DDE, DE	(b)
Fire Damper 46x14 (2FD-2)	H/111	C	HE, DDE, DE	(b)
Fire Damper 46x14 (FD-3)	H/110	A	HE, DDE, DE	(b)
Fire Damper 46x14 (2FD-3)	H/110	C	HE, DDE, DE	(b)
Fire Damper 32x68 (FD-24)	J/100	A	HE, DDE, DE	(b)
Fire Damper 32x68 (2FD-24)	J/100	C	HE, DDE, DE	(b)
Motors for Fans E-43, E-44, S-43 and S-44	H/163	A	HE, DDE, DE	(b)
Motors for Fans E-45, E-46, S-45 and S-46	H/163	C	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Fire Damper 42x24 (FD-25)	A/135	A	HE, DDE, DE	(b)
Fire Damper 42x24 (2FD-25)	A/135	C	HE, DDE, DE	(b)
Fire Damper 42x24 (FD-26)	A/135	A	HE, DDE, DE	(b)
Fire Damper 42x24 (2FD-26)	A/135	C	HE, DDE, DE	(b)
Fire Damper 42x24 (FD-27)	A/135	A	HE, DDE, DE	(b)
Fire Damper 42x24 (2FD-27)	A/135	C	HE, DDE, DE	(b)
Fire Damper 36x39 (FD-19)	A/140	A	HE, DDE, DE	(b)
Fire Damper 36x39 (2FD-19)	A/140	C	HE, DDE, DE	(b)
Fire Damper 36x39 (FD-20)	A/140	A	HE, DDE, DE	(b)
Fire Damper 36x39 (2FD-20)	A/140	C	HE, DDE, DE	(b)
Fire Damper 36x39 (FD-21)	A/140	A	HE, DDE, DE	(b)
Fire Damper 36x39 (2FD-21)	A/140	C	HE, DDE, DE	(b)
Fire Damper 14x10 (FD-26)	H/151	T	HE, DDE, DE	(b)
Fire Damper 14x10 (2FD-26)	H/151	T	HE, DDE, DE	(b)
Fire Damper 20x10 (FD-27)	H/153	T	HE, DDE, DE	(b)
Fire Damper 20x10 (2FD-27)	H/153	T	HE, DDE, DE	(b)
Fire Damper 12x12 (FD-28)	H/158	T	HE, DDE, DE	(b)
Fire Damper 12x12 (2FD-28)	H/158	T	HE, DDE, DE	(b)
Varicel-Roughing Filter (EFR-1)	L/120	A	HE, DDE, DE	(b)
Varicel-Roughing Filter (2EFR-1)	L/120	C	HE, DDE, DE	(b)
Varicel-Roughing Filter (EFR-2a)	L/122	A	HE, DDE, DE	(b)
Varicel-Roughing Filter (2EFR-2a)	L/122	C	HE, DDE, DE	(b)
Varicel-Roughing Filter (EFR-2b)	L/104	A	HE, DDE, DE	(b)
Varicel-Roughing Filter (2EFR-2b)	L/104	C	HE, DDE, DE	(b)
Electric Duct Heater (EH-27)	H/163	A	HE, DDE, DE	(b)
Electric Duct Heater (2EH-27)	H/163	A	HE, DDE, DE	(b)
Electric Duct Heaters (OEH-28A & 28B)	Tech. Support Center/109	A	HE, DDE, DE	(a) (b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Mode Damper 8 in. dia (0-17)	Tech. Support Center/109	A	HE, DDE, DE	(a) (b)
Ceiling Registers & Diffusers Wall Registers & Diffusers(Unit 1)	Aux./Varies	A	HE, DDE, DE	(b)
Ceiling Registers & Diffusers Wall Registers & Diffusers (Unit 2)	Aux./Varies	A, C	HE, DDE, DE	(b)
Aluminum Air Outlets (26" wide or less) "Metalaire"	Aux./Varies	A	HE, DDE, DE	(b)
Aluminum Air Outlets (26" wide or less) "Metalaire" for Unit 2	Aux./Varies	C	HE, DDE, DE	(b)
Air Monitors (AM FE-5001, 5013, 5015, 5016, 5018A, 5018B, 5019 and 5020) and Flow Evaluators (FE-5014, 5017A and 5017B).	Aux./Varies	A	HE, DDE, DE	(b)
Air Monitors (AM 2-FE-5001, 5002, 5003, 5004, 5005, 5006, 5007, 5008, 5009, 5010, 5011, 5012, 5013, 5015, 5019 and 5020) and Flow Evaluators (2-FE-5014, 5017A, 5017B, 5018A, 5018B).	Aux./Varies	C	HE, DDE, DE	(b)
Air Flow Controllers Johnson Service R-317-1 for Fans	Aux./Varies	T	HE, DDE, DE	(b)
Pressure Reducing Valve Johnson Service R-130-A for Fan & Dampers	Aux./Varies	T	HE, DDE, DE	(b) (g)
Restrictors Johnson Service T-5210-100 for Fans	Aux./Varies	T	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Solenoid Valves ASCO HT-8316-B15, C15 and D45; HT-8320-A20, A24 and A185; and HT-8331-A45 for Fans & Dampers	Aux./Varies	T	HE, DDE, DE	(b)
Speed Controllers ASCO VO221	Aux./Varies	T	HE, DDE, DE	(b)
Speed Controllers ASCO V0222 and VO223	Aux./Varies	C	HE, DDE, DE	(b)
Position Switches NAMCO D-2400-X-R2-WS for Dampers	Aux./Varies	T, C	HE, DDE, DE	(b)
Brandt Air Flow Controller (Pi-DPT-2000)	Aux./Varies	T, A	HE, DDE, DE	(b)
Portion of Refrigerant Piping with Solenoid Valve Exp. Valve with Sight Glass	H/158	T	HE, DDE, DE	(b)
Position Switches Allen-Bradley 802T-HW1 for Dampers	Aux./Varies	T	HE, DDE, DE	(b)
Position Switches Allen-Bradley 802T-HW1 for Dampers	Aux./Varies	C	HE, DDE, DE	(b)
Air Flow Switches Dwyer 1638 & 1640 for Fans	Aux./Varies, Turbine bldg., Technical Support Center/Varies	T, A	HE, DDE, DE	(a) (b)
Air Flow Switches McDonald-Miller AF1-S for Fans	Aux./Varies	T, A	HE, DDE, DE	(a) (b)
Thermostats Honeywell T8400C1057 or TH5110D1006	H/145	T	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Thermostats Penn A28AA	K/145	T	HE, DDE, DE	(b)
Thermostats Penn T26J-2	A/124	T	HE, DDE, DE	(b)
Thermostats Penn T26S-18	A/124	C	HE, DDE, DE	(b)
Thermostats Johnson Controls T26S-18C	A/124	C	HE, DDE, DE	(h)
Control Relay Cabinets for CRC-1 and CRC-3	H/157	T, A	HE, DDE, DE	(b)
Control Relay Cabinets for CRC-6 and CRC-8	H/157	T, A	HE, DDE, DE	(b)
Common Control Relay Cabinets for CCRC-2	H/157	T, A	HE, DDE, DE	(b)
Common Control Relay Cabinets for CCRC-7	H/157	T, A	HE, DDE, DE	(b)
Control Panels for Compressors CP-35 & CP-36	H/156	T	HE, DDE, DE	(b)
Control Panels for Compressors CP-37 & CP-38	H/156	T	HE, DDE, DE	(b)
Heating Relay in Cabinet 3	H/157	T	HE, DDE, DE	(b)
Thermostat Honeywell Model T675A1565 & T6031A1029	H/154	T	HE, DDE, DE	(b)
Motors for Fans S-1, S-2, S-31, S-32, S-39, S-40, E-1, E-2, E-4, E-5, E-6, E-101, E-102, E-103 and E-104	Aux./Varies Intake Structure/Varies	T	HE, DDE, DE	(b)
Motors for Fans	Aux./Varies	C	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
2S-1, 2S-2, S-33, S-34, S-41, S-42, 2E-1, 2E-2, 2E-4, 2E-5 and 2E-6	Intake Structure/Varies			
Motors for Fans CR-35, CR-36, S-35, S-36, S-67, S-68, S-69	Aux./Varies & Turbine /Varies	T, C, A	HE, DDE, DE	(b)
Motors for Fans CR-37, CR-38, S-37, S-38, 2S-67, 2S-68, 2S-69	Aux./Varies & Turbine/Varies	C, A	HE, DDE, DE	(b)
Flex connection in 48-in. ϕ Purge Air Supply Duct FC-1 & FC-2	L/Varies	A	HE, DDE, DE (Bldg. Displ. Per DCM C-28)	(b)
Flex connection in 48-in. ϕ Purge Air Supply Duct 2FC-1 & 2FC-2	L/Varies	C	HE, DDE, DE (Bldg. Displ. per DCM C-28)	(b)
Flex connection in 12-in. ϕ Excess Pressure Relief Duct FC-3 & FC-4	L/Varies	A	HE, DDE, DE (Bldg. Displ. per	(b)
Flex connection in 12-in. ϕ Excess Pressure Relief Duct 2FC-3 & 2FC-4	L/Varies	C	HE, DDE, DE (Bldg. Displ. per	(b)
Flex Connections in 14-in. Pipes OFC-11, through OFC-16, OFC-18 through OFC-21	Turbine bldg. /varies	A	HE, DDE, DE (Bldg. Displ. per DCM C-28)	(b)
Flex Connection in 14-in. Pipe OFC-17	Between Aux. bldg. & Turbine bldg./ 167	A	HE, DDE, DE (Bldg. Displ. per DCM C-28)	(b)
Flex Connection in 14-in. Pipe OFC-22	Aux. bldg./163	A	HE, DDE, DE (Bldg. Displ. Per DCM C-28)	(b) (d)
Flex Connection in 14-in.	Aux. bldg./163	A	HE, DDE, DE	(b) (d)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Pipe OFC-22 Unit 2			(Bldg. Displ. Per DCM C-28)	
Nutherm/Cleveland Airflow Switches Model AFS-951-1 for over heater in CRPS	H/169	T, A	HE, DDE, DE	(a) (b)
Motors for Compressors CP-35 and CP-36	H/154	A	HE, DDE, DE	(b)
Motors for Compressors CP-37 and CP-38	H/154	C	HE, DDE, DE	(b)
Fire Damper 24x12 (FD-7)	H/110	A	HE, DDE, DE	(b)
Fire Damper 24x12 (2FD-7)	H/110	C	HE, DDE, DE	(b)
Fire Damper 24x12 (FD-8)	H/110	A	HE, DDE, DE	(b)
Fire Damper 24x12 (2FD-8)	H/110	C	HE, DDE, DE	(b)
Fire Damper 24x12 (FD-9)	H/110	A	HE, DDE, DE	(b)
Fire Damper 24x12 (2FD-9)	H/110	C	HE, DDE, DE	(b)
Fire Damper (FD-34)	H/127	T	HE, DDE, DE	(b)
Fire Damper (2FD-34)	H/127	C	HE, DDE, DE	(b)
Fire Damper (FD-36)	H/127	T	HE, DDE, DE	(b)
Fire Damper (2FD-36)	H/127	C	HE, DDE, DE	(b)
Fire Damper (FD-38)	H/126	T	HE, DDE, DE	(b)
Fire Damper (FD-39)	H/126	T	HE, DDE, DE	(b)
Fire Damper (FD-40)	H/123	T	HE, DDE, DE	(b)
Fire Damper (FD-43)	A/119	T	HE, DDE, DE	(b)
Fire Damper (2FD-43)	A/119	C	HE, DDE, DE	(b)
Fire Damper (FD-44)	A/119	T	HE, DDE, DE	(b)
Fire Damper (2FD-44)	A/119	C	HE, DDE, DE	(b)
Fire Damper (FD-45)	A/119	T	HE, DDE, DE	(b)
Fire Damper (2FD-45)	A/119	C	HE, DDE, DE	(b)
Fire Damper (FD-10)	H/121	A	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location(c) Building/ Elevation, ft	Qualification Method(c)	Qualifying Spectra(c)	Notes
Fire Damper (2FD-10)	H/121	C	HE, DDE, DE	(b)
Fire Damper (FD-11)	H/121	A	HE, DDE, DE	(b)
Fire Damper (2FD-11)	H/121	C	HE, DDE, DE	(b)
Fire Damper (FD-12)	H/121	A	HE, DDE, DE	(b)
Fire Damper (2FD-12)	H/121	C	HE, DDE, DE	(b)
Smoke Damper (SD-26)	H/151	A	HE, DDE, DE	(b)
Smoke Damper (2SD-26)	H/151	C	HE, DDE, DE	(b)
Smoke Damper (SD-27)	H/151	A	HE, DDE, DE	(b)
Smoke Damper (2SD-27)	H/151	C	HE, DDE, DE	(b)
Smoke Damper (SD-35)	H/127	A	HE, DDE, DE	(b)
Smoke Damper (2SD-35)	H/127	C	HE, DDE, DE	(b)
Smoke Damper (SD-37)	H/127	A	HE, DDE, DE	(b)
Smoke Damper (2SD-37)	H/127	C	HE, DDE, DE	(b)
Pneumatic Bettis Actuator for Damper (4A)	Aux/Varies	T	HE, DDE, DE	(b)
Pneumatic Bettis Actuator for Damper (2-4A)	Aux/Varies	T	HE, DDE, DE	(b)
Pneumatic Bettis Actuator for Damper (4B)		T	HE, DDE, DE	(b)
Pneumatic Bettis Actuator for Damper (2-4B)	Aux/Varies	T	HE, DDE, DE	(b)
Pneumatic Bettis Actuator for Damper (8A)		T	HE, DDE, DE	(b)
Pneumatic Bettis Actuator for Damper (2-8A)	Aux/Varies	T	HE, DDE, DE	(b)
Pneumatic Bettis Actuator for Damper (8B)		T	HE, DDE, DE	(b)
Pneumatic Bettis Actuator for Damper (2-8B)	Aux/Varies	T	HE, DDE, DE	(b)
Pneumatic Bettis Actuator for Damper (10)		T	HE, DDE, DE	(b)
Pneumatic Bettis Actuator for Damper (2-10)	Aux/Varies	T	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Power Regulator Co. Actuator for Damper (12)	Aux/Varies	T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (16A)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-16A)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (16B)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-16B)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (17A)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-17A)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (17B)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-17B)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (20)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-20)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (21)	Aux/Varies	T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-21)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (22A)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-22A)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (22B)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-22B)	Aux/Varies	C	HE, DDE, DE	(b)

TABLE 3.10-3

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Power Regulator Co. Actuator for Damper (23A)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-23)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (23B)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-23B)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (24A)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-24)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (24B)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-24B)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (25A)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-25)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (25B)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-25B)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (26A)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-26A)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (26B)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-26B)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (33)		T	HE, DDE, DE	(b)
Power Regulator Co. Actuator for Damper (2-33)	Aux/Varies	C	HE, DDE, DE	(b)
Power Regulator Co. Actuator		T	HE, DDE, DE	(b)

Equipment	Location[(c)] Building/ Elevation, ft	Qualification Method[(c)]	Qualifying Spectra[(c)]	Notes
for Damper (34)				
Power Regulator Co. Actuator for Damper (2-34)	Aux/Varies	C	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Mode Damper (1A)	L/127	A	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Mode Damper (2-1A)	L/127	A	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Mode Damper (1B)	L/131	C	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Mode Damper (2-1B)	L/131	C	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Damper (13A)	K/97	A	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Damper (2-13A)	K/97	C	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Damper (13B)	K/97	A	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Damper (2-13B)	K/97	C	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Damper (14A)	K/81	A	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Damper (2-14A)	K/81	C	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Damper (14B)	K/81	A	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Damper (2-14B)	K/81	C	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Damper (15A)	Aux/70	A	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Damper (2-15A)	Aux/70	C	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin Actuator for Damper (15B)	Aux/70	A	HE, DDE, DE	(b)
Pneumatic Parker-Hannifin	Aux/70	C	HE, DDE, DE	(b)

Equipment	Location[c] Building/ Elevation, ft	Qualification Method[c]	Qualifying Spectra[c]	Notes
Actuator for Damper (2-15B)				
Position Switches NAMCO SL-3B1W & SL-170D for Dampers	L/115	T	HE, DDE, DE	(b)

(a) Turbine building, Unit 2, response spectra applicable for Qualification Spectra of equipment in the Technical Support Center.

(b) Envelope of 4% HE and 2% DDE Acceleration used in Qualification Spectra. Per DCM T-10 acceptance criteria, DE stresses shall not exceed the maximum allowable stress values specified in building codes (Uniform Building Code, 1973 and AISC, 1969). Increase in allowable stresses, permitted by code for seismic loads, shall not be used. In lieu of these, DDE and Hosgri stresses shall not exceed 90% of the yield strength and 150% of the AISC allowable stress, respectively.

(c) Legend:

ISA = Intake structure area
A = Qualification Spectra by analysis (Qualification Method column)
T = Qualification Spectra by testing
C = Comparison with similarly qualified equipment
DE = Design Earthquake
DDE = Double Design Earthquake
HE = Hosgri Earthquake

The letters in the Location column refer to standard area designations as defined in Figure 1.2-3, Piping and Mechanical Area Location Plan.

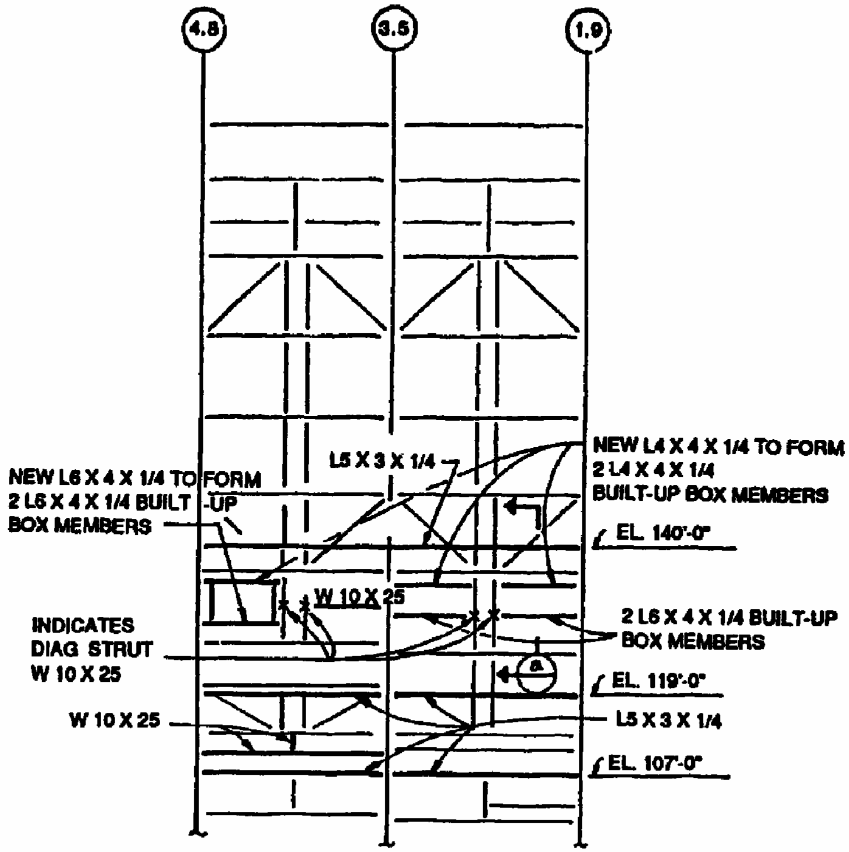
(d) Tag number of flexible connection OFC-22 has been duplicated.

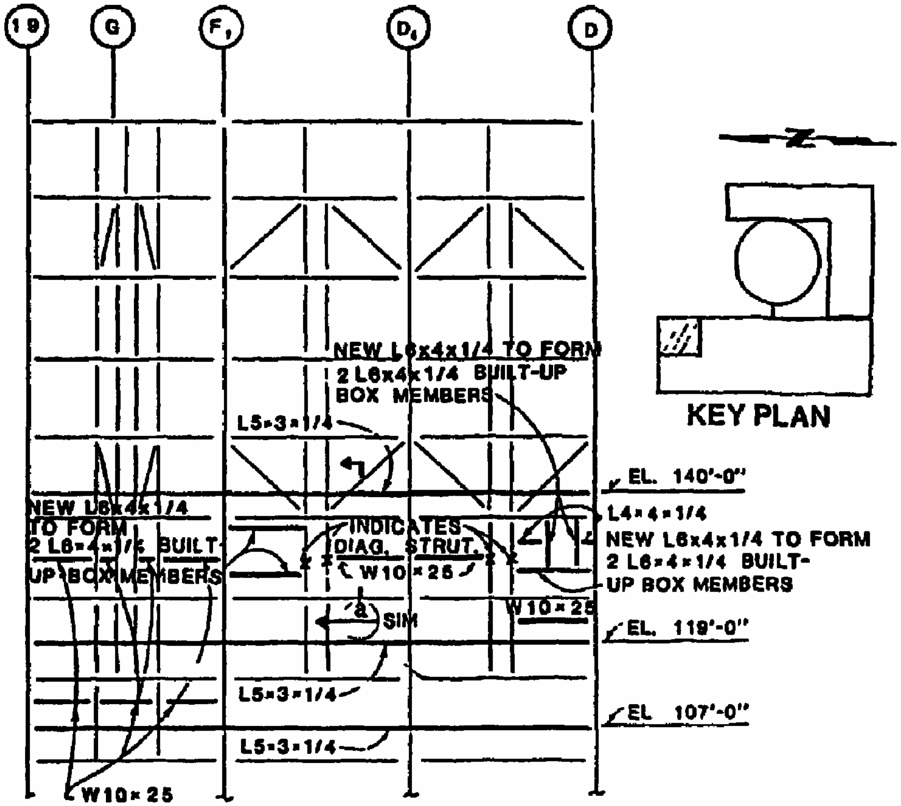
(e) Quadrant dampers supported on flexible slab.

(f) Applicable to Units 1 and 2.

(g) Due to obsolescence of Johnson air pressure reducing regulators, Fisher 67CSR regulators may be installed.

(h) Due to obsolescence of Penn thermostats, Johnson Controls T26S-18C thermostats may be installed.





NORTH ELEV. GIRT FRAMING ALONG ①

1/16" = 1'-0"

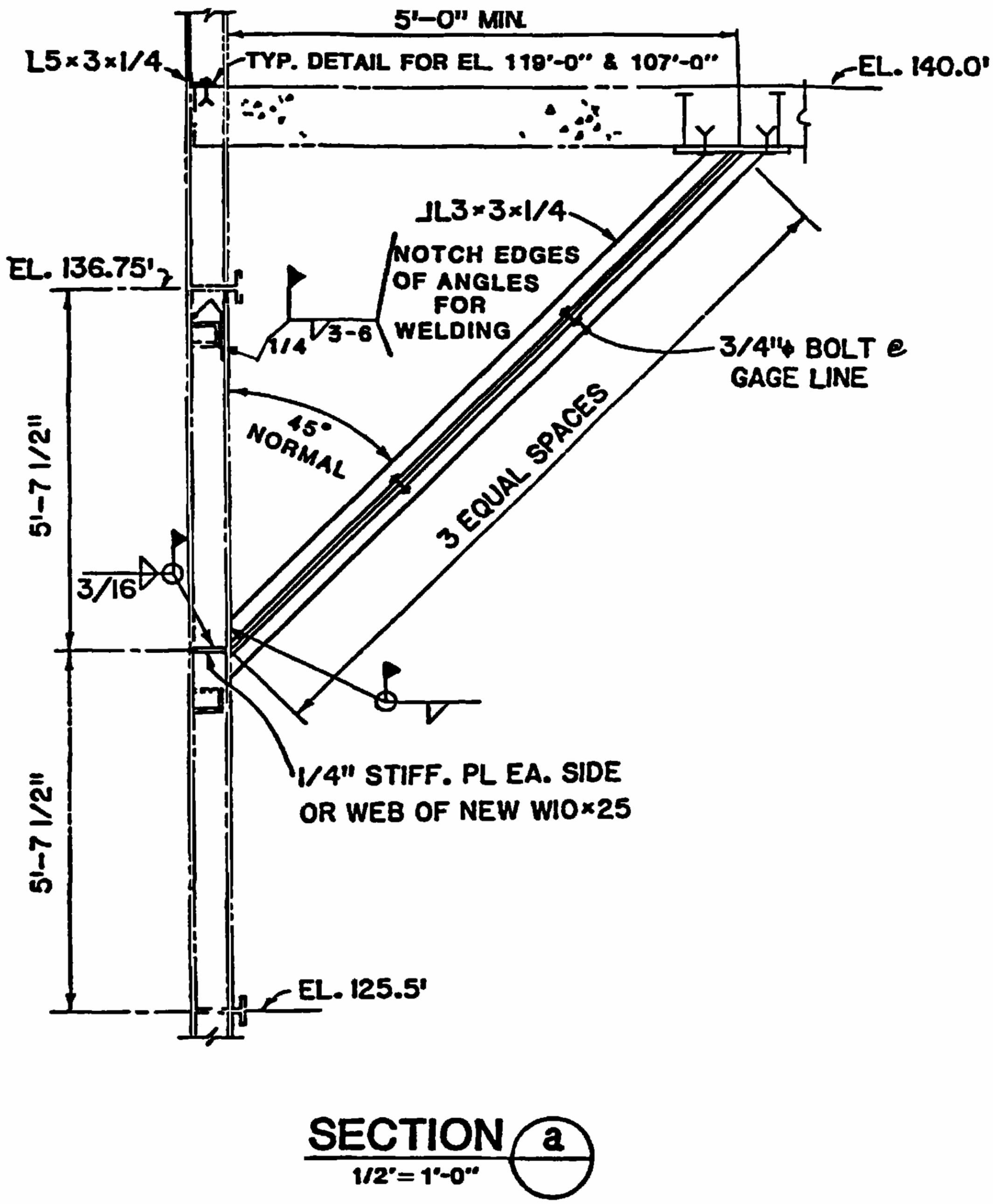
NOTES:

1 NEW MEMBERS ARE INDICATED BY HEAVY. DARK LINES
2. MODIFICATIONS AT NORTH AND EAST SIDE OF UNIT 1 ARE SHOWN. MODIFICATIONS AT SOUTH AND EAST SIDE OF UNIT 2 ARE IDENTICAL.

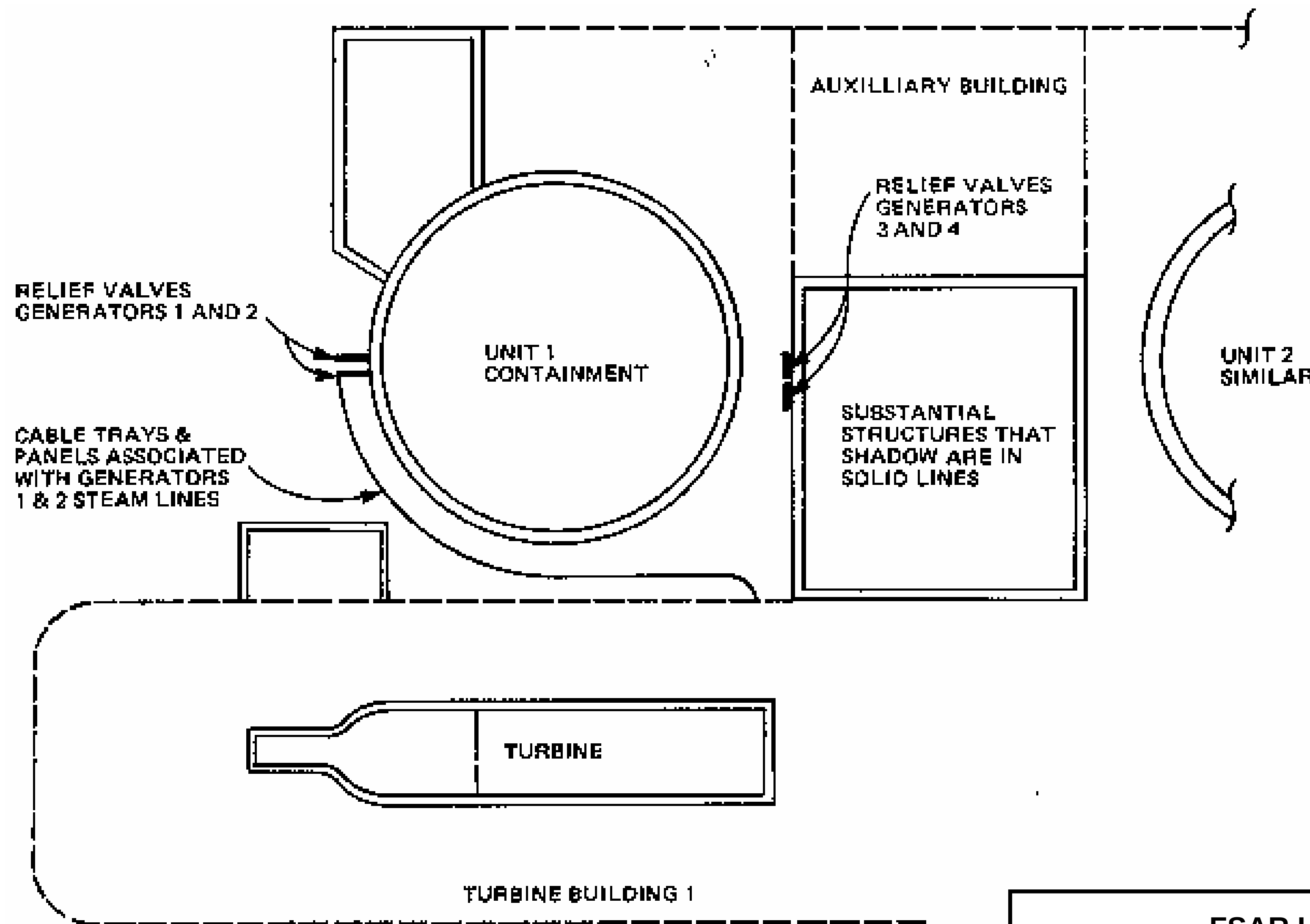
EAST ELEV. GIRT FRAMING ALONG Ⓖ

1/16" 1'-0"

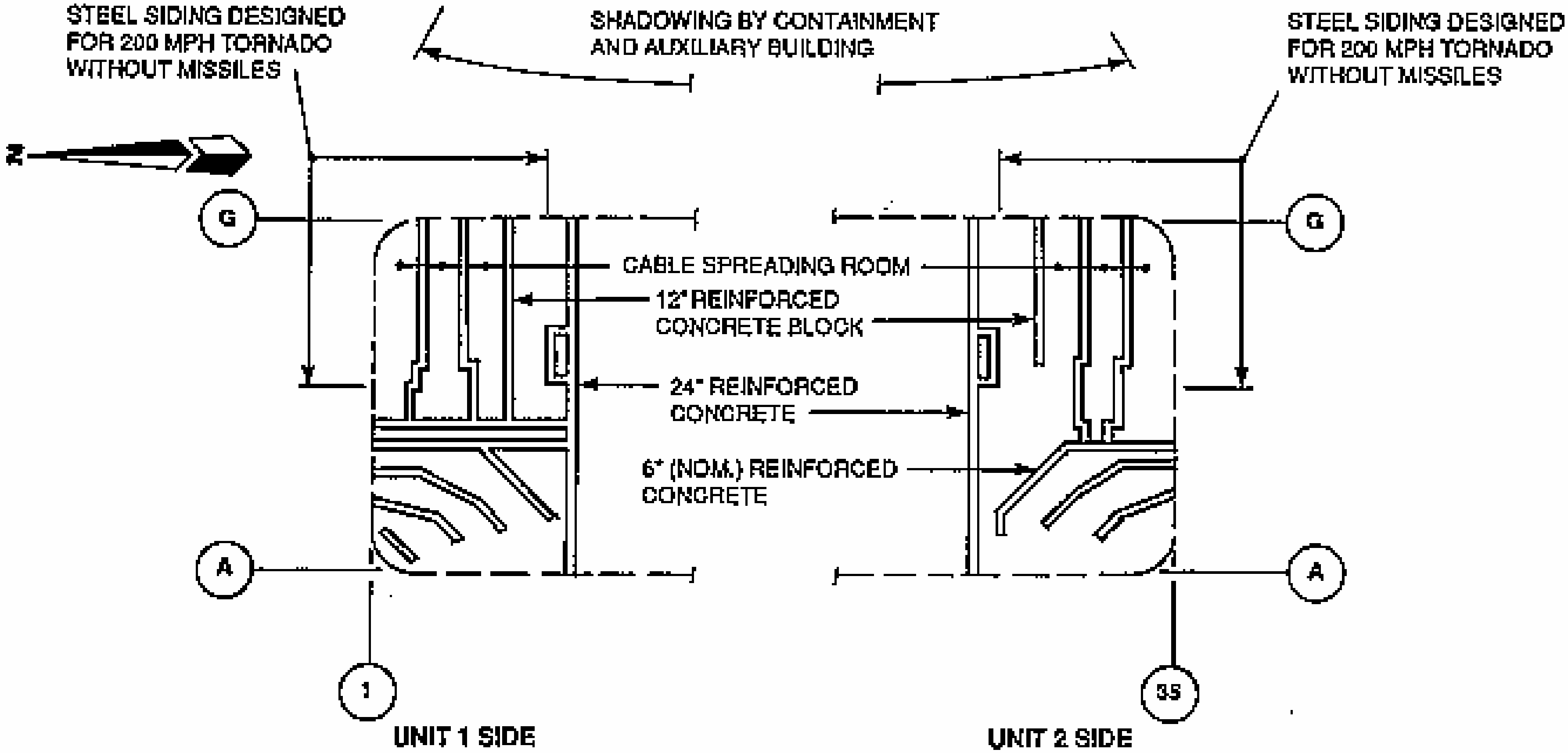
FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 3.3-1 TURBINE BUILDING FRAMING MODIFICATIONS FOR TORNADO RESISTANCE - ELEVATION



FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 3.3-2 TURBINE BUILDING FRAMING MODIFICATIONS FOR TORNADO RESISTANCE - SECTION



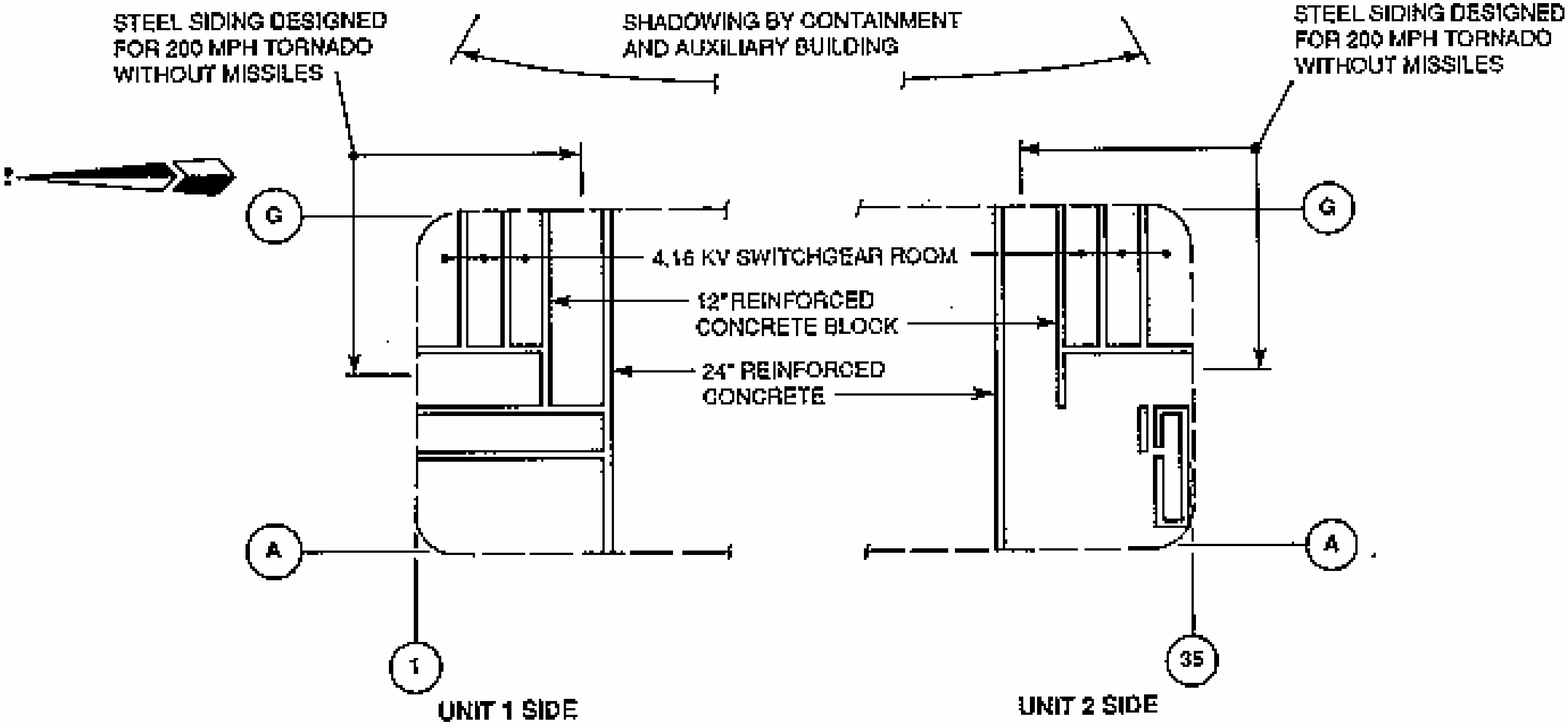
FSAR UPDATE
UNITS 1 AND 2 **DIABLO CANYON SITE**
FIGURE 3.3-3 **LAYOUT OF VULNERABLE MAIN STEAM RELIEF VALVES AND CABLE TRAY OUTSIDE OF PLANT**



PLAN @ EL 107'-0"

1. CABLE SPREADING ROOMS HAVE 10" REINFORCED CONCRETE FLOORS ABOVE AND BELOW.
2. ALL WALLS SHOWN ARE 8" REINFORCED CONCRETE MASONRY EXCEPT AS NOTED.

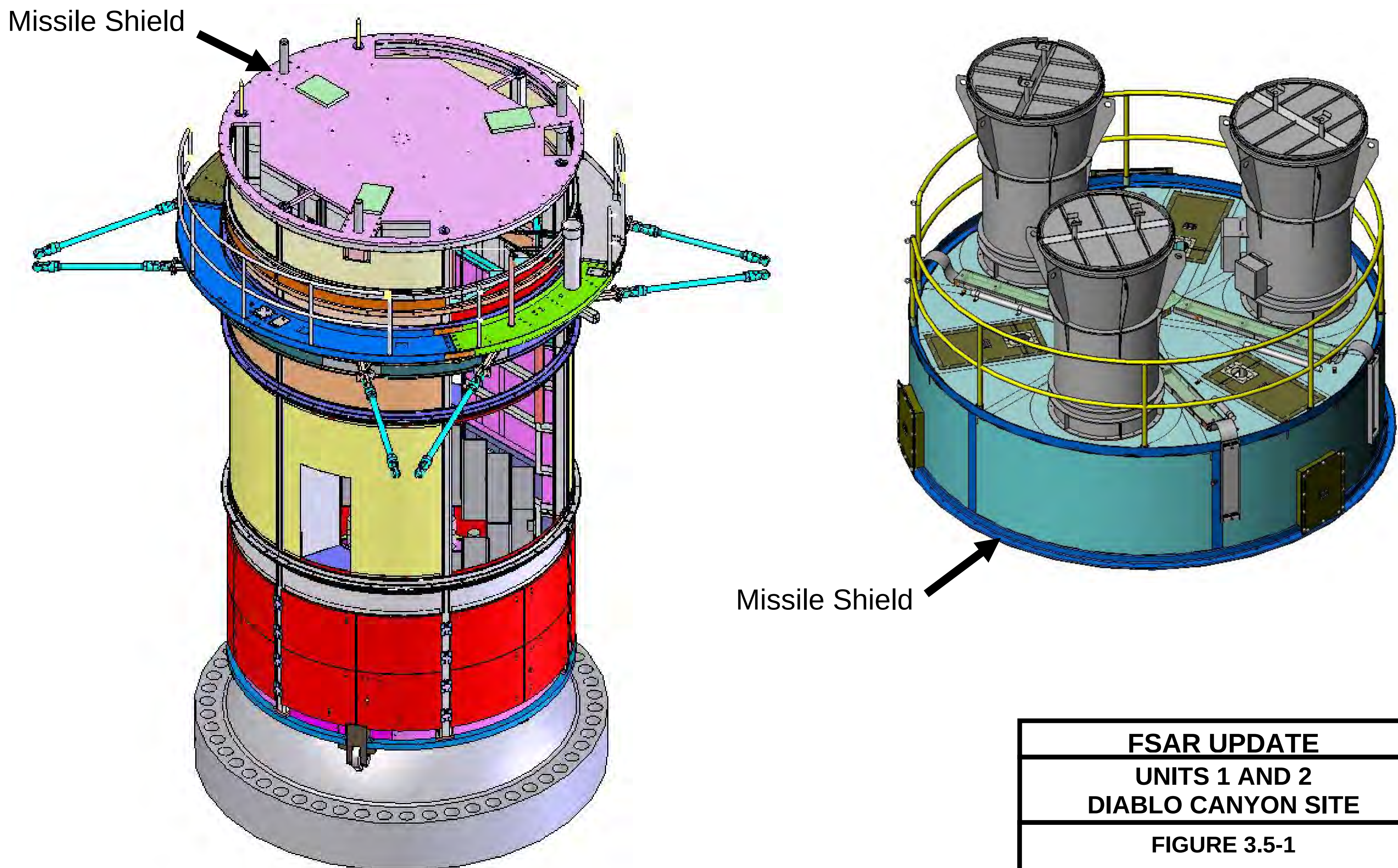
FSAR UPDATE
UNITS 1 AND 2 **DIABLO CANYON SITE**
FIGURE 3.3-4 (SHEET 1 OF 2) **SCHEMATIC LAYOUT OF CABLE SPREADING AND SWITCHGEAR ROOMS IN THE TURBINE BUILDING**



PLAN @ EL 119'-0"

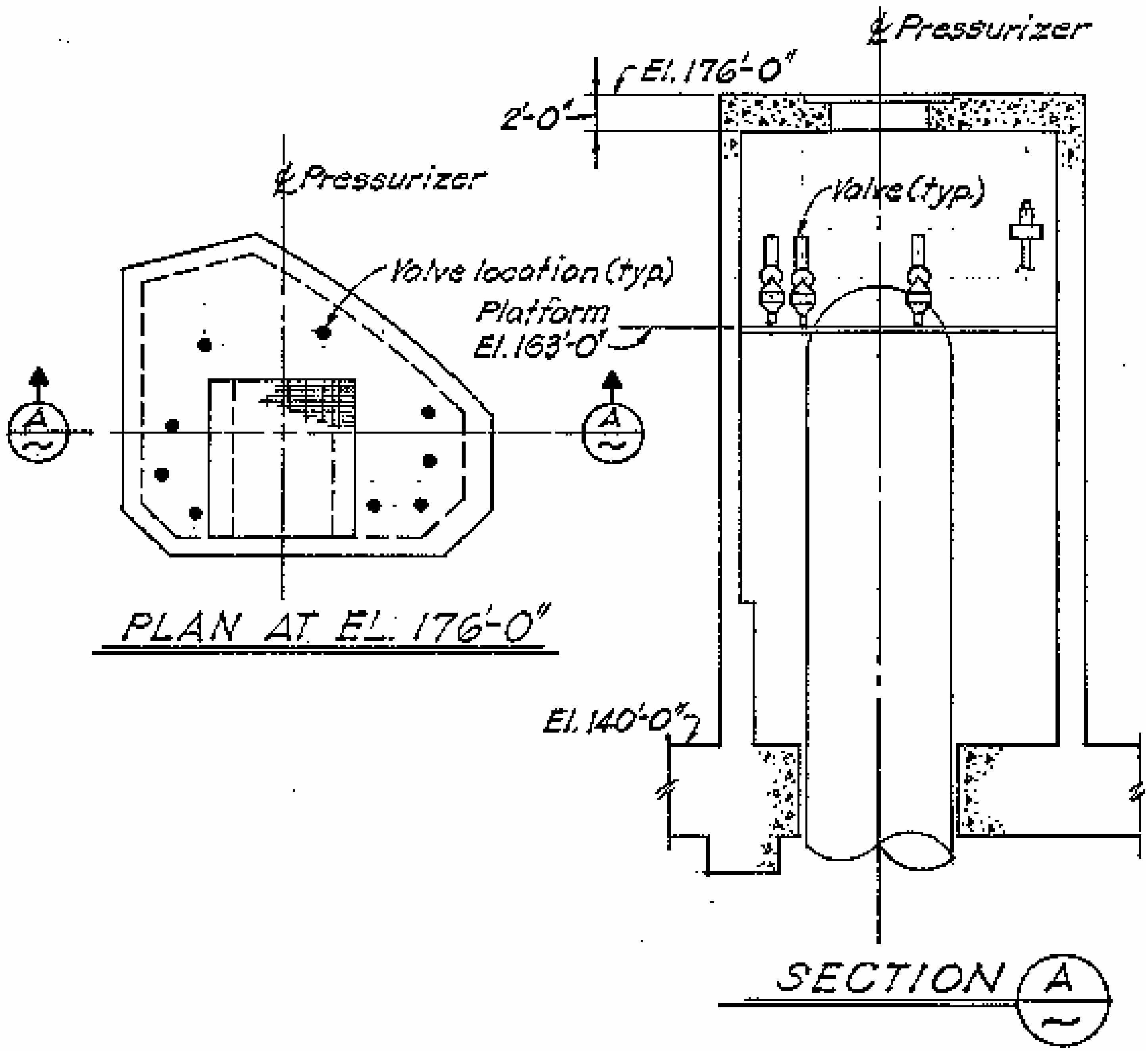
1. SWITCHGEAR ROOMS HAVE 12" REINFORCED CONCRETE FLOORS ABOVE AND 10" REINFORCED CONCRETE FLOORS BELOW.
2. ALL WALLS SHOWN ARE 8" REINFORCED CONCRETE MASONRY EXCEPT AS NOTED

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 3.3-4 (SHEET 2 OF 2) SCHEMATIC LAYOUT OF CABLE SPREADING AND SWITCHGEAR ROOMS IN THE TURBINE BUILDING



FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 3.5-1 INTEGRATED HEAD ASSEMBLY REACTOR MISSILE SHIELD

Revision 20 November 2011



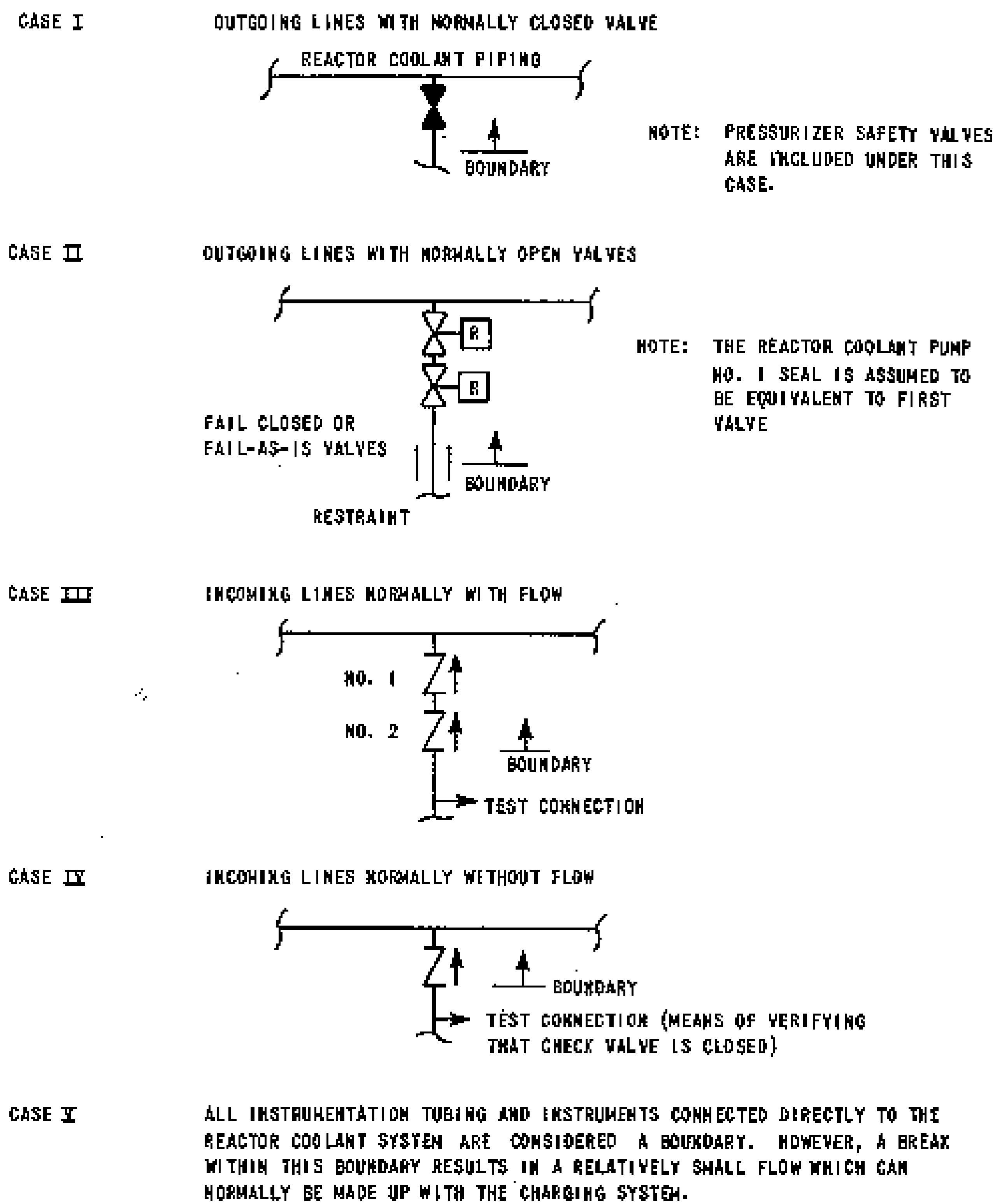
FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 3.5-2

CONTAINMENT STRUCTURE
PRESSURIZER MISSILE SHIELD

Revision 11 November 1996

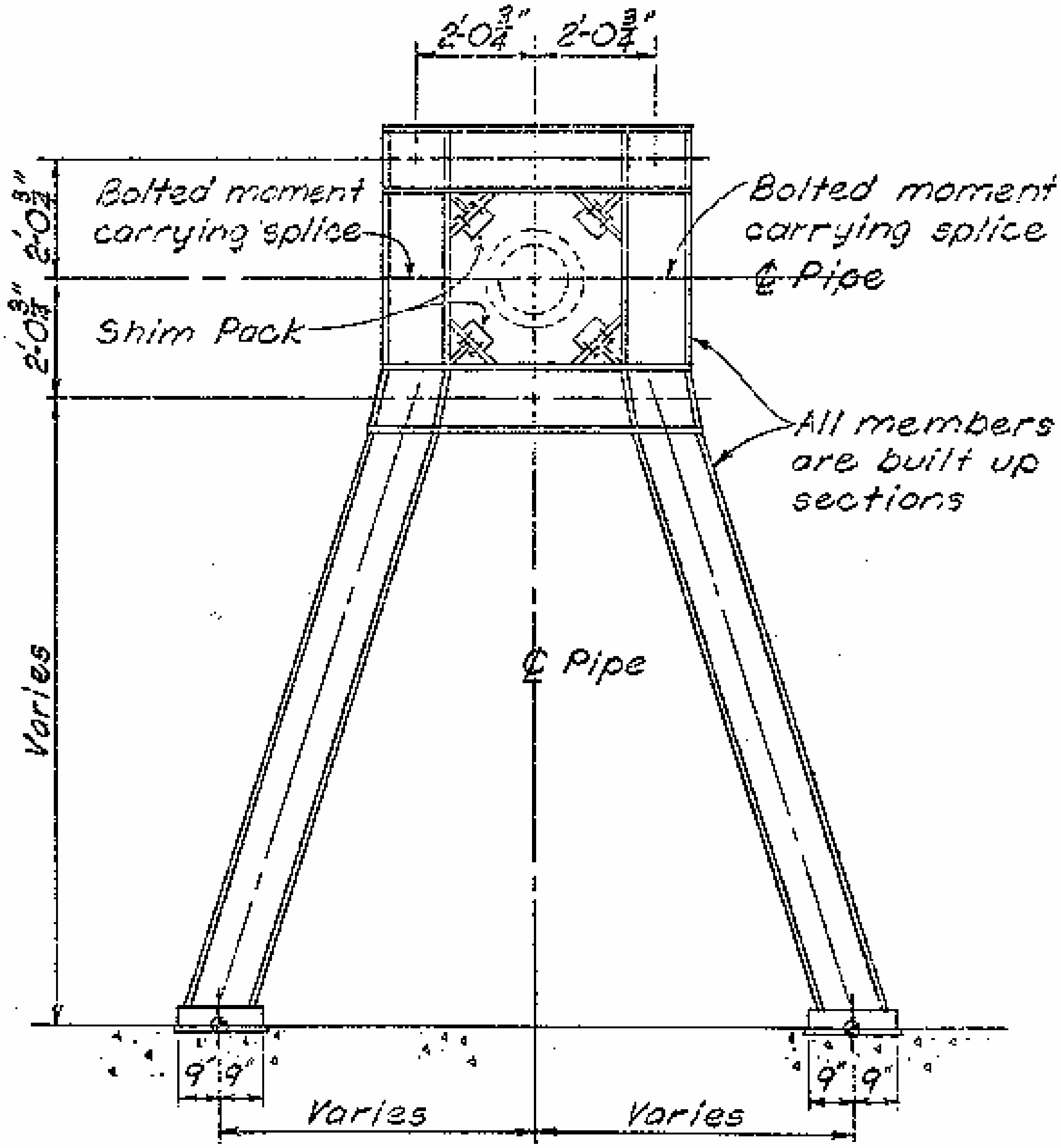


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 3.6-1

LOSS OF COOLANT ACCIDENT BOUNDARY LIMITS

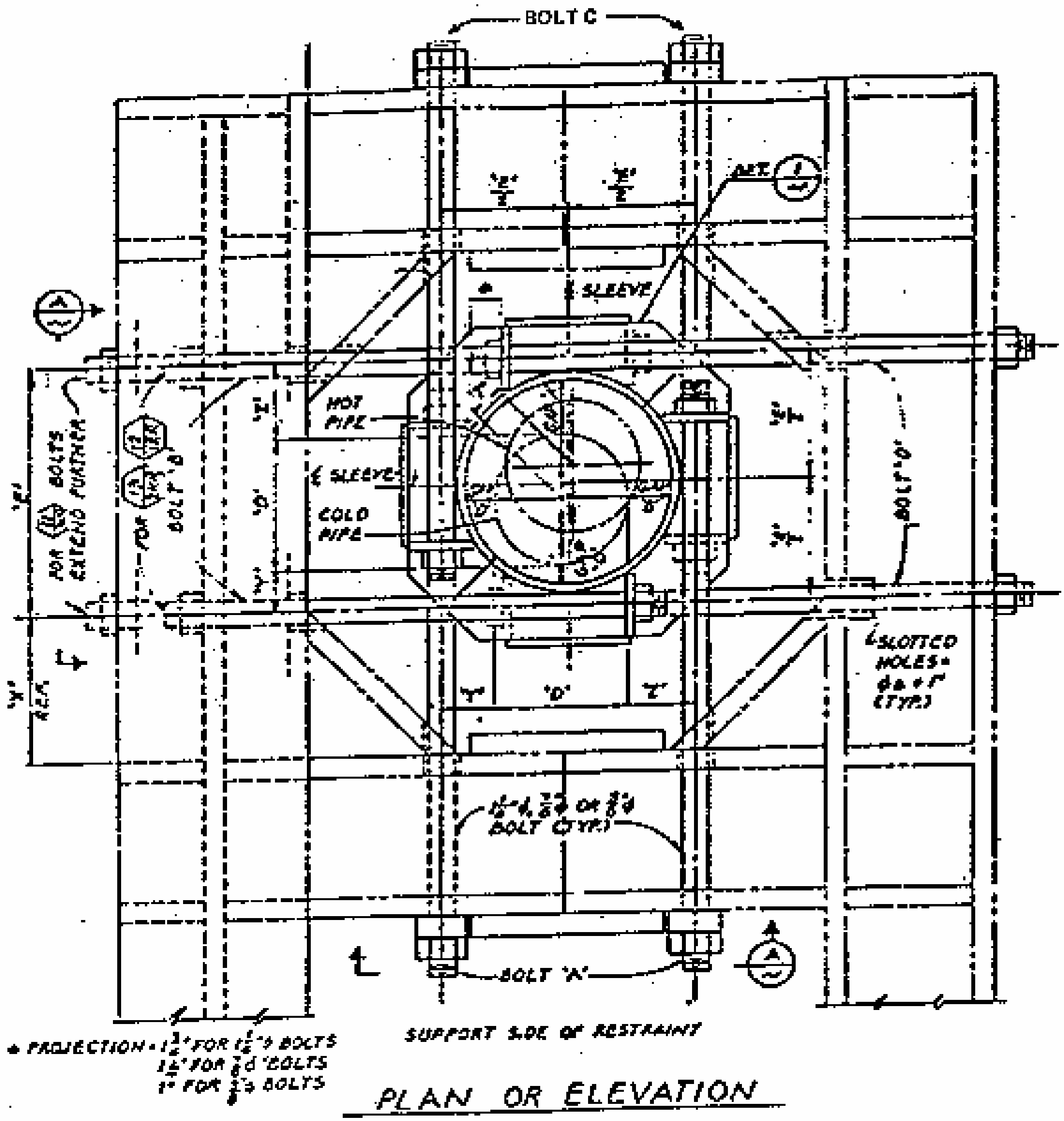


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 3.6-3

TYPICAL PIPE RUPTURE RESTRAINT

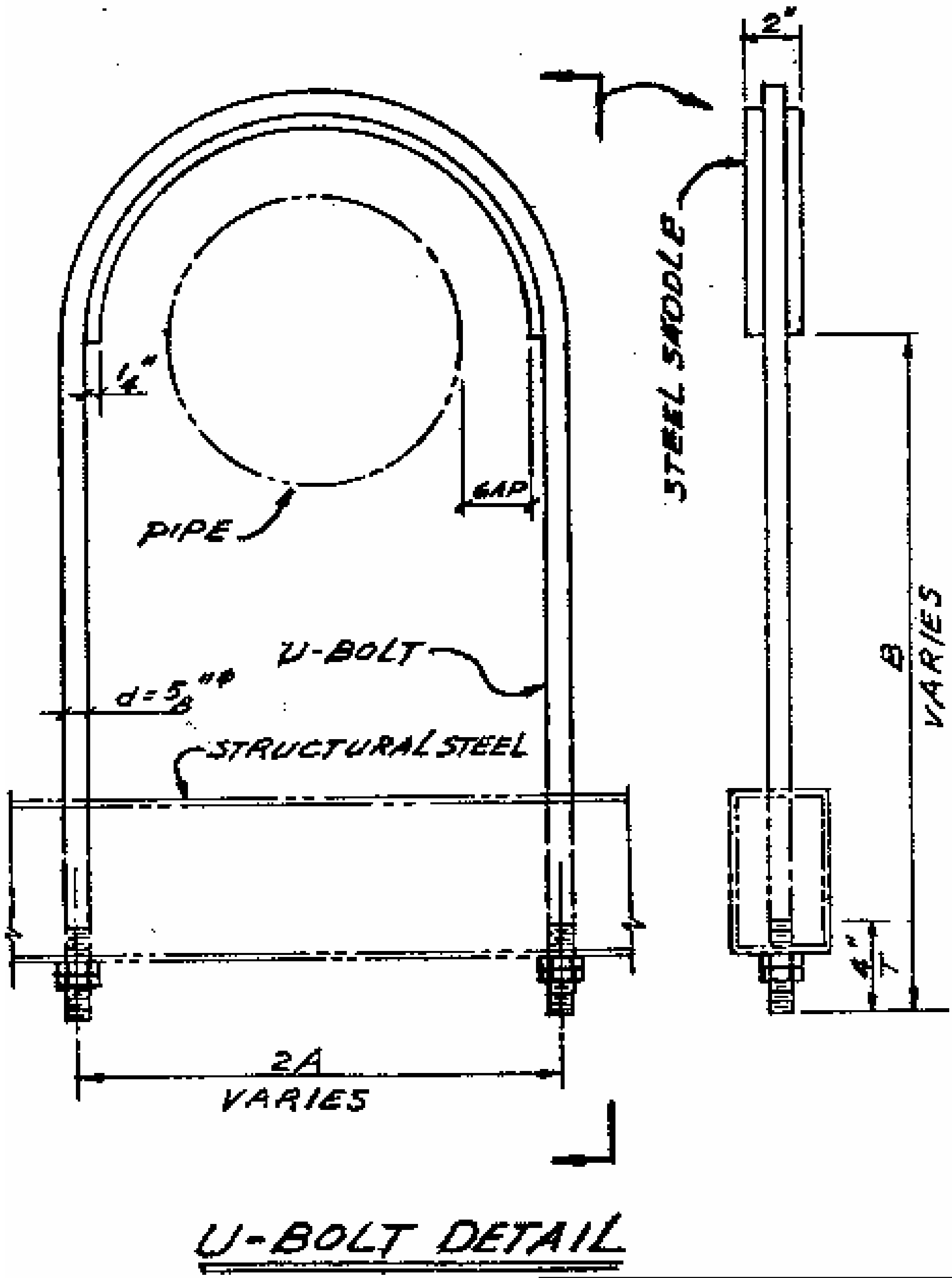


FSAR UPDATE

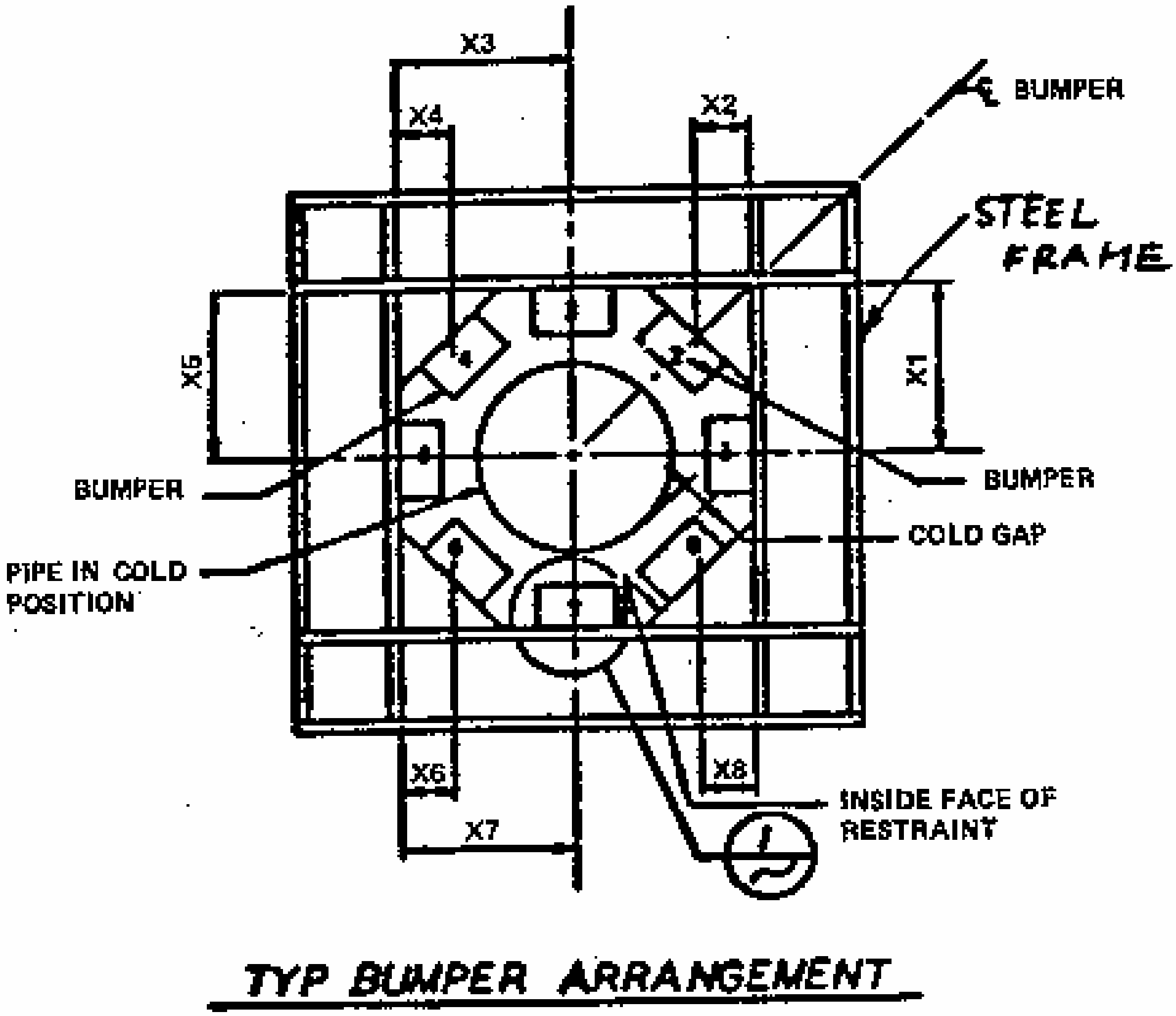
UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 3.6-3A

PIPE RUPTURE RESTRAINT
TYPICAL ROD ARRANGEMENT

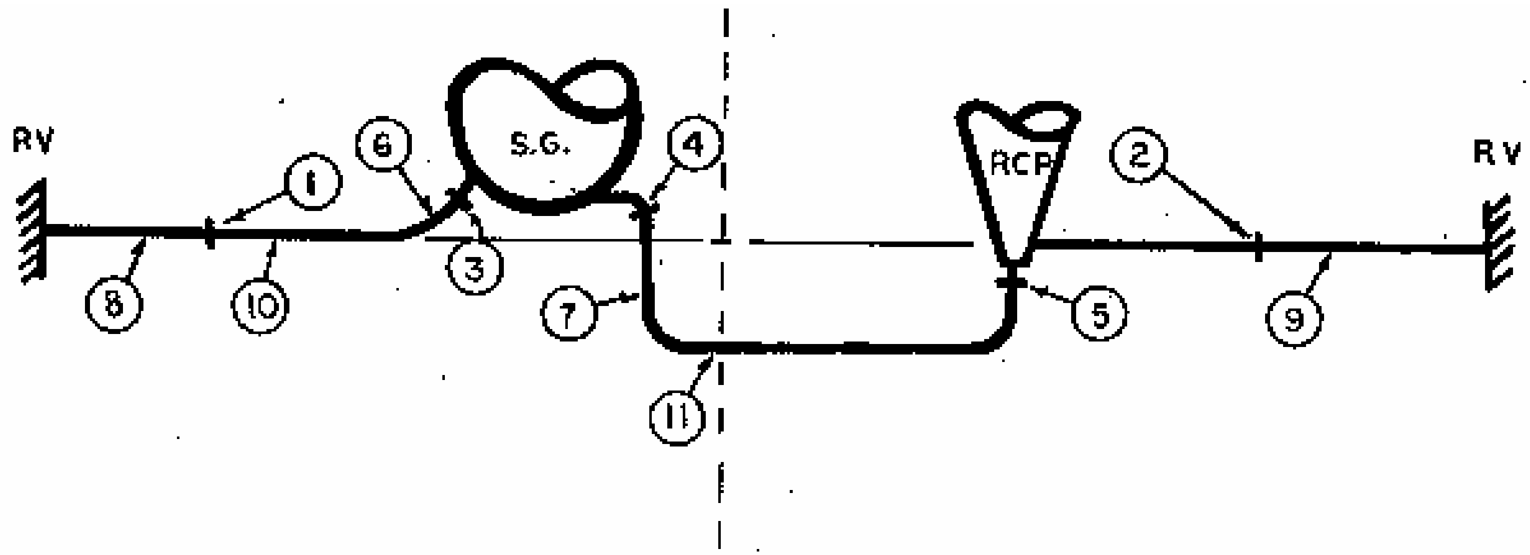


FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 3.6-3B PIPE RUPTURE RESTRAINT TYPICAL U-BOLT



FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 3.6-3C PIPE RUPTURE RESTRAINT CRUSHABLE BUMPER ARRANGEMENT

Revision 11 November 1996



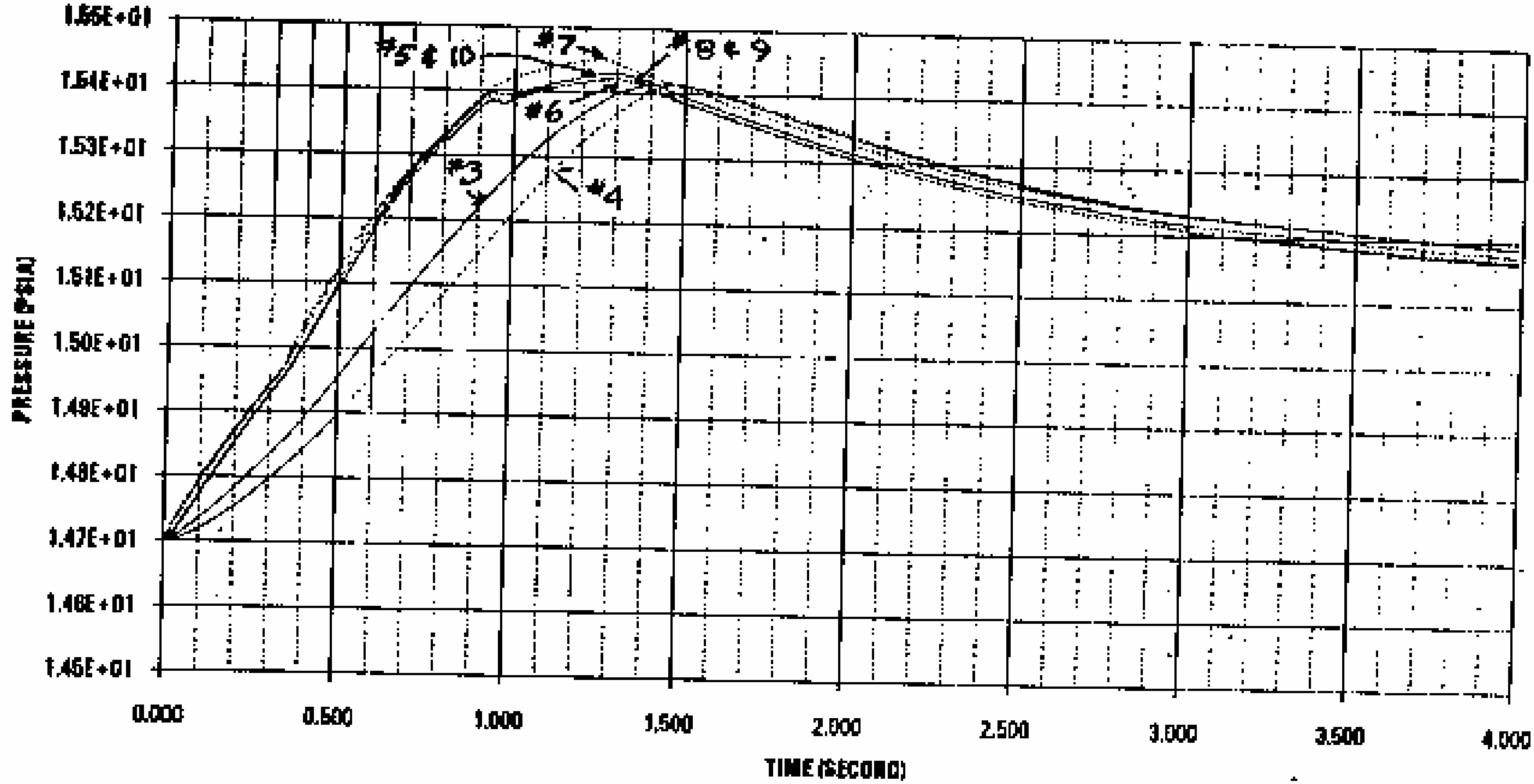
RV = REACTOR VESSEL
SG = STEAM GENERATOR
RCP = REACTOR COOLANT PUMP

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 3.6-4

PRIMARY COOLANT LOOP BREAKS
(See Section 3.6.2.2.1)

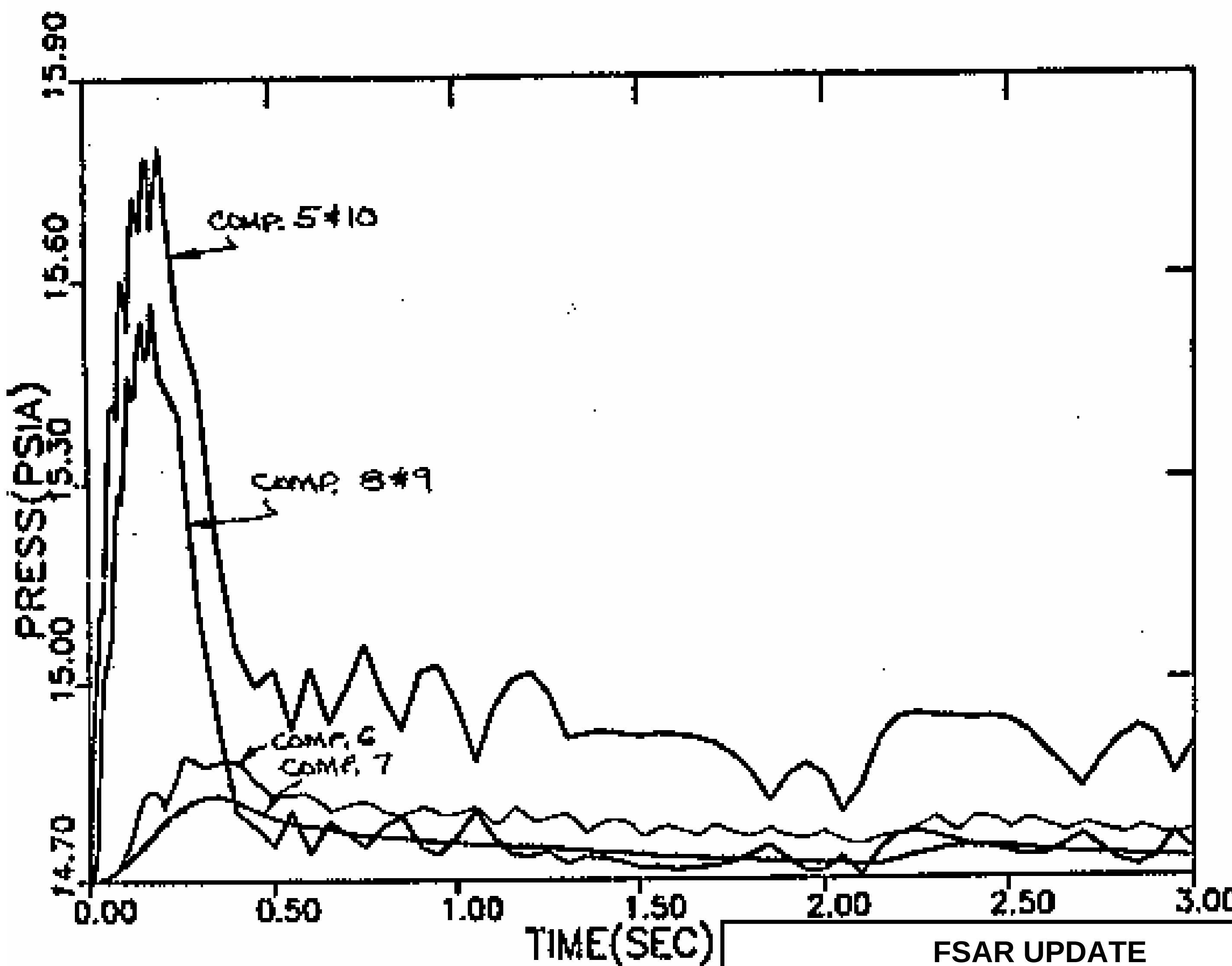


FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 3.6-16

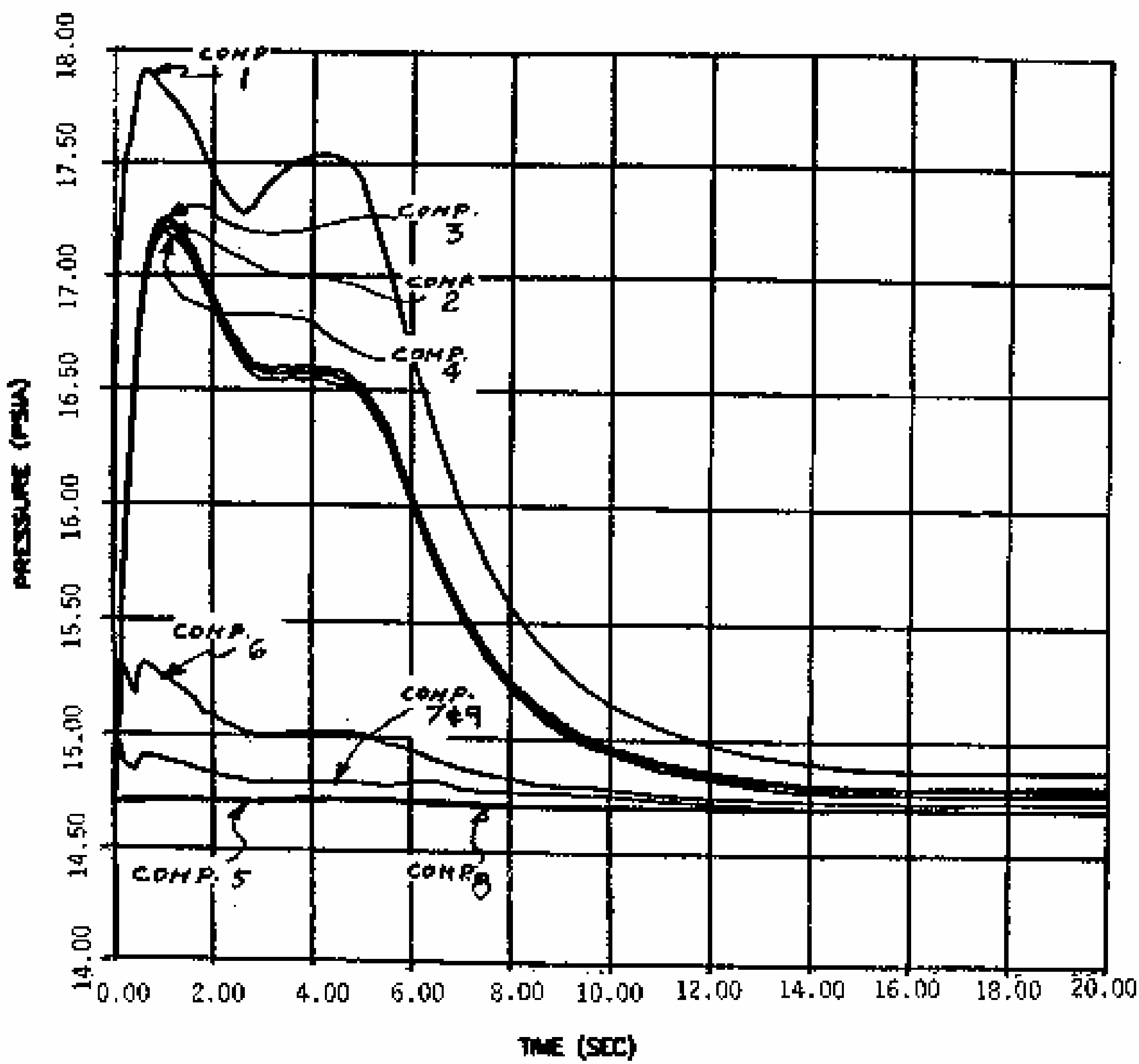
TURBINE BUILDING
PRESSURE vs. TIME RESPONSE
MSLB AT ELEVATION 140'
(DER) STEAM WITH ENTRAINMENT



FSAR UPDATE

UNIT 1
DIABLO CANYON SITE

FIGURE 3.6-17
TURBINE BUILDING
PRESSURE vs. TIME RESPONSE
MSLB AT ELEVATION 85'
(DER) STEAM WITH ENTRAINMENT

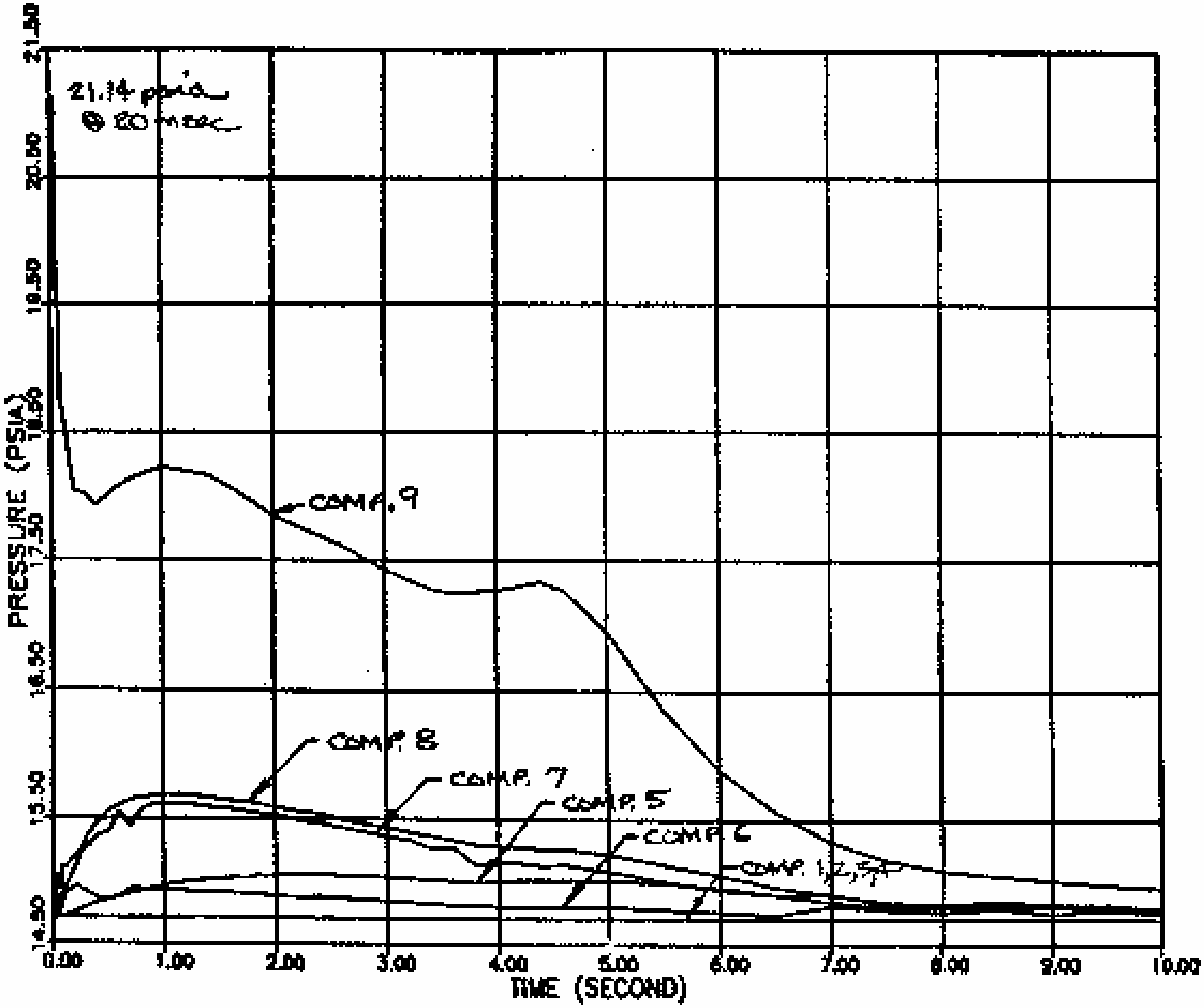


FSAR UPDATE

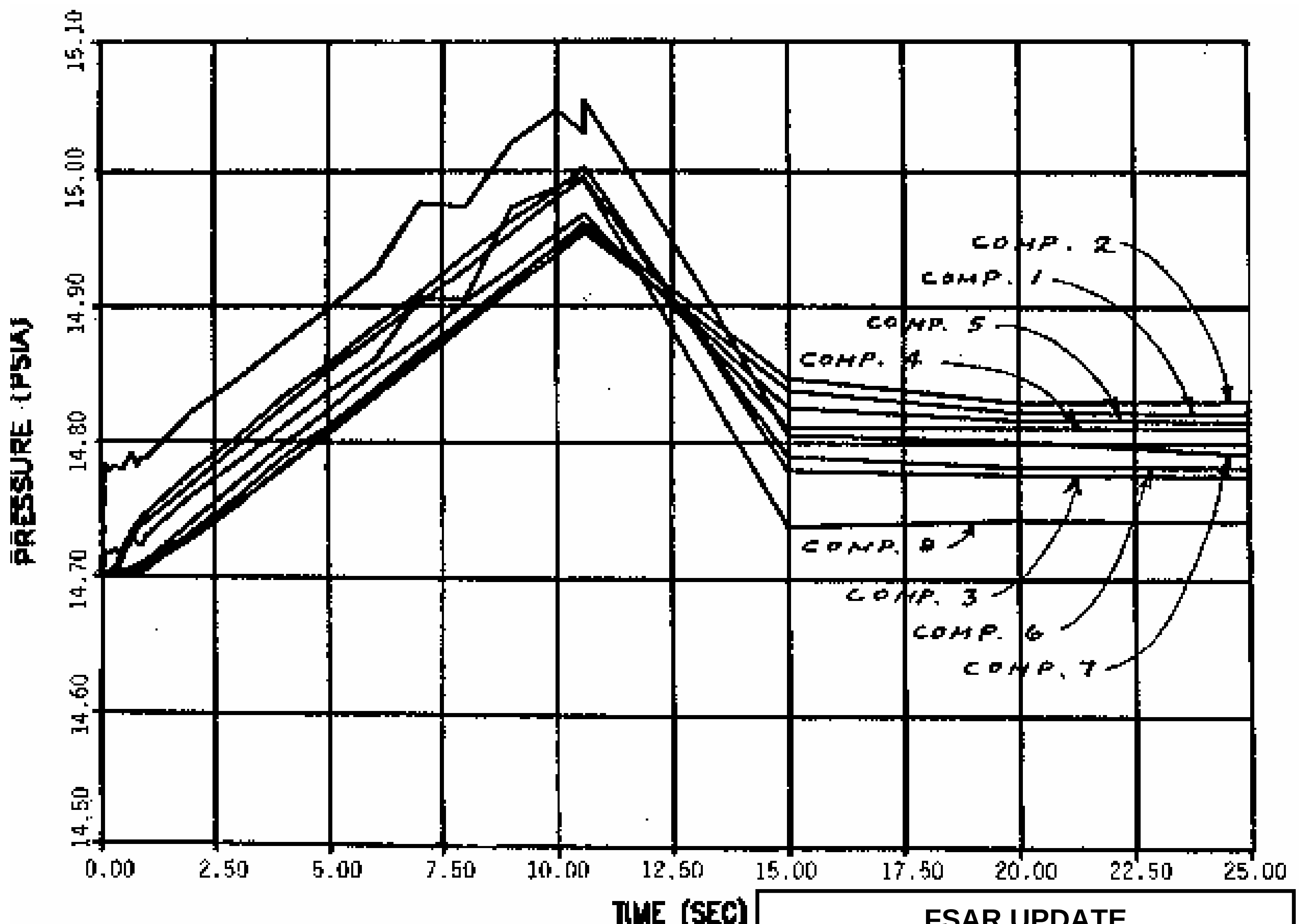
UNIT 1
DIABLO CANYON SITE

FIGURE 3.6-18

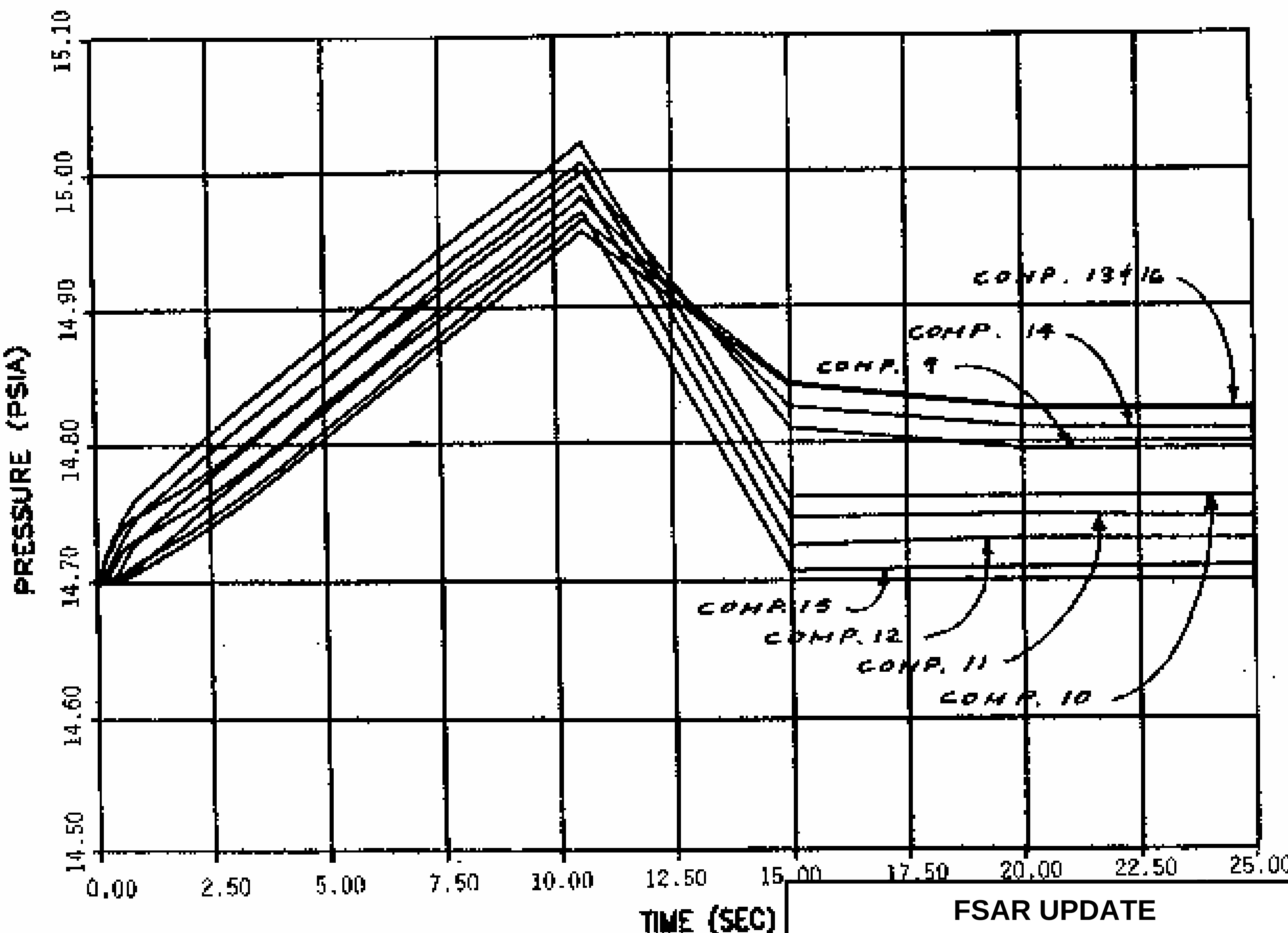
AREA GE/GW
PRESSURE vs. TIME RESPONSE
MSLB AT ELEVATION 115'
(DER) STEAM WITH ENTRAINMENT



FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-19 AREA GE/GW PRESSURE vs. TIME RESPONSE MSLB AT G-ROW ANCHOR (DER) STEAM WITH ENTRAINMENT



FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-20 AREA H, K PRESSURE vs. TIME RESPONSE CVCS LETDOWN LINE IN AREA K AT ELEVATION 85' (DER)

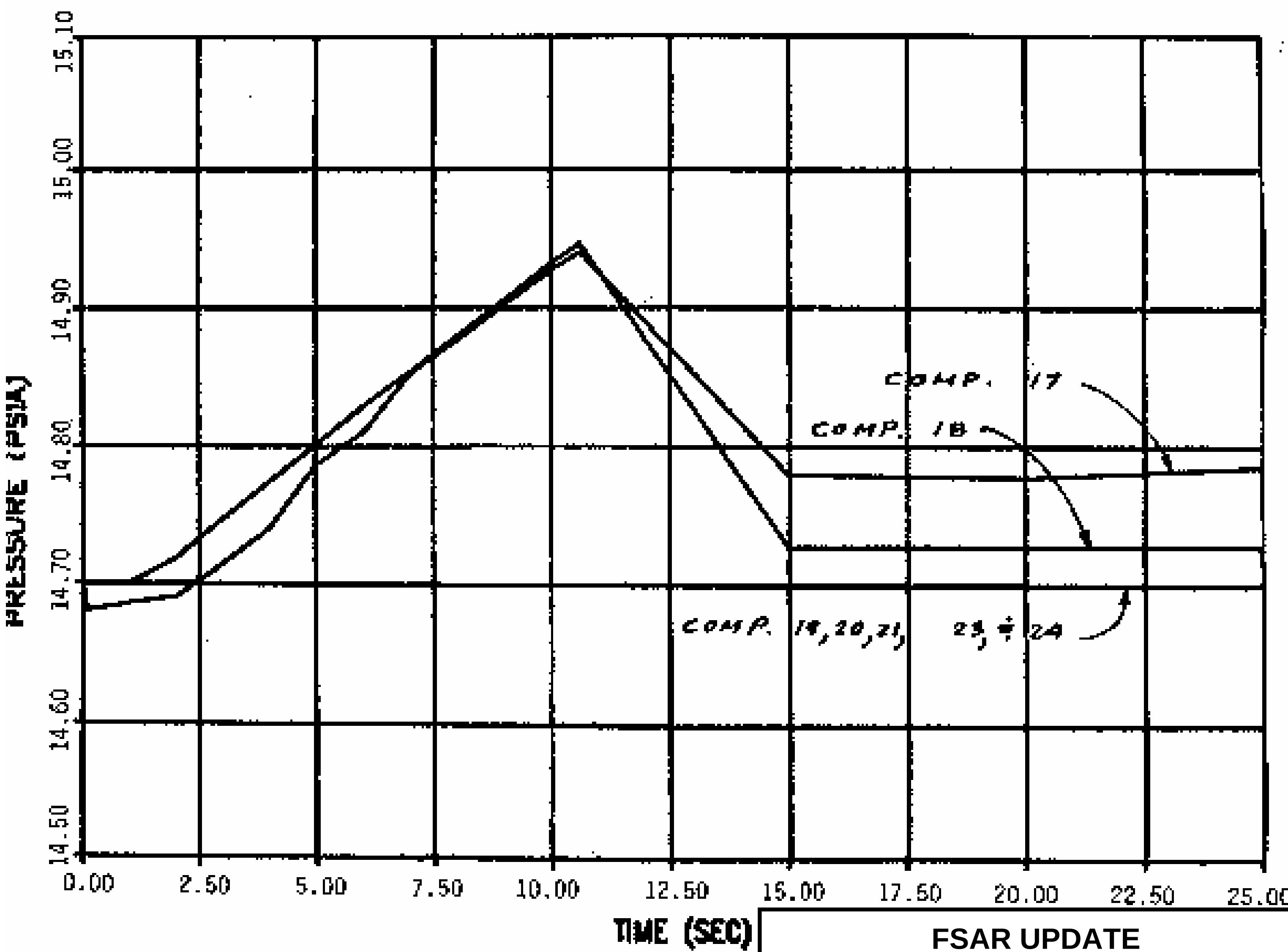


FSAR UPDATE

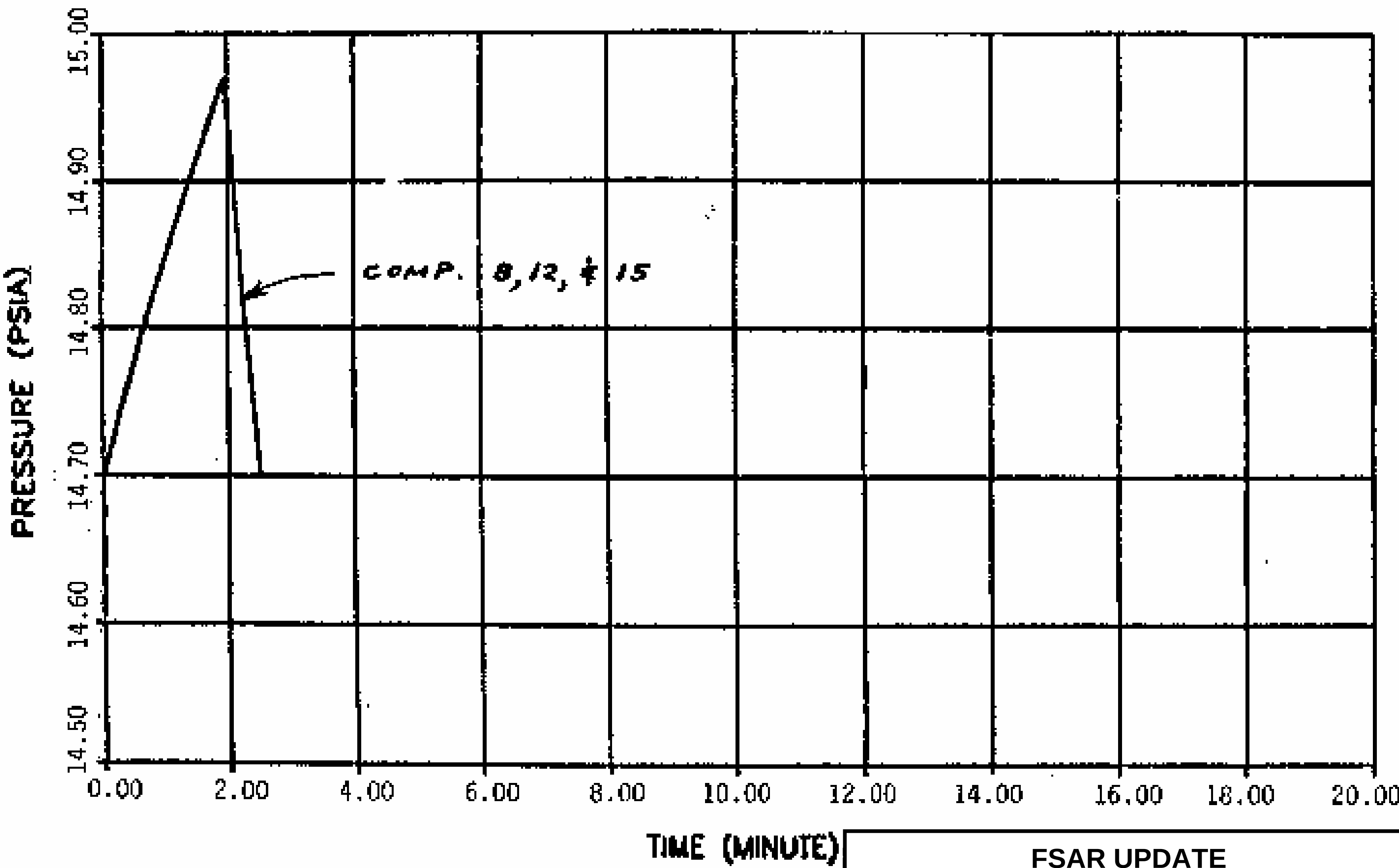
UNIT 1
DIABLO CANYON SITE

FIGURE 3.6-21
AREA H, K
PRESSURE vs. TIME RESPONSE
CVCS LETDOWN LINE IN AREA K
AT ELEVATION 85' (DER)

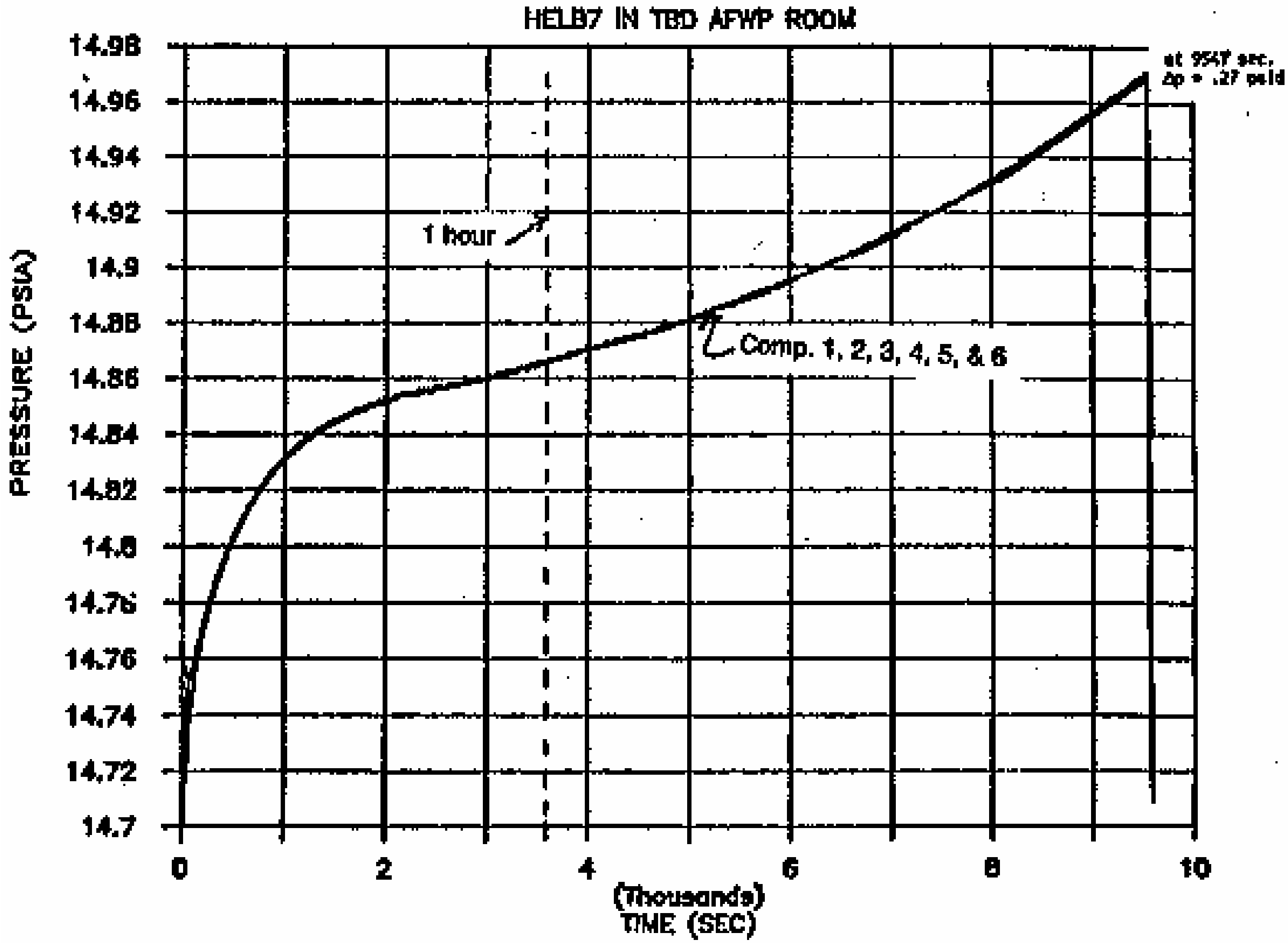
Revision 11 November 1996



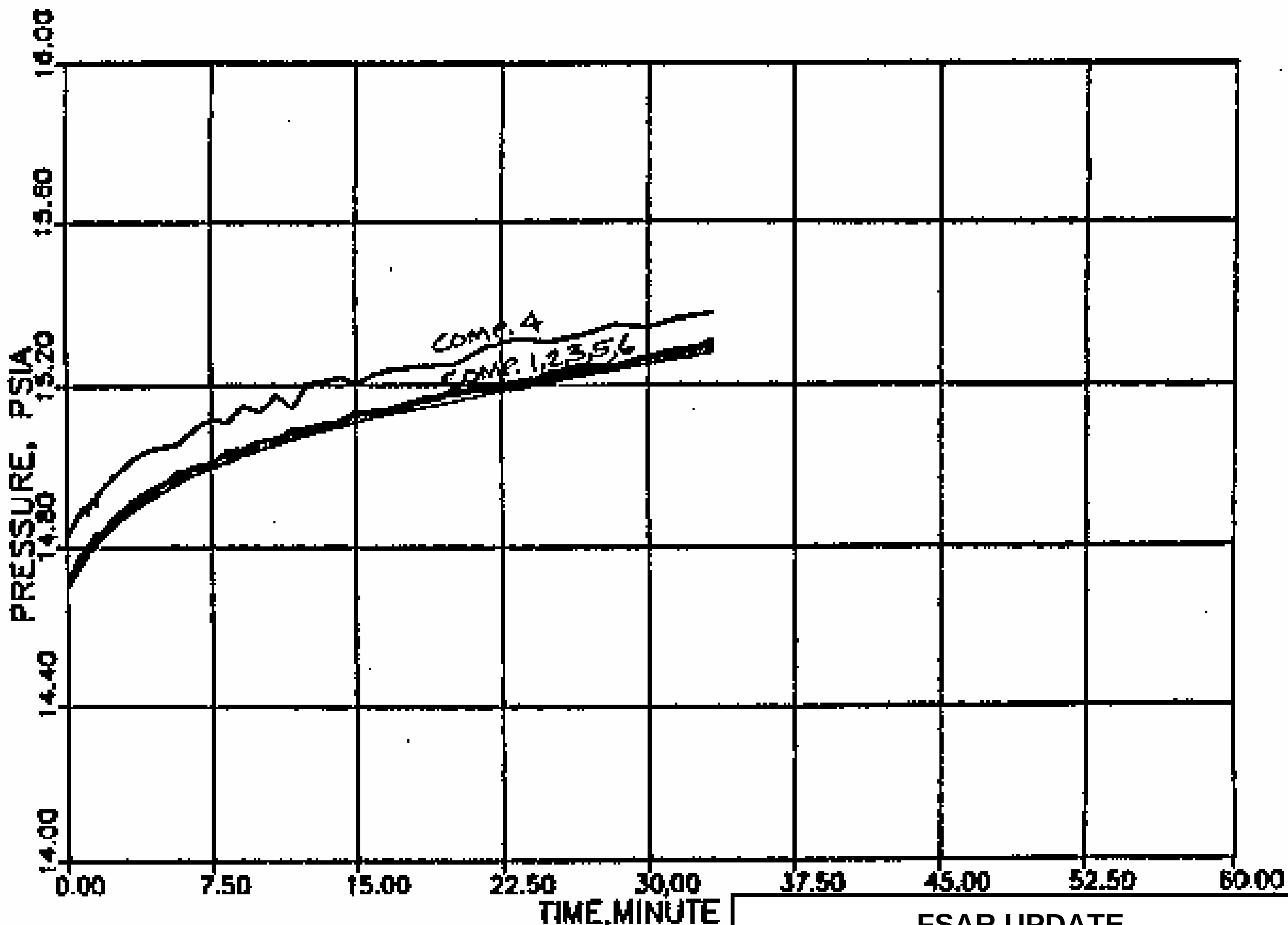
FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-22 AREA H, K PRESSURE vs. TIME RESPONSE CVCS LETDOWN LINE IN AREA K AT ELEVATION 85' (DER)



FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-23 AREA H, K PRESSURE vs. TIME RESPONSE AUXILIARY STEAM LINE IN AREA K AT ELEVATION 100' (CRACK BREAK)



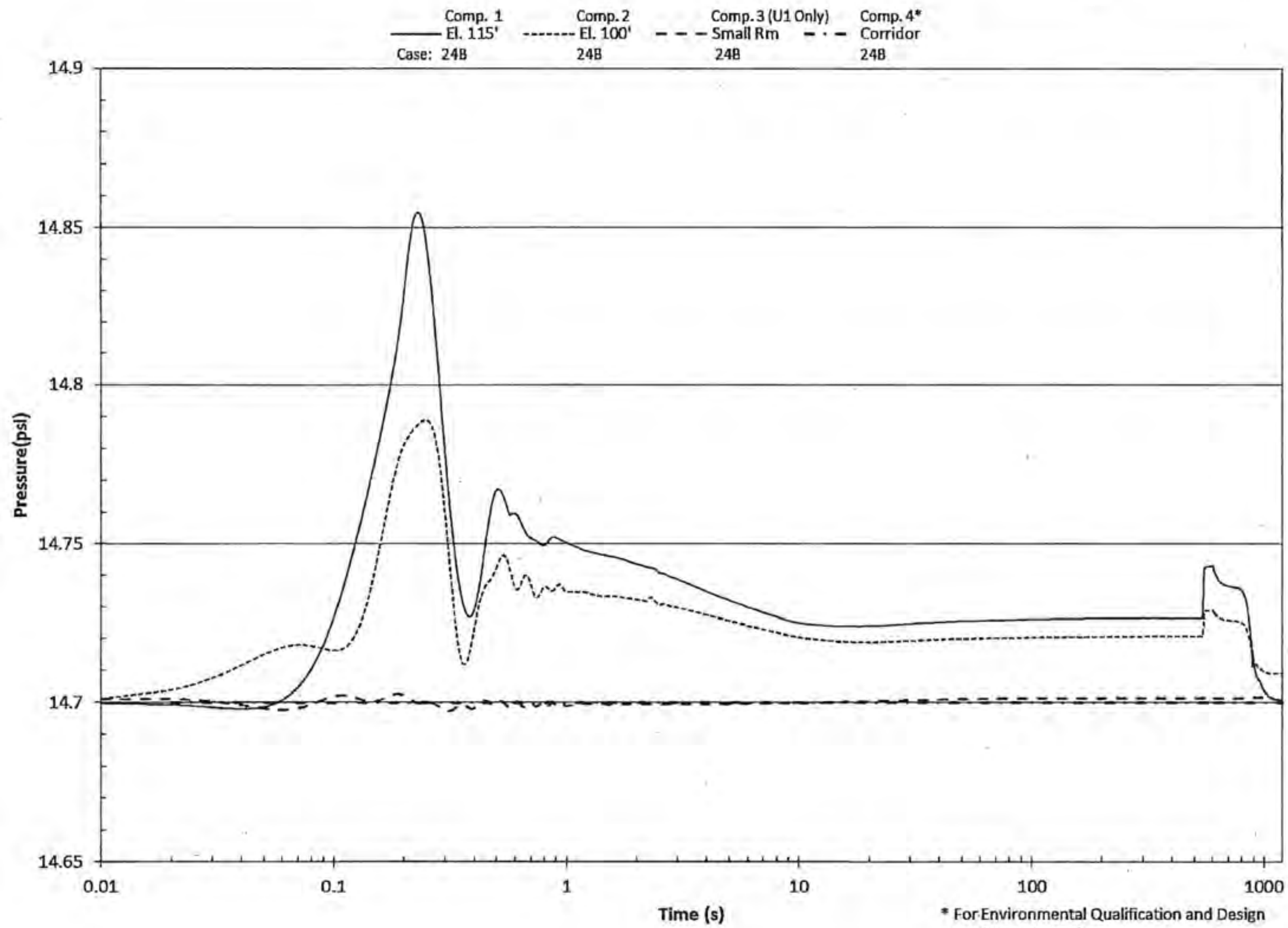
FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-24 AREA J PRESSURE vs. TIME RESPONSE AUXILIARY STEAM LINE IN TURBINE DRIVEN AUXILIARY FW PUMP ROOM (CRACK BREAK)



FSAR UPDATE

UNIT 1
DIABLO CANYON SITE

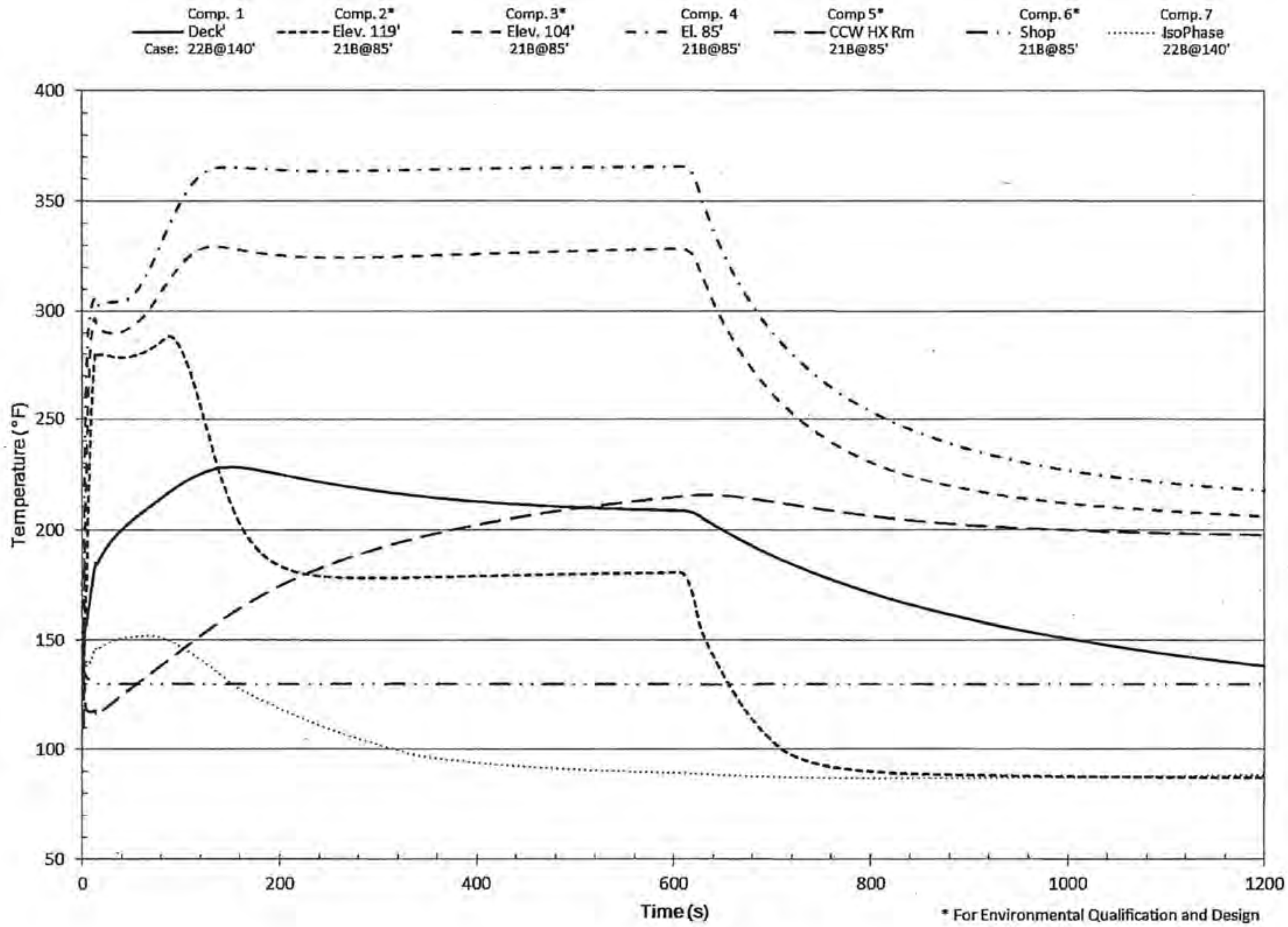
FIGURE 3.6-25
AREA J
PRESSURE vs. TIME RESPONSE
AUXILIARY STEAM LINE IN MOTOR DRIVEN
AUXILIARY FW PUMP ROOM, (CRACK BREAK)



* For Environmental Qualification and Design use Comp. 1 results.

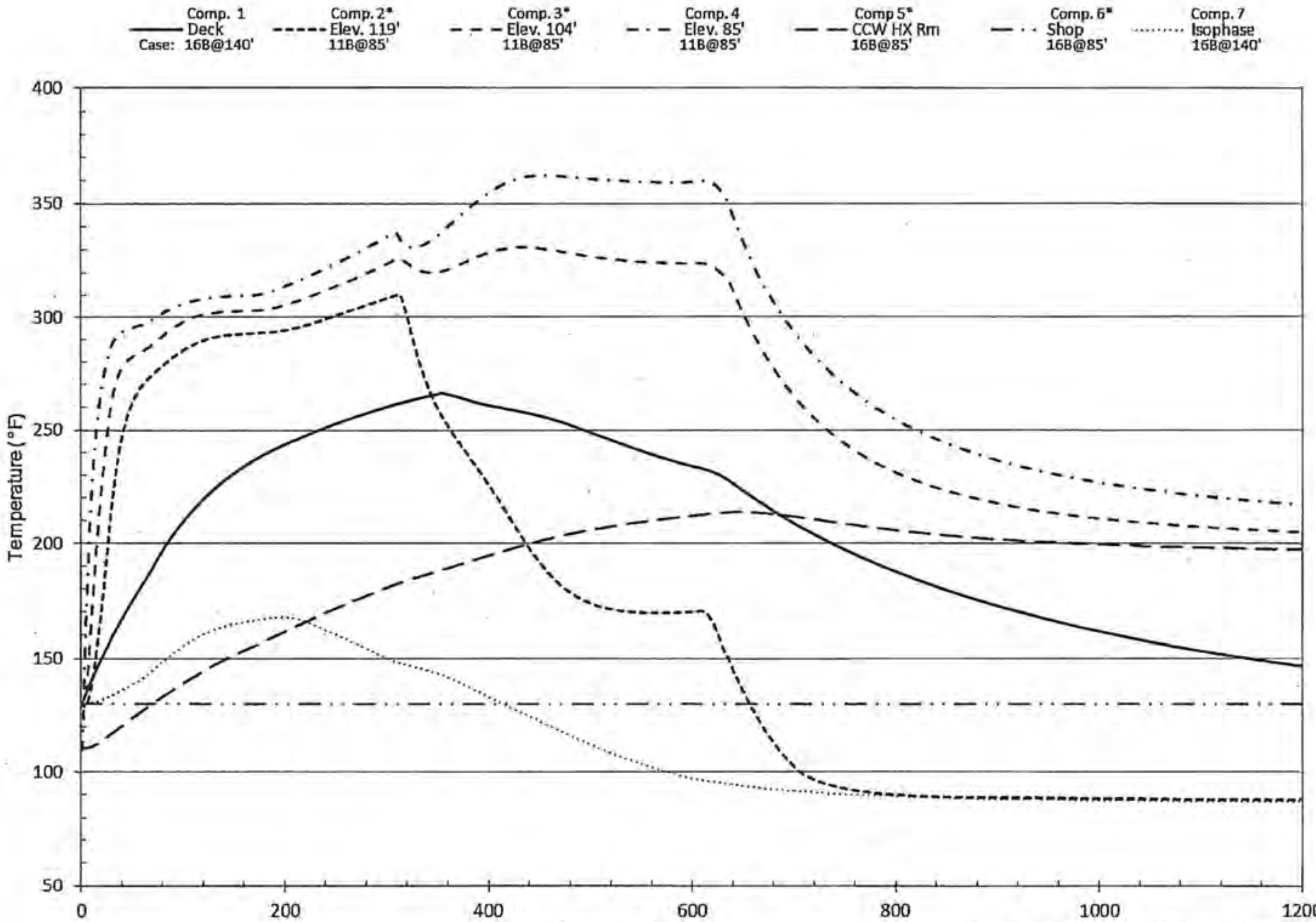
FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-26 AREA L PRESSURE vs. TIME RESPONSE AFW PUMP STEAM SUPPLY LINE AT ELEVATION 115' (DER)

Revision 22 May 2015



* For Environmental Qualification and Design use El. 85' results.

FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-28 TURBINE BUILDING TEMPERATURE vs. TIME RESPONSE MSLB (DER)



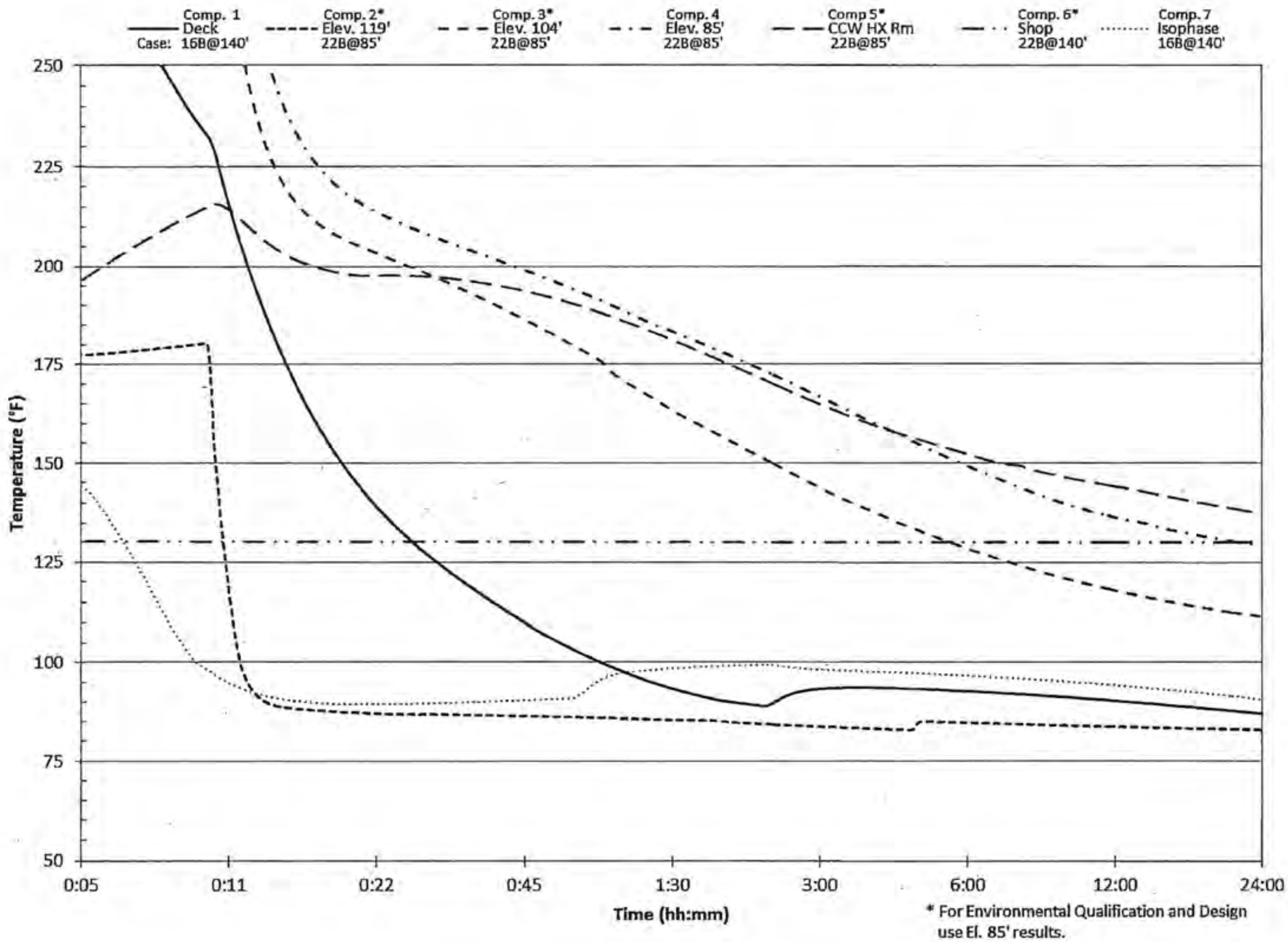
* For Environmental Qualification and Design use El. 85' results.

FSAR UPDATE

UNIT 1
DIABLO CANYON SITE

FIGURE 3.6-29
TURBINE BUILDING TEMPERATURE vs. TIME RESPONSE
MSLB (SPLIT RUPTURE)

Revision 22 May 2015

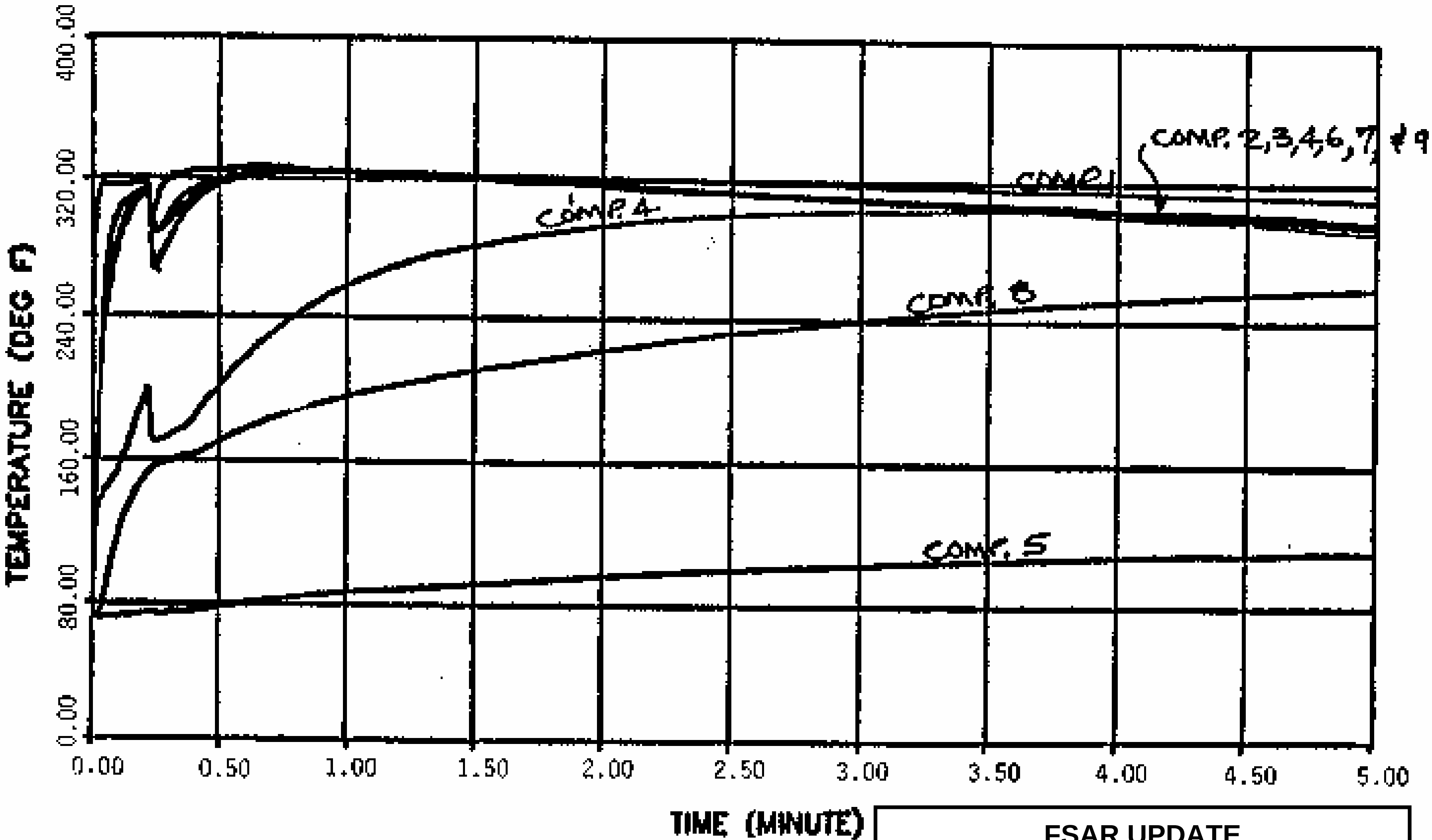


FSAR UPDATE

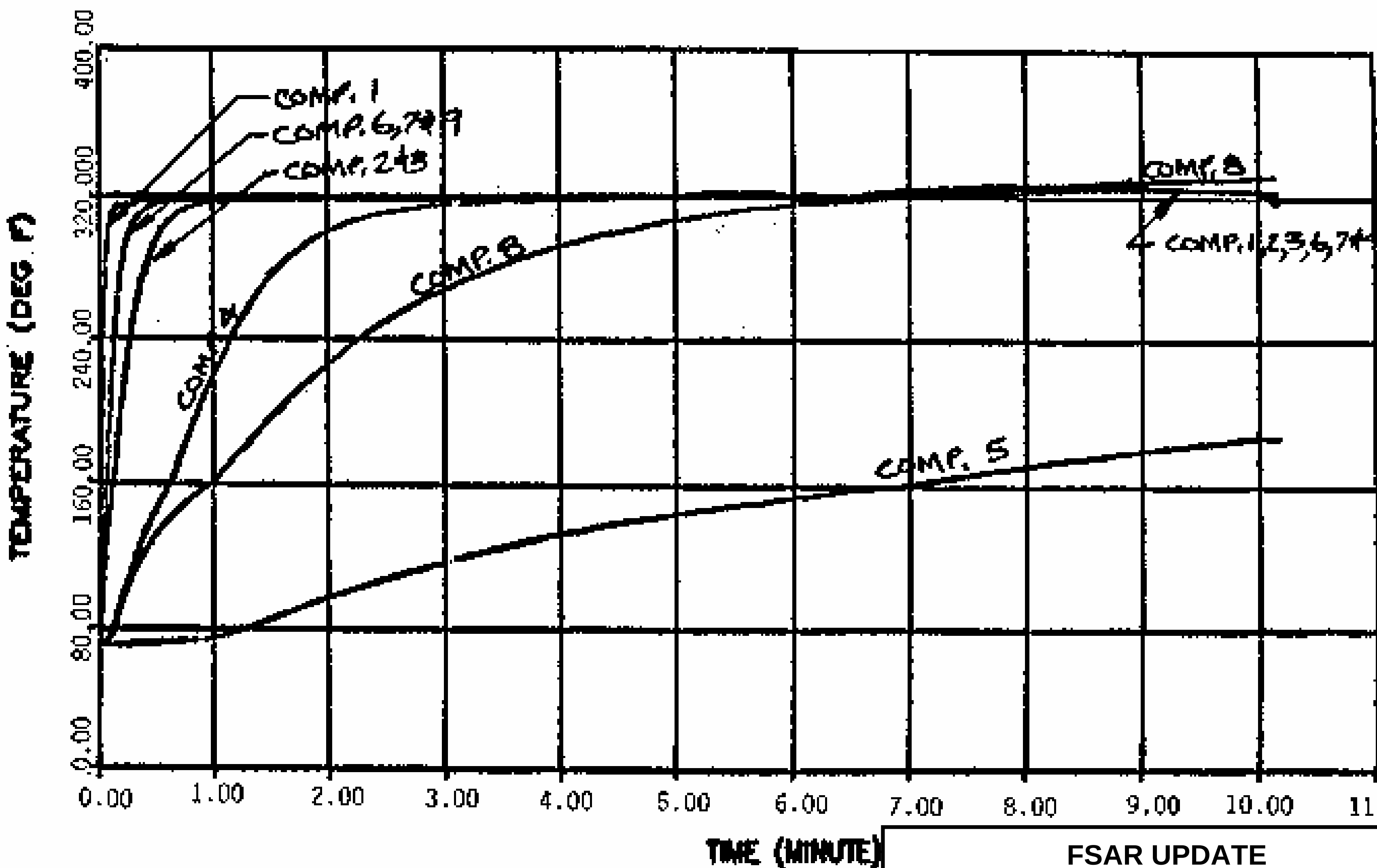
UNIT 1
DIABLO CANYON SITE

FIGURE 3.6-30
TURBINE BUILDING LONG TERM
TEMPERATURE vs. TIME RESPONSE

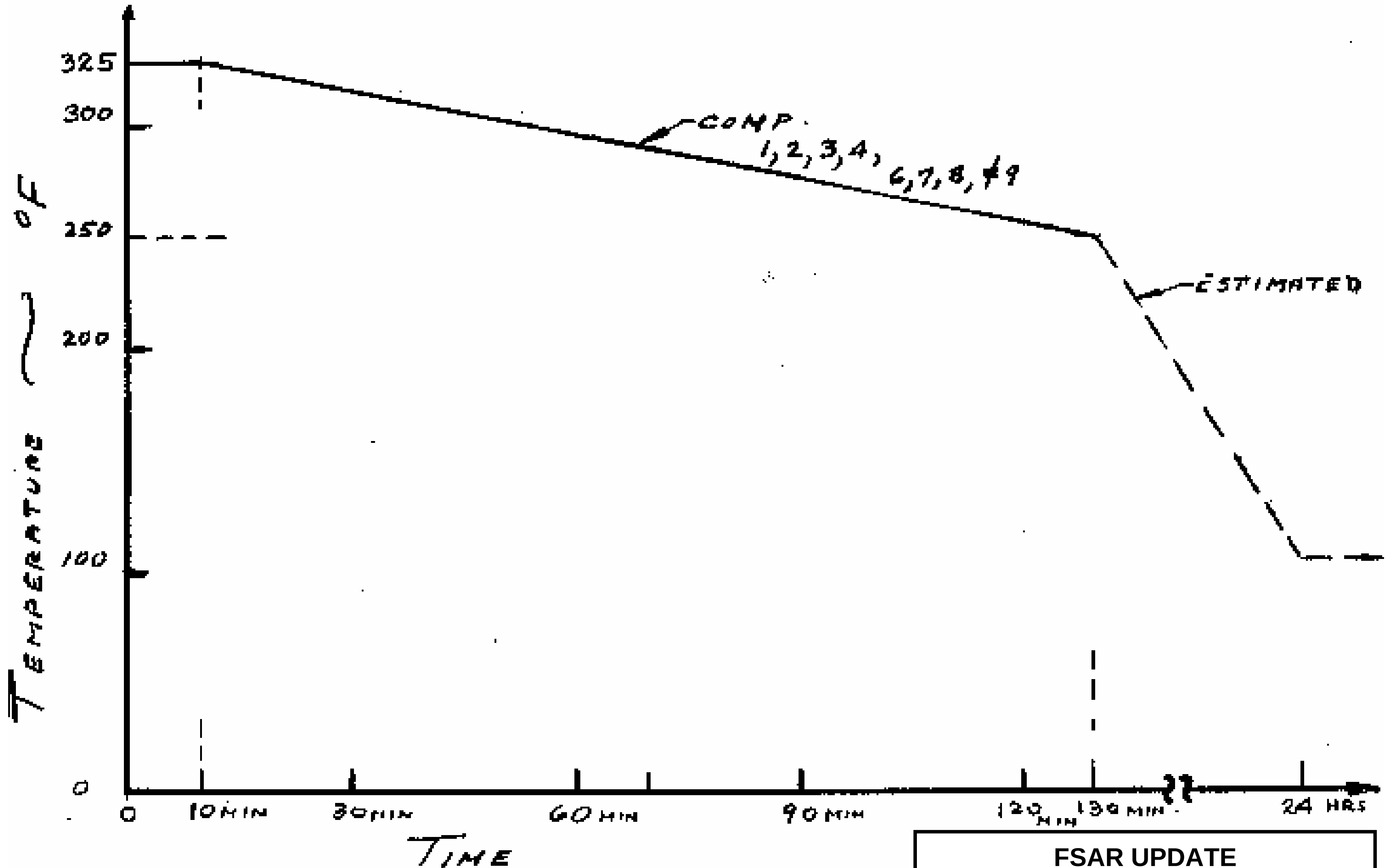
Revision 22 May 2015



FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-31 AREA GE/GW TEMPERATURE vs. TIME RESPONSE MSLB AT ELEVATION 115' (DER), STEAM WITHOUT ENTRAINMENT



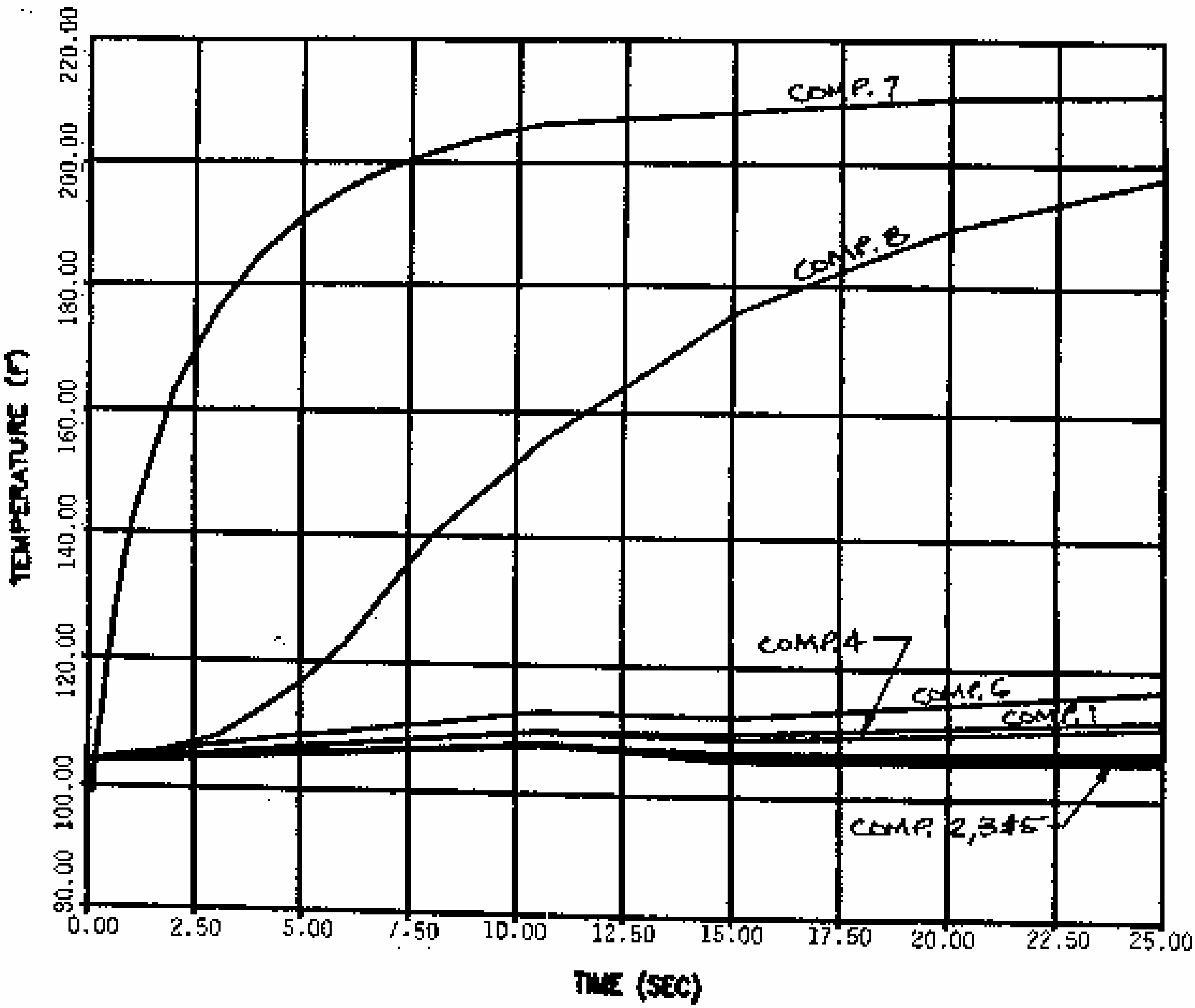
FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-32 AREA GE/GW TEMPERATURE vs. TIME RESPONSE MSLB AT ELEVATION 115' (PLIT RUPTURE)



FSAR UPDATE

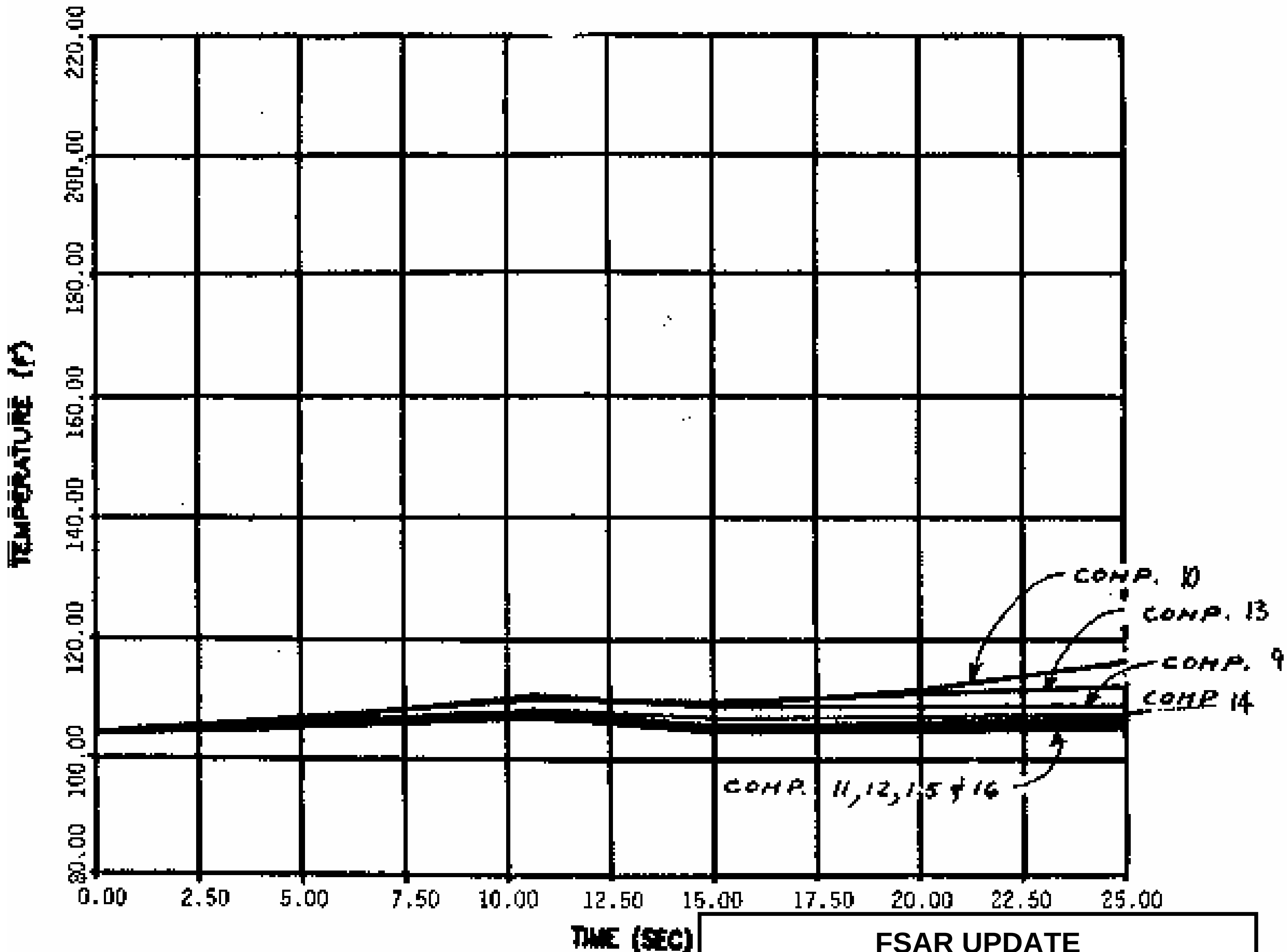
UNIT 1
DIABLO CANYON SITE

FIGURE 3.6-33
AREA GE/GW
LONG TERM TEMPERATURE vs.
TIME RESPONSE

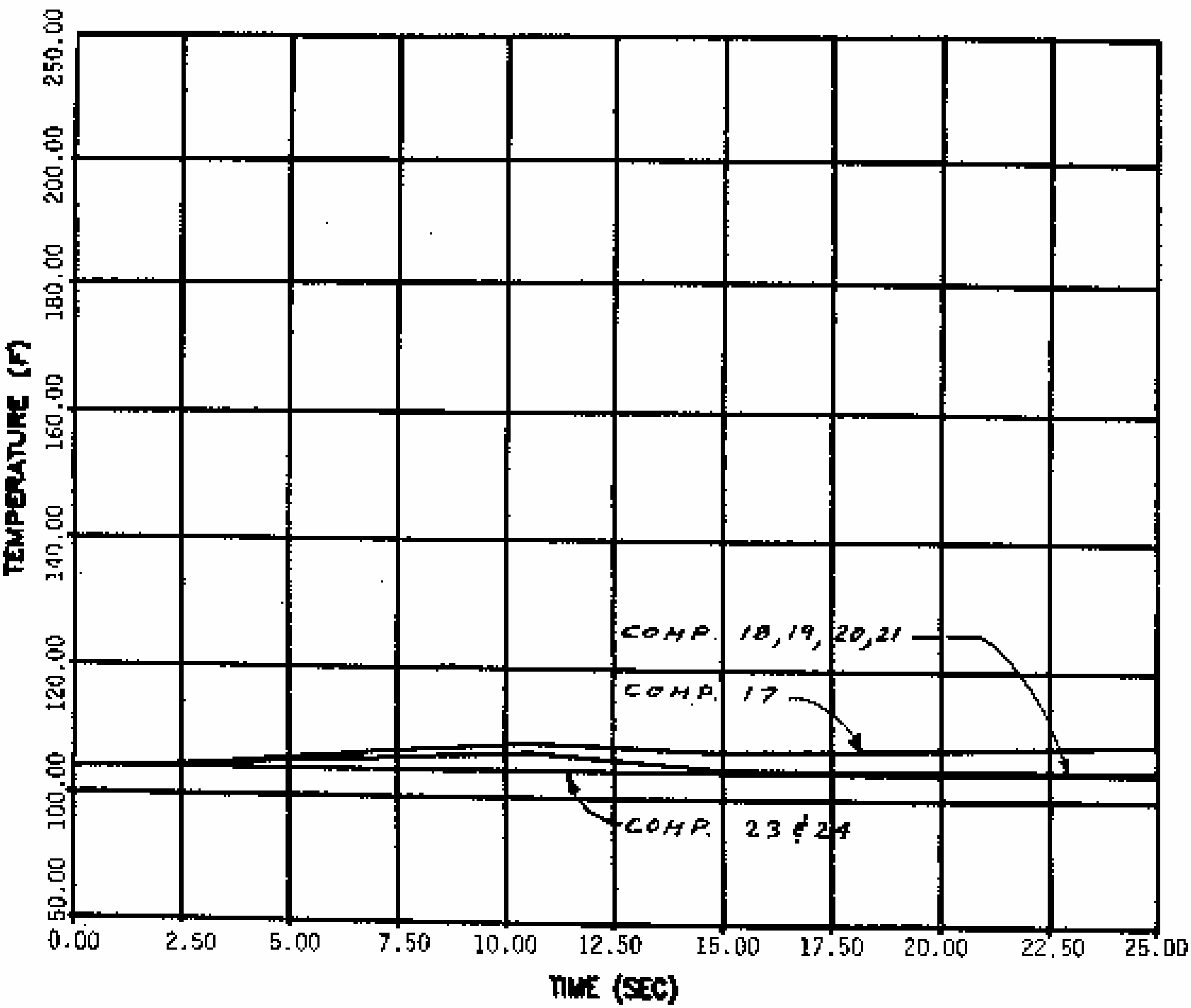


FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-34 AREA H, K TEMPERATURE vs. TIME RESPONSE CVCS LETDOWN LINE BREAK IN AREA K AT ELEVATION 85', (DER)

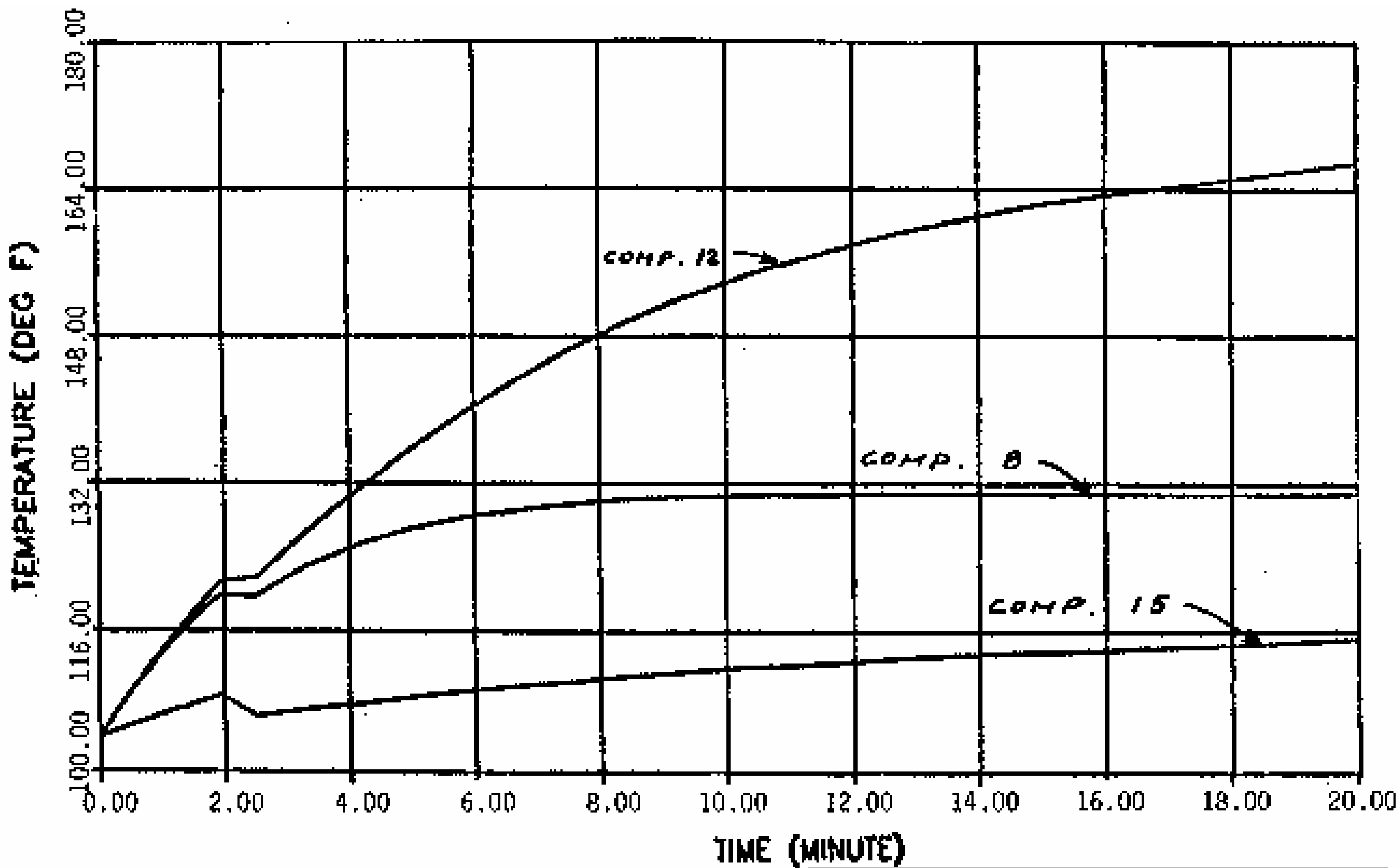
Revision 11 November 1996



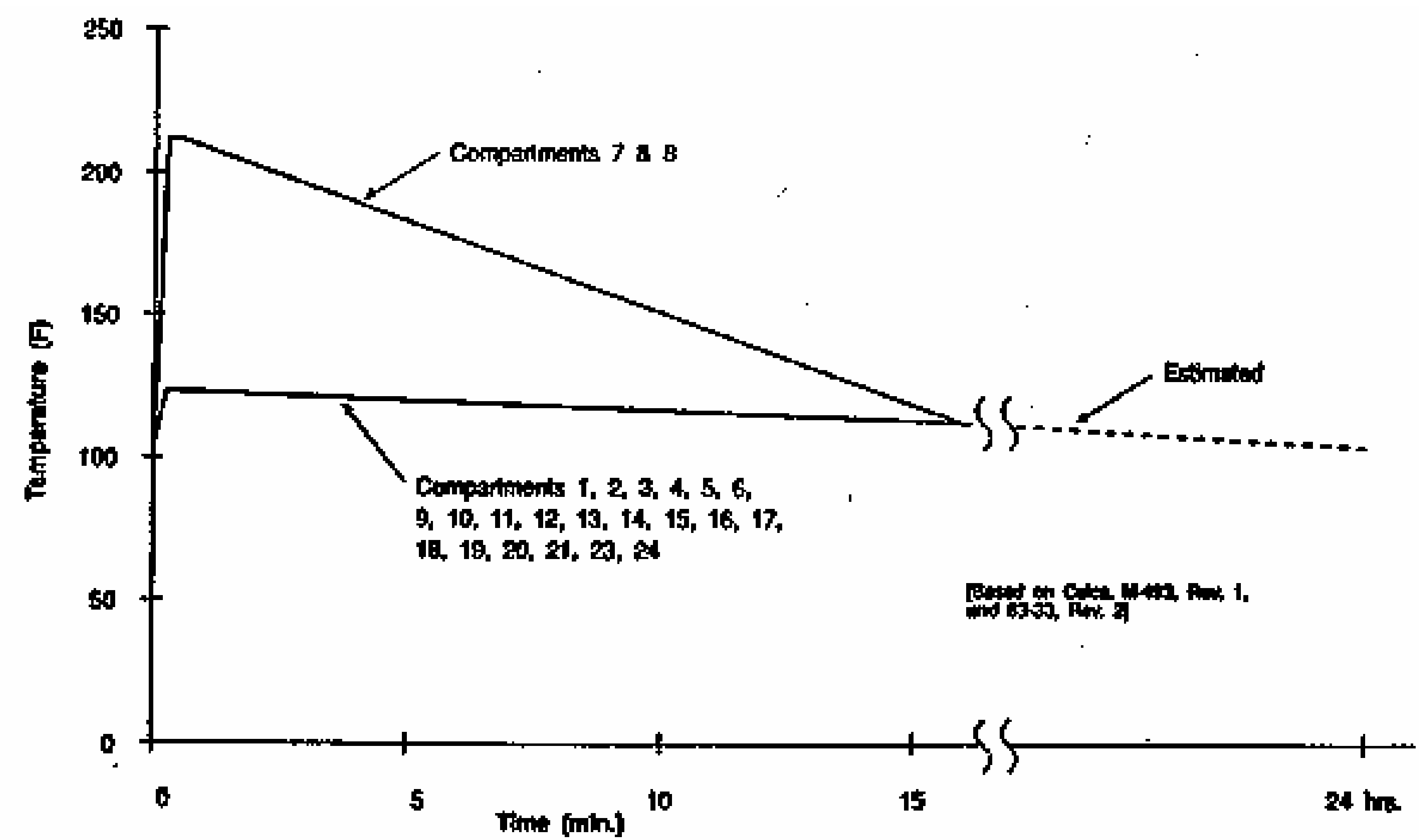
FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-35 AREA H, K TEMPERATURE vs. TIME RESPONSE CVCS LETDOWN LINE IN AREA K AT ELEVATION 85', (DER)



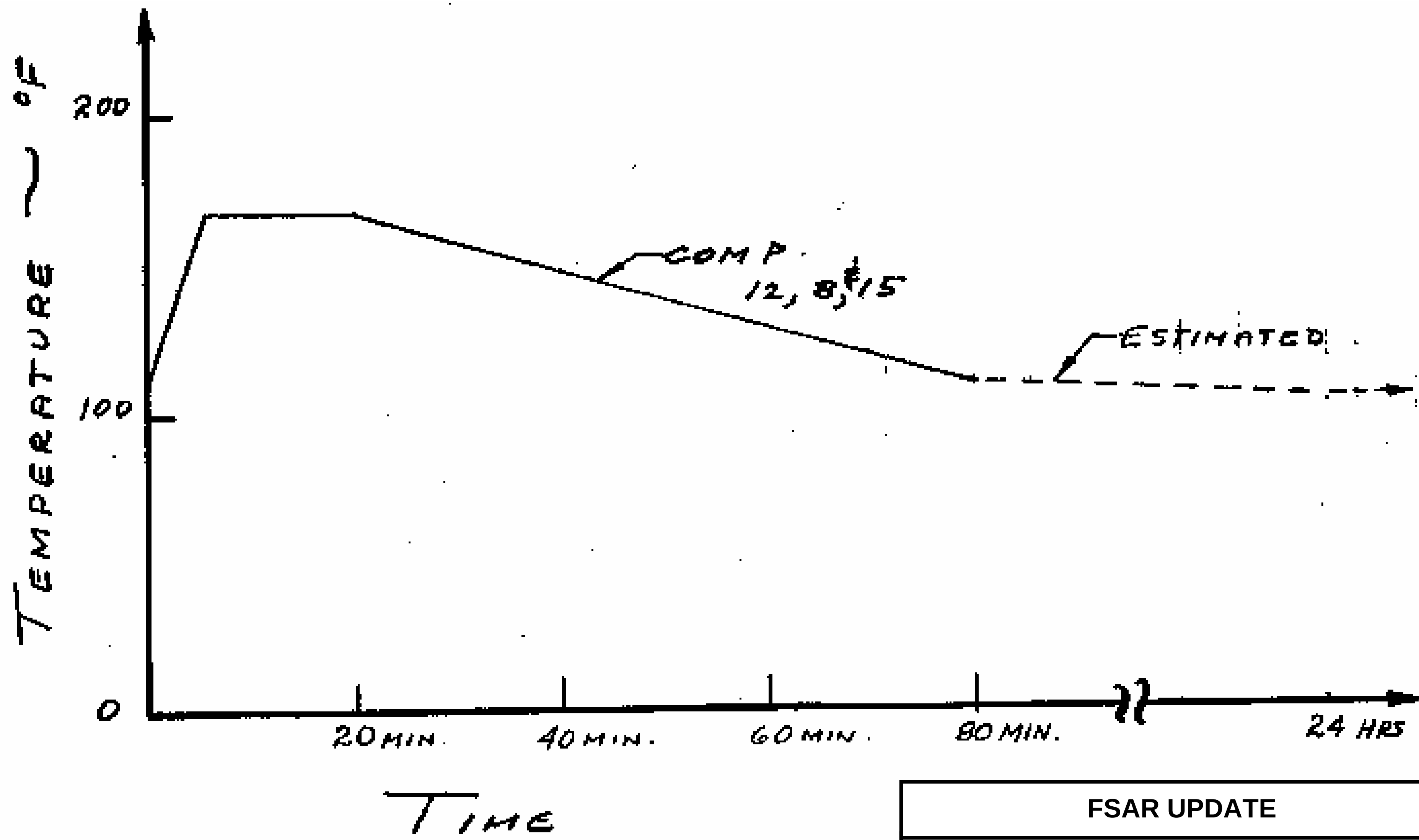
FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-36 **AREA H, K** **TEMPERATURE vs. TIME RESPONSE** **CVCS LETDOWN LINE IN** **AREA K AT ELEVATION 85' (DER)**



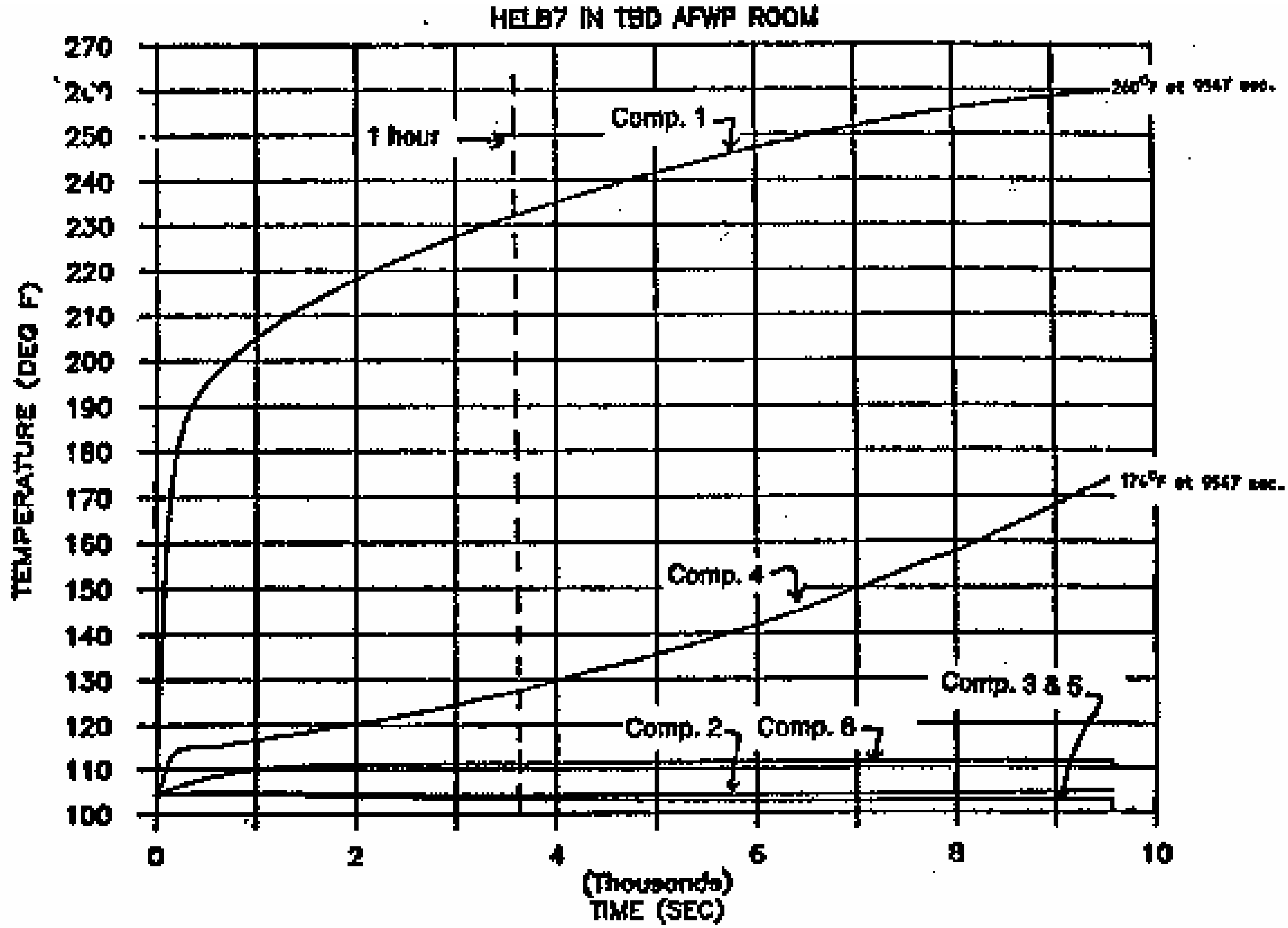
FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-37 AREA H, K TEMPERATURE vs. TIME RESP0NSE AUXILIARY STEAM LINE IN AREA K AT ELEVATION 100' (CRACK BREAK)



FSAR UPDATE
UNIT 1 **DIABLO CANYON SITE**
FIGURE 3.6-38 AREA H, K LONG TERM TEMPERATURE vs. TIME RESPONSE CVCS LETDOWN LINE IN AREA K AT ELEVATION 85 FT (DER)



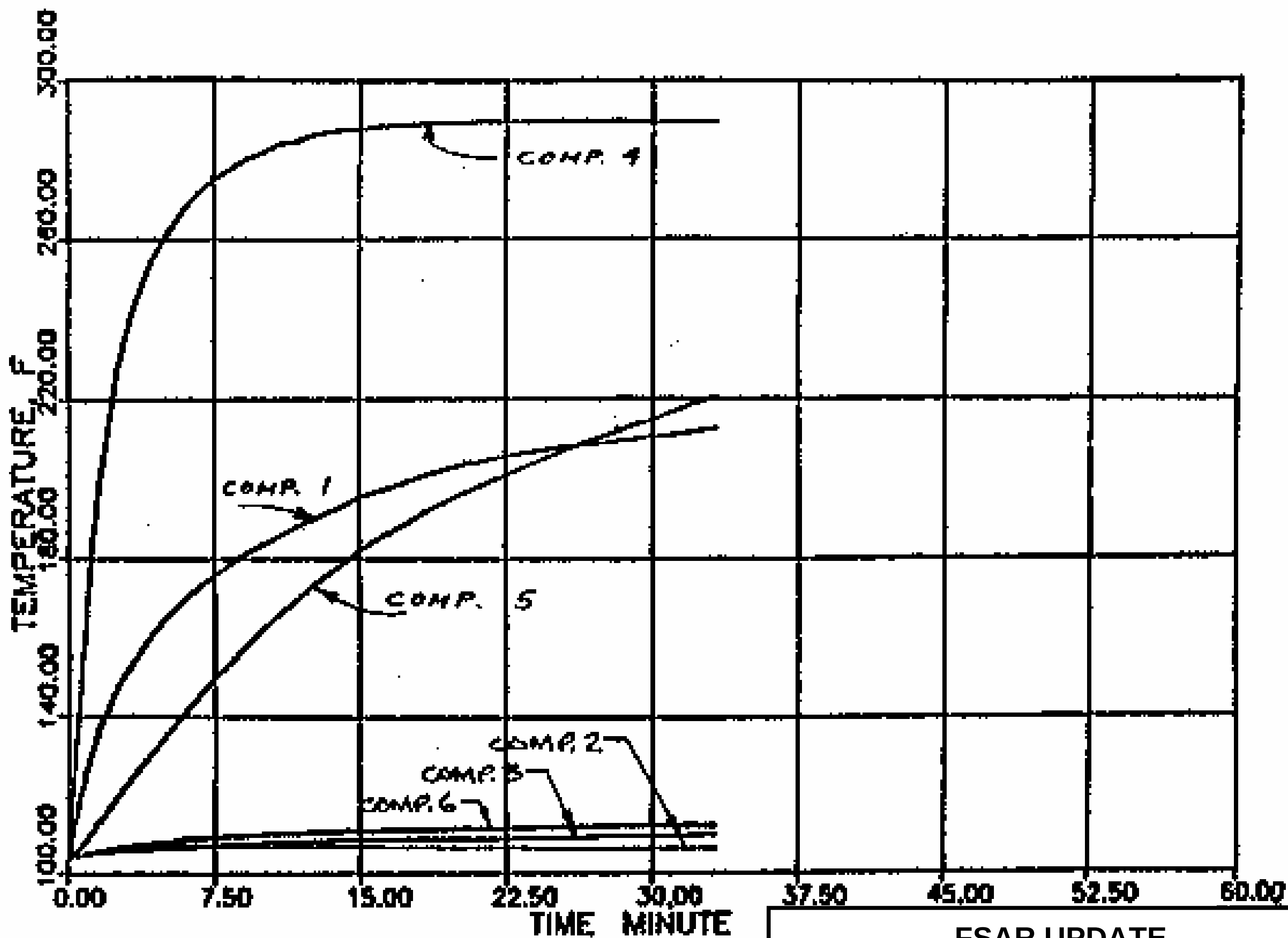
FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-39 AREA H, K LONG TERM TEMPERATURE vs. TIME RESP0NSE AUXILIARY STEAM LINE IN AREA K AT ELEVATION 100' (CRACK BREAK)



FSAR UPDATE

UNIT 1 DIABLO CANYON SITE

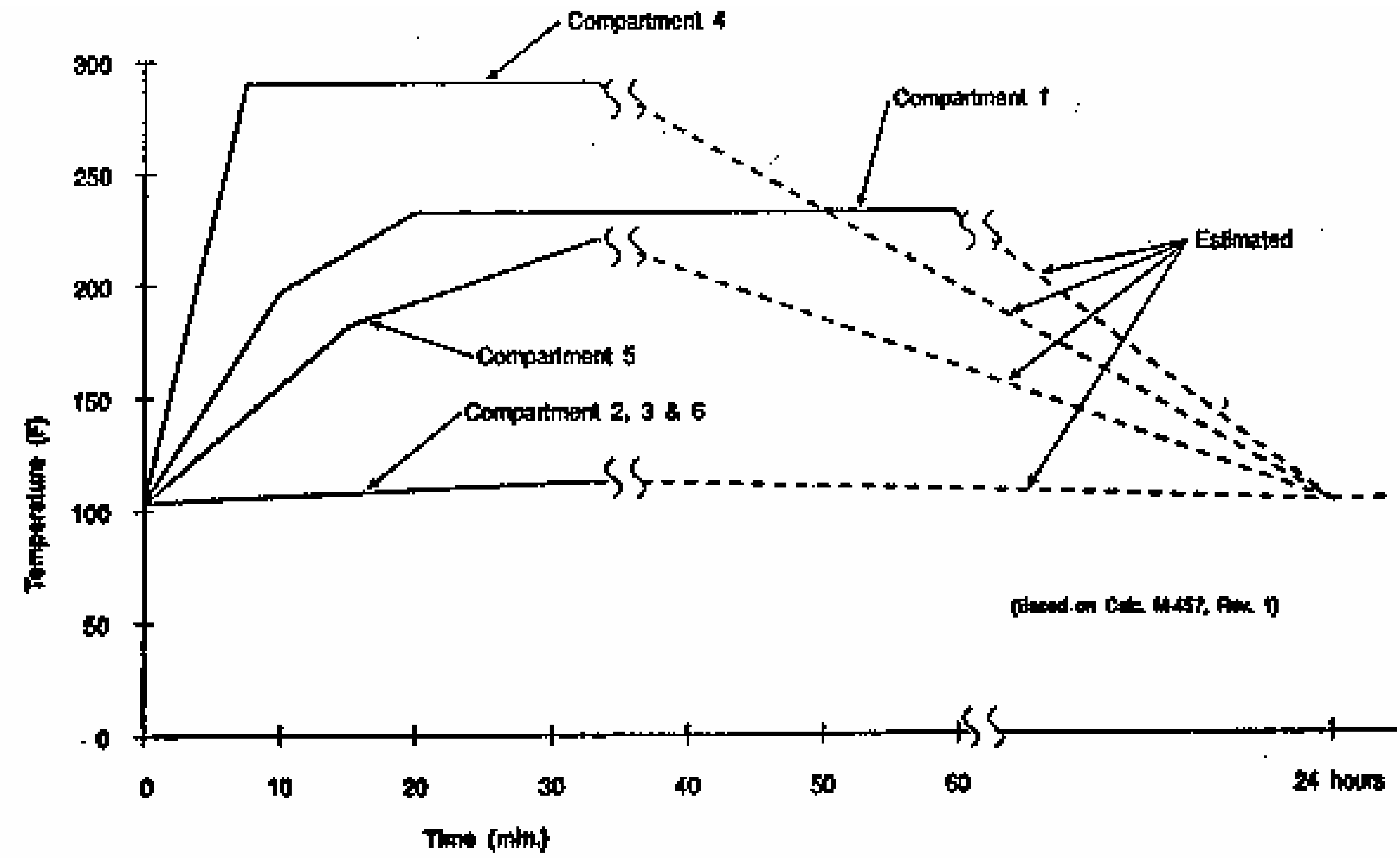
FIGURE 3.6-40
AREA J
TEMPERATURE vs. TIME RESPONSE
AUXILIARY STEAM LINE IN TURBINE DRIVEN
AUXILIARY FW PUMP ROOM (CRACK BREAK)



FSAR UPDATE

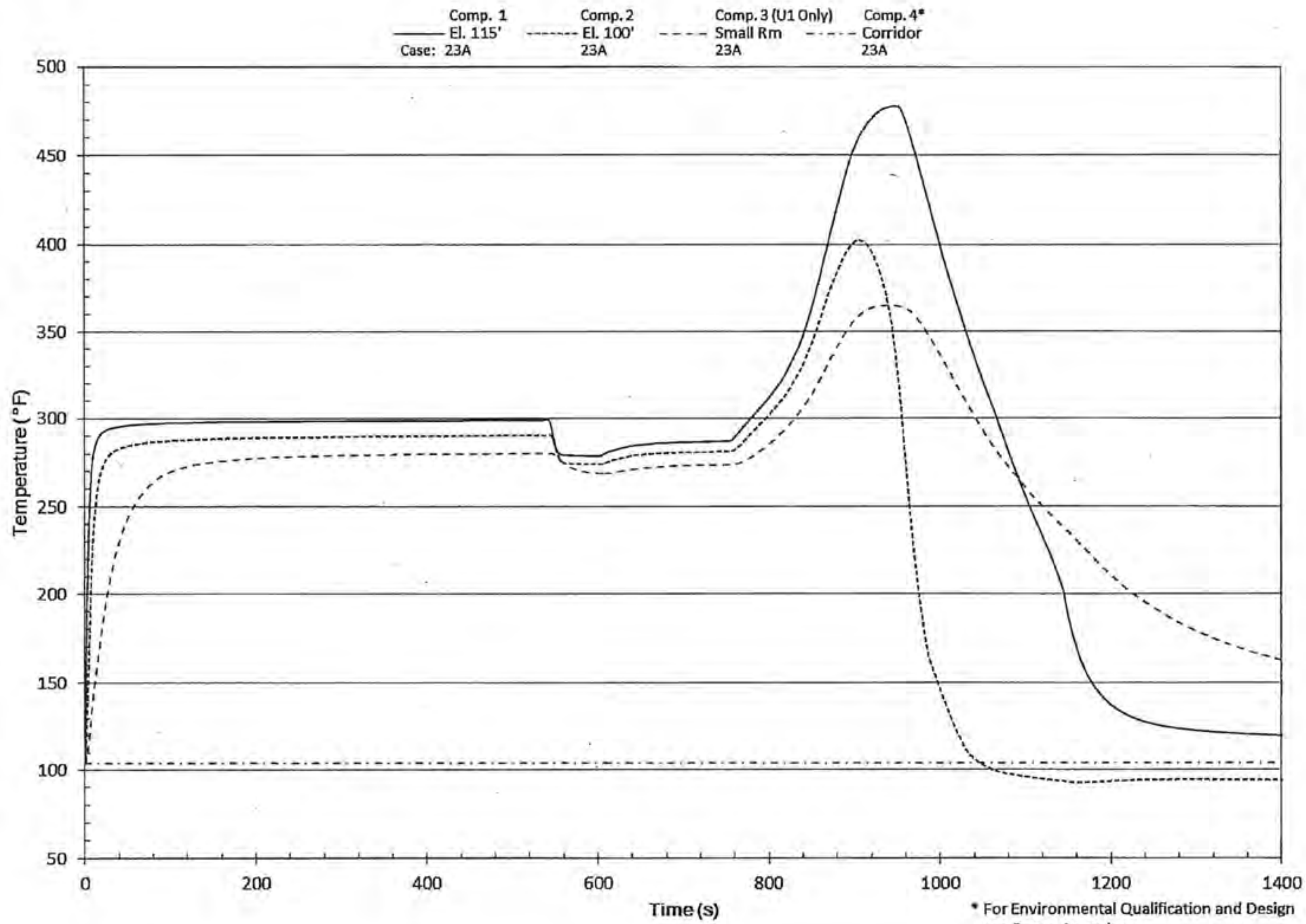
UNIT 1
DIABLO CANYON SITE

FIGURE 3.6-41
AREA J
TEMPERATURE vs. TIME RESP0NSE
AUXILIARY STEAM LINE IN MOTOR DRIVEN
AUXILIARY FW PUMP ROOM, (CRACK BREAK)



FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-42 AREA J LONG TERM TEMPERATURE vs. TIME RESPONSE

Revision 12 September 1998

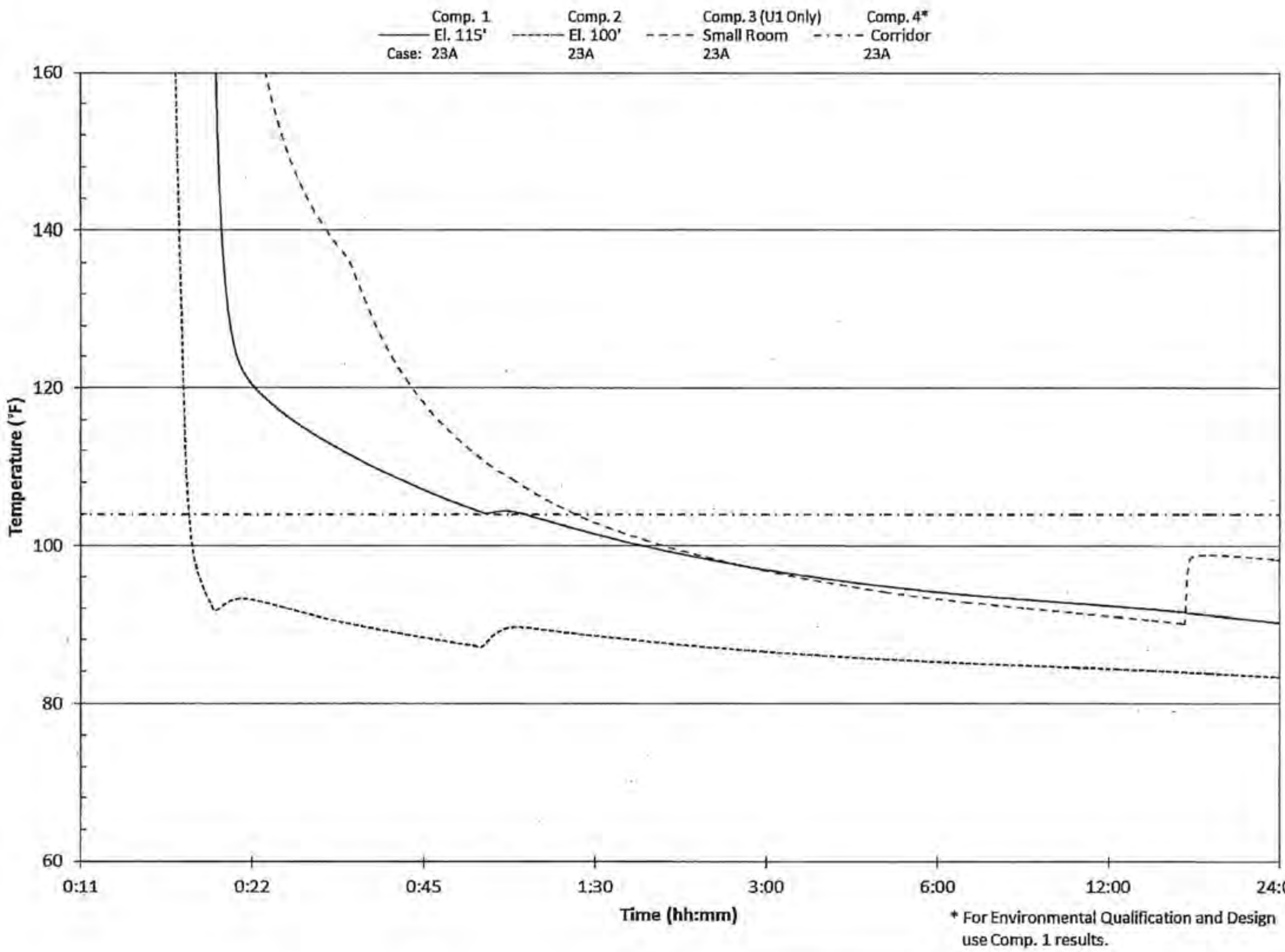


* For Environmental Qualification and Design use Comp. 1 results.

FSAR UPDATE

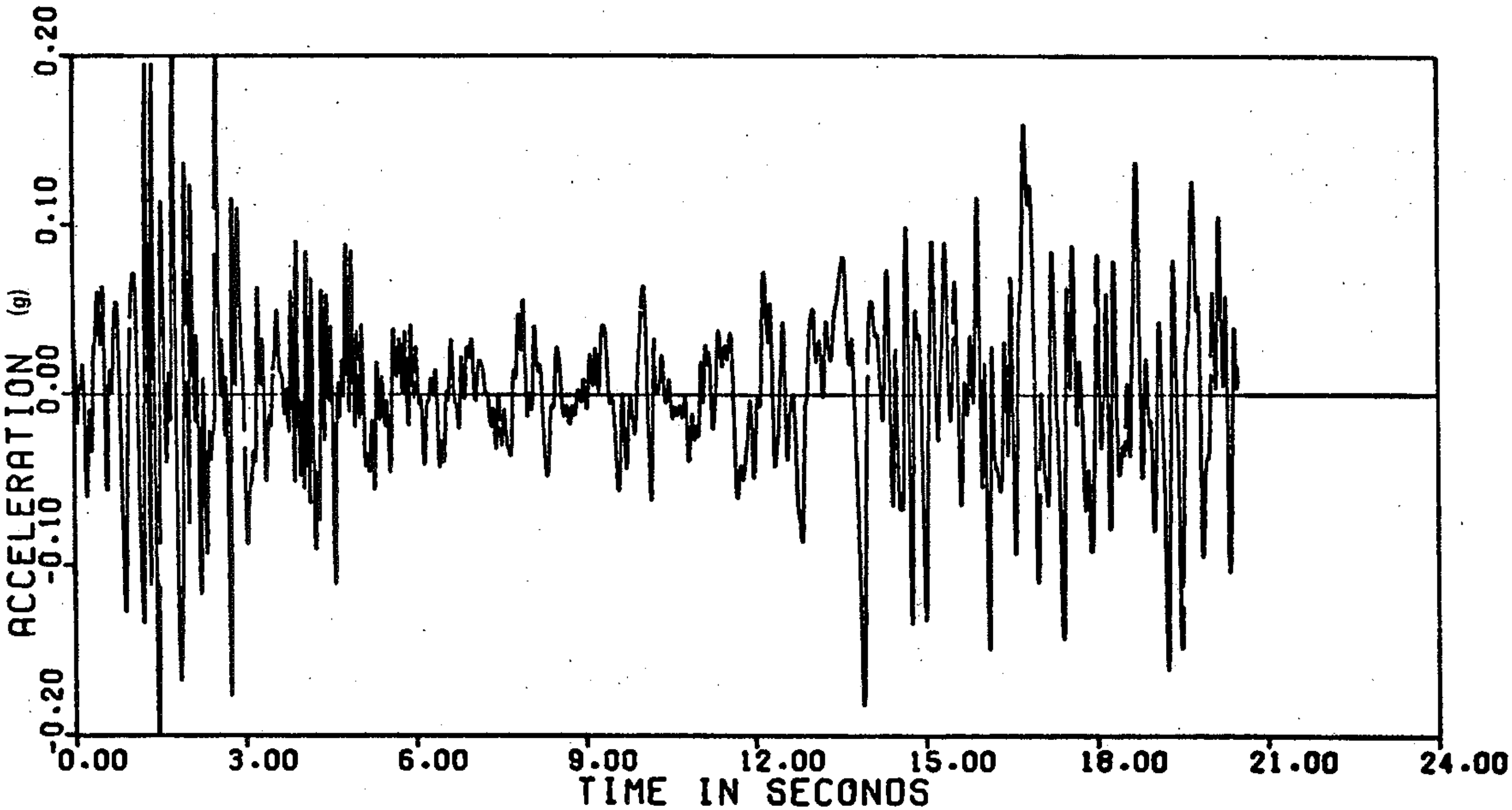
UNIT 1
DIABLO CANYON SITE

FIGURE 3.6-43
AREA L TEMPERATURE vs. TIME RESPONSE
AFW PUMP STEAM SUPPLY LINE AT ELEVATION 115' (DER)



FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.6-44 AREA L LONG TERM TEMPERATURE vs. TIME RESPONSE AFW PUMP STEAM SUPPLY LINE AT ELEVATION 115' (DER)

Revision 22 May 2015



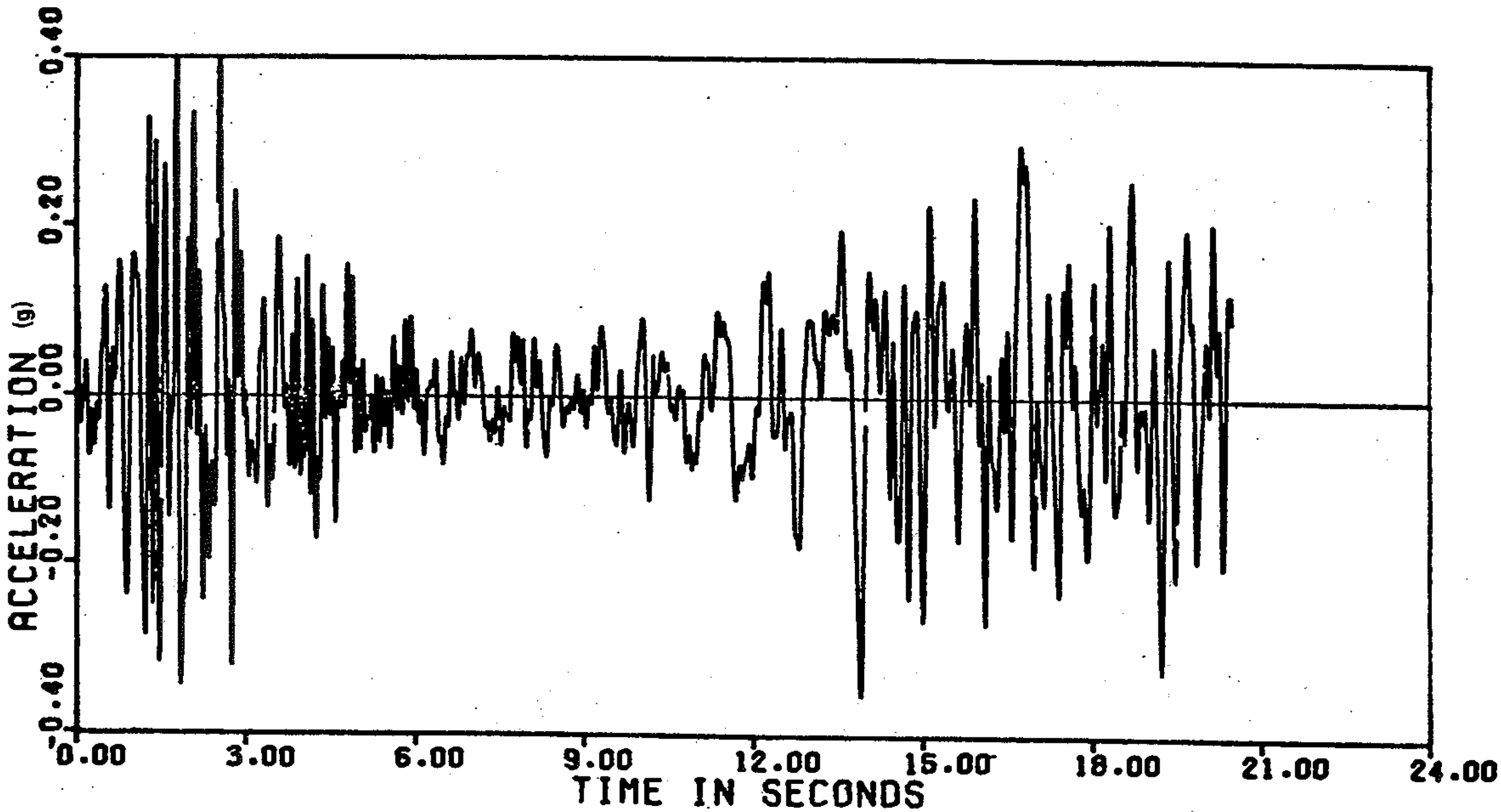
FSAR UPDATE

UNITS I AND 2
DIABLO CANYON SITE

FIGURE 3.7-1

FREE FIELD GROUND MOTION

DE ANALYSIS



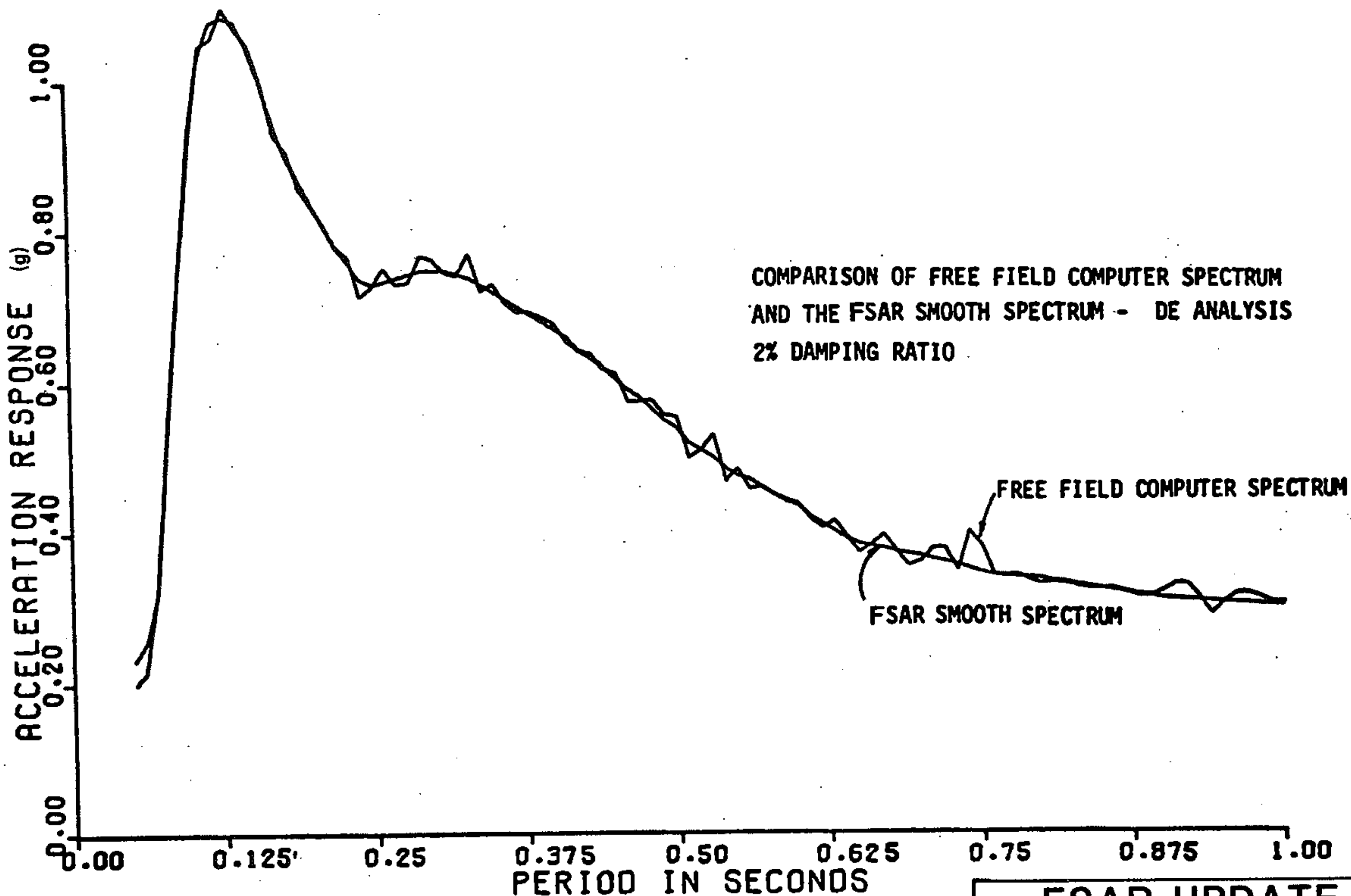
FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 3.7-2

FREE FIELD GROUND MOTION

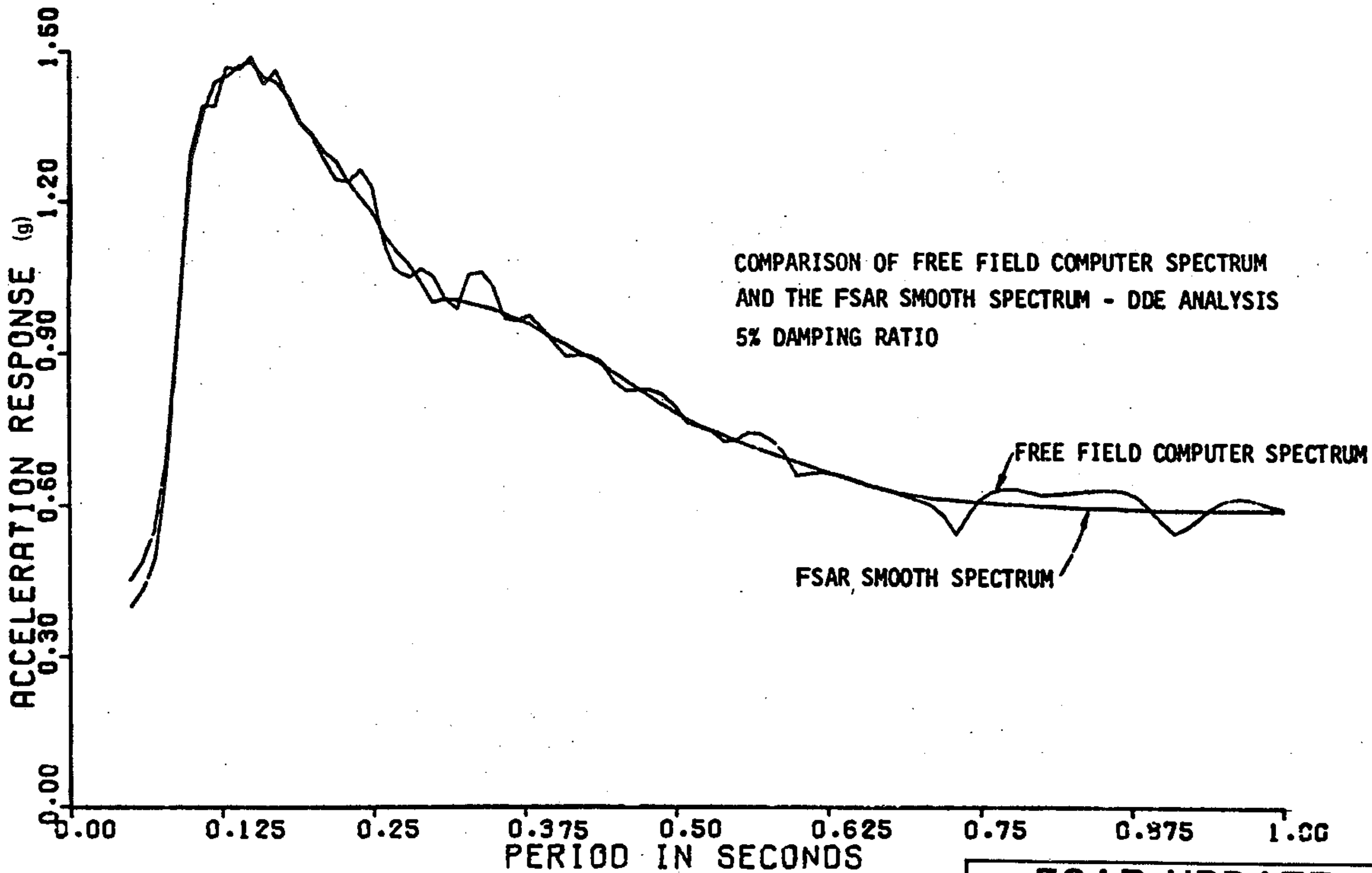
DDE ANALYSIS



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

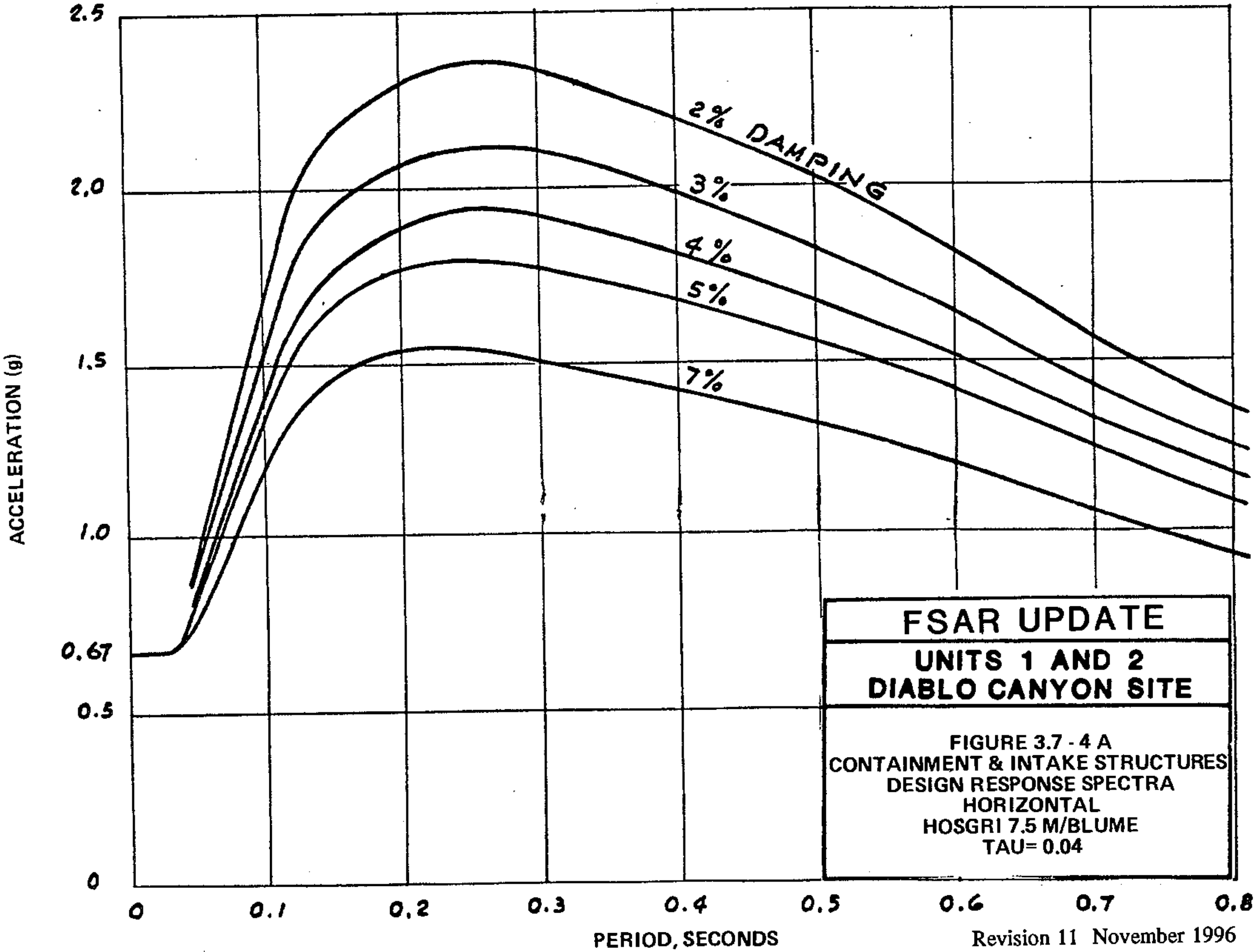
FIGURE 3.7-3
COMPARISON OF SPECTRA
2% DAMPING RATIO



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

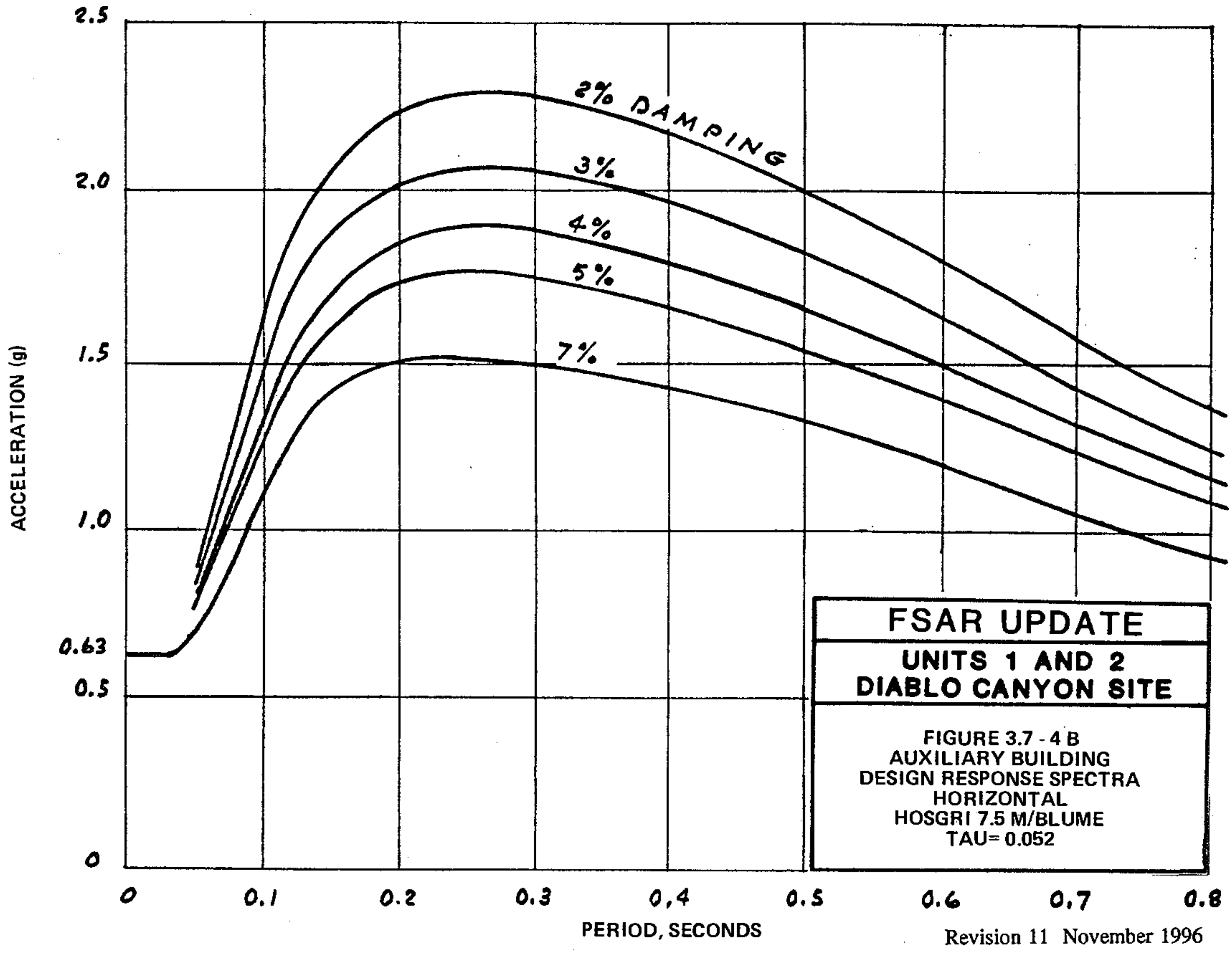
FIGURE 3.7-4
COMPARISON OF SPECTRA
5% DAMPING RATIO



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

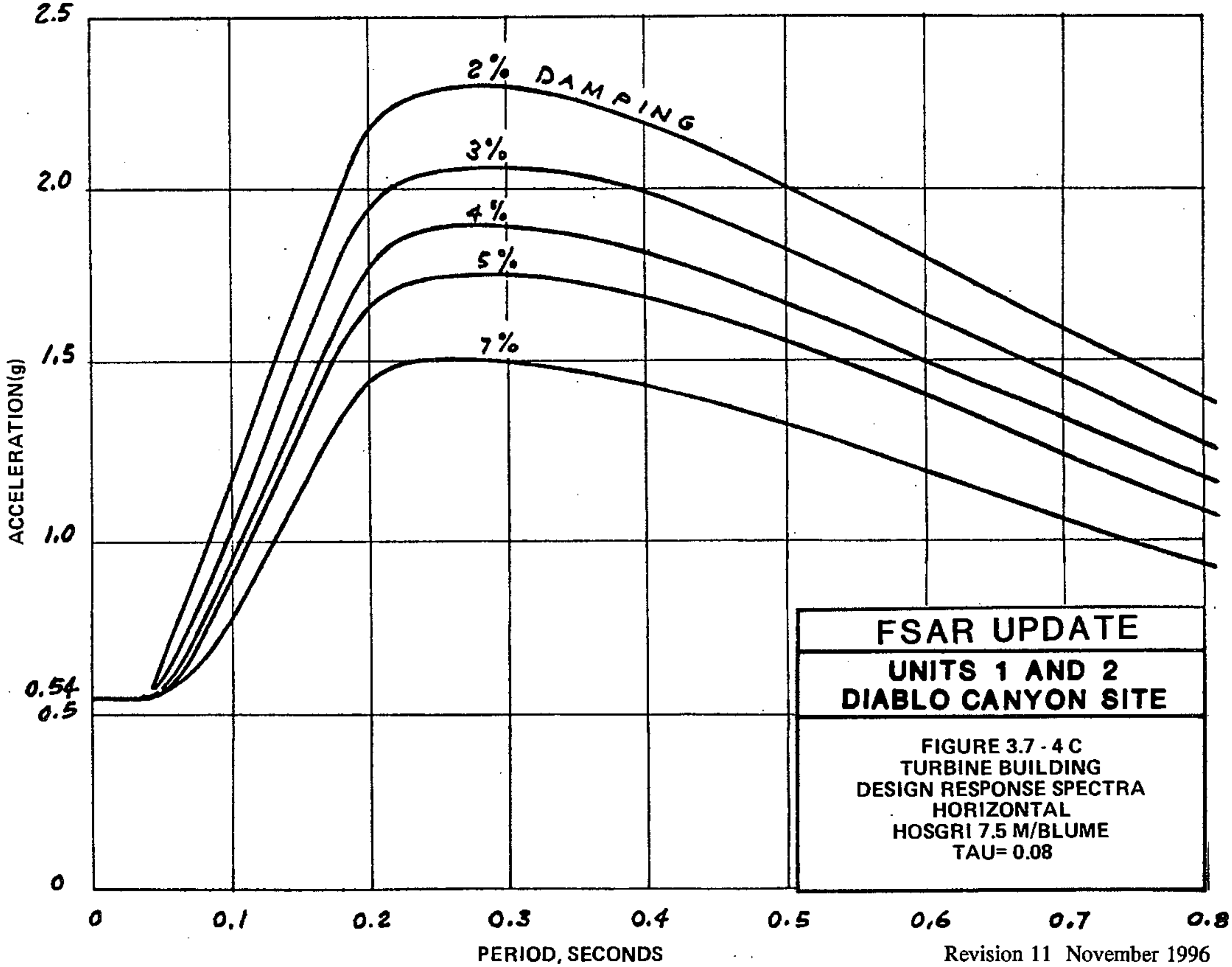
FIGURE 3.7 - 4 A
CONTAINMENT & INTAKE STRUCTURES
DESIGN RESPONSE SPECTRA
HORIZONTAL
HOSGRI 7.5 M/BLUME
TAU= 0.04



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

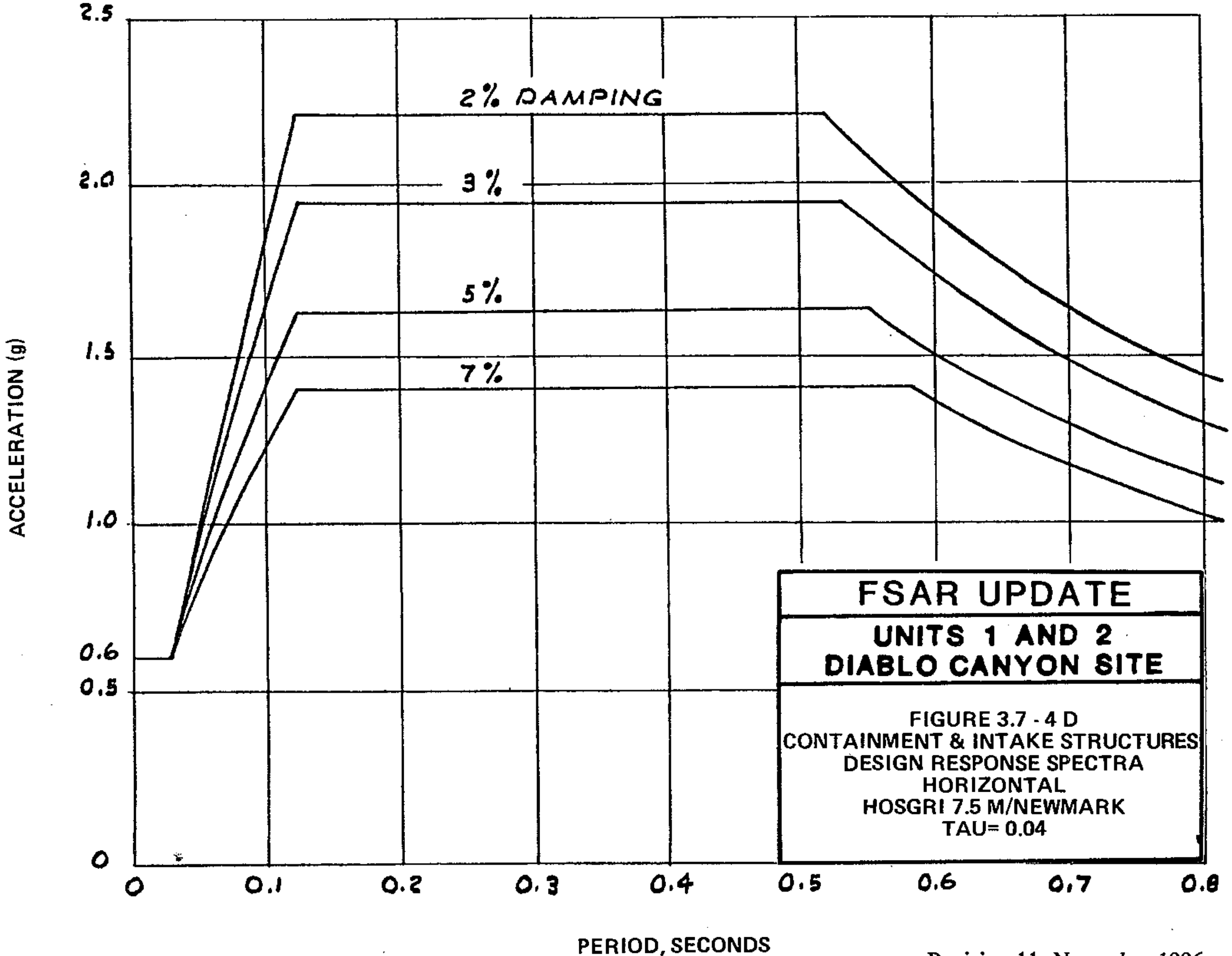
FIGURE 3.7 - 4 B
AUXILIARY BUILDING
DESIGN RESPONSE SPECTRA
HORIZONTAL
HOSGRI 7.5 M/BLUME
TAU= 0.052



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

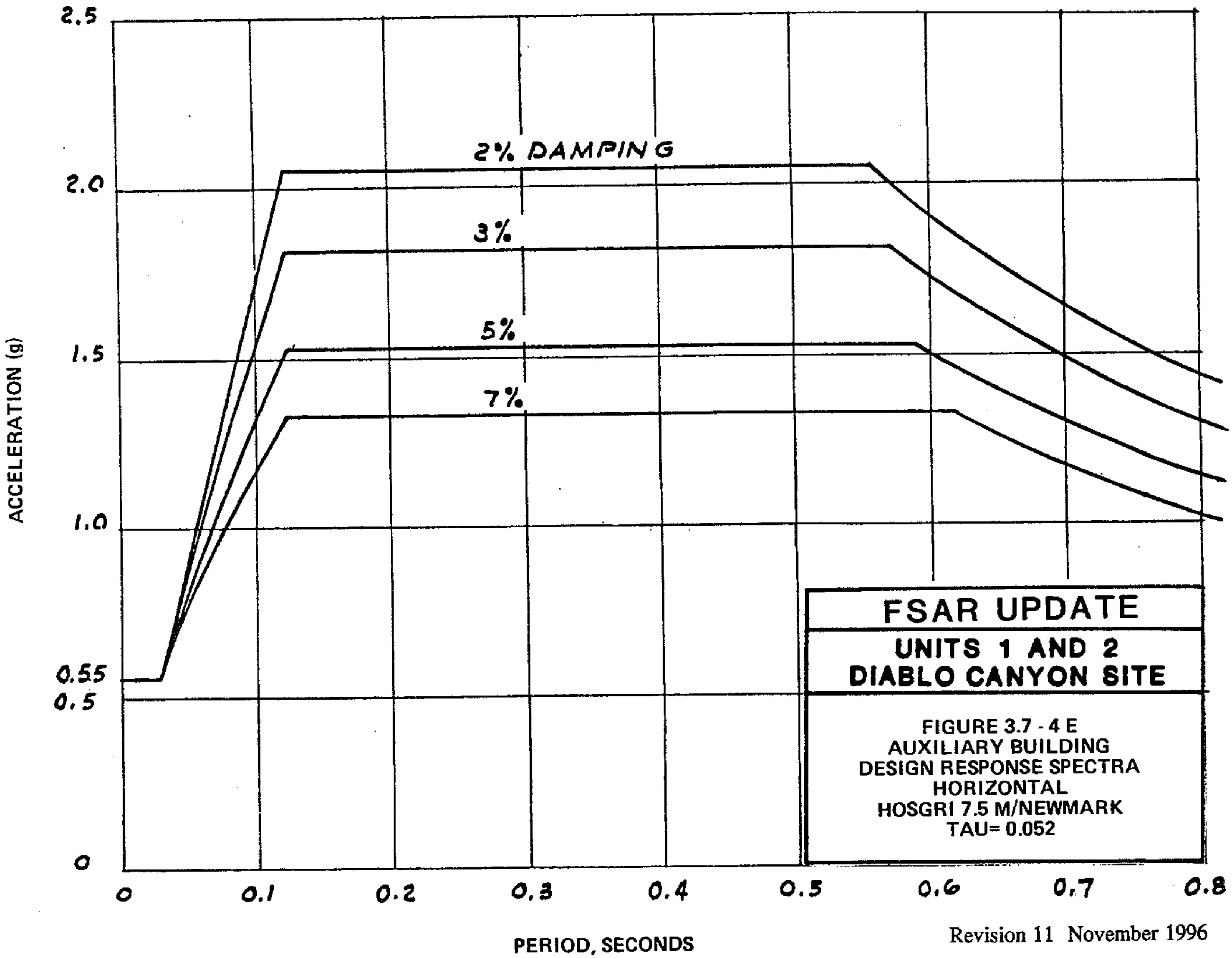
FIGURE 3.7 - 4 C
TURBINE BUILDING
DESIGN RESPONSE SPECTRA
HORIZONTAL
HOSGRI 7.5 M/BLUME
TAU= 0.08



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

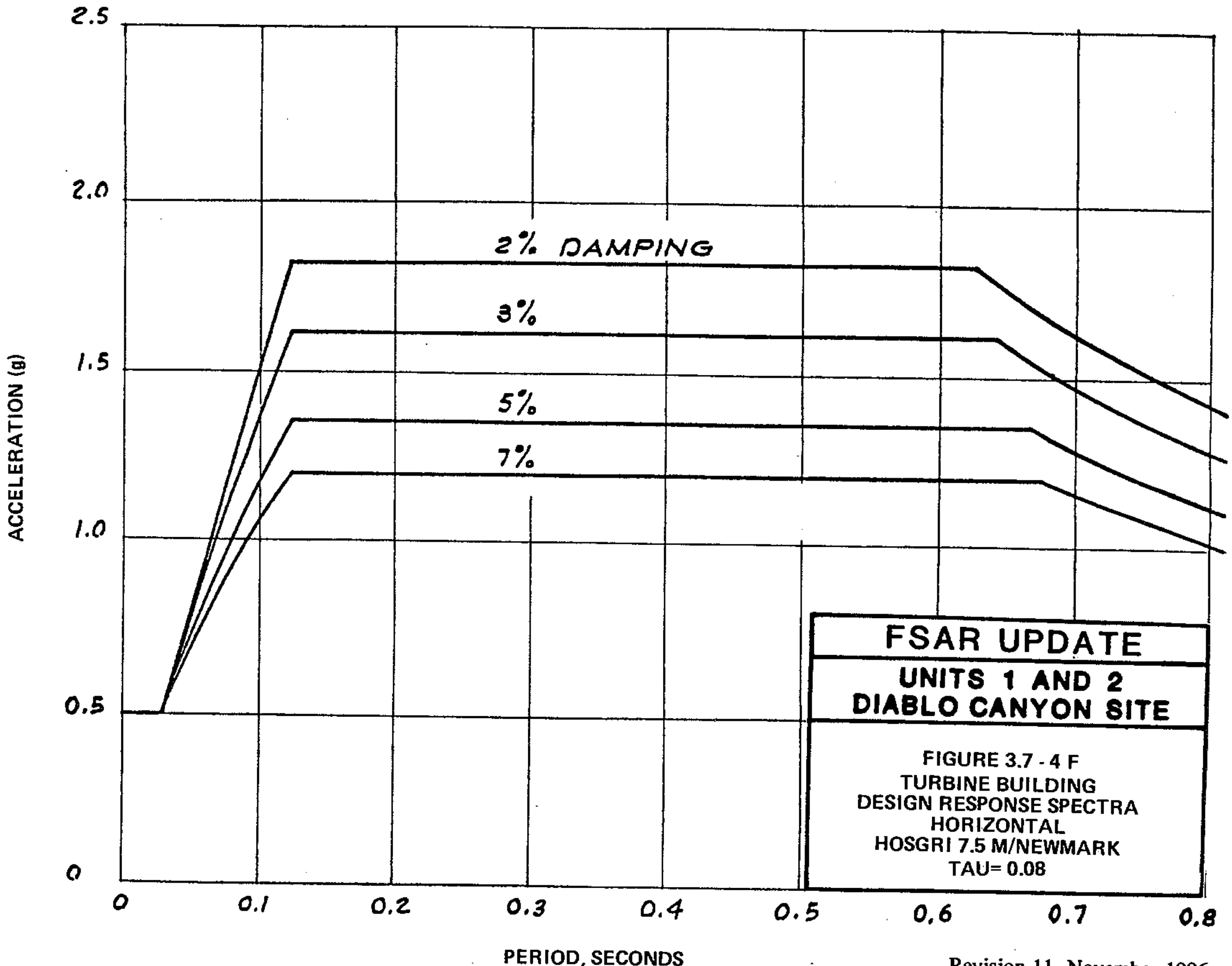
FIGURE 3.7 - 4 D
CONTAINMENT & INTAKE STRUCTURES
DESIGN RESPONSE SPECTRA
HORIZONTAL
HOSGRI 7.5 M/NEWMARK
TAU= 0.04



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

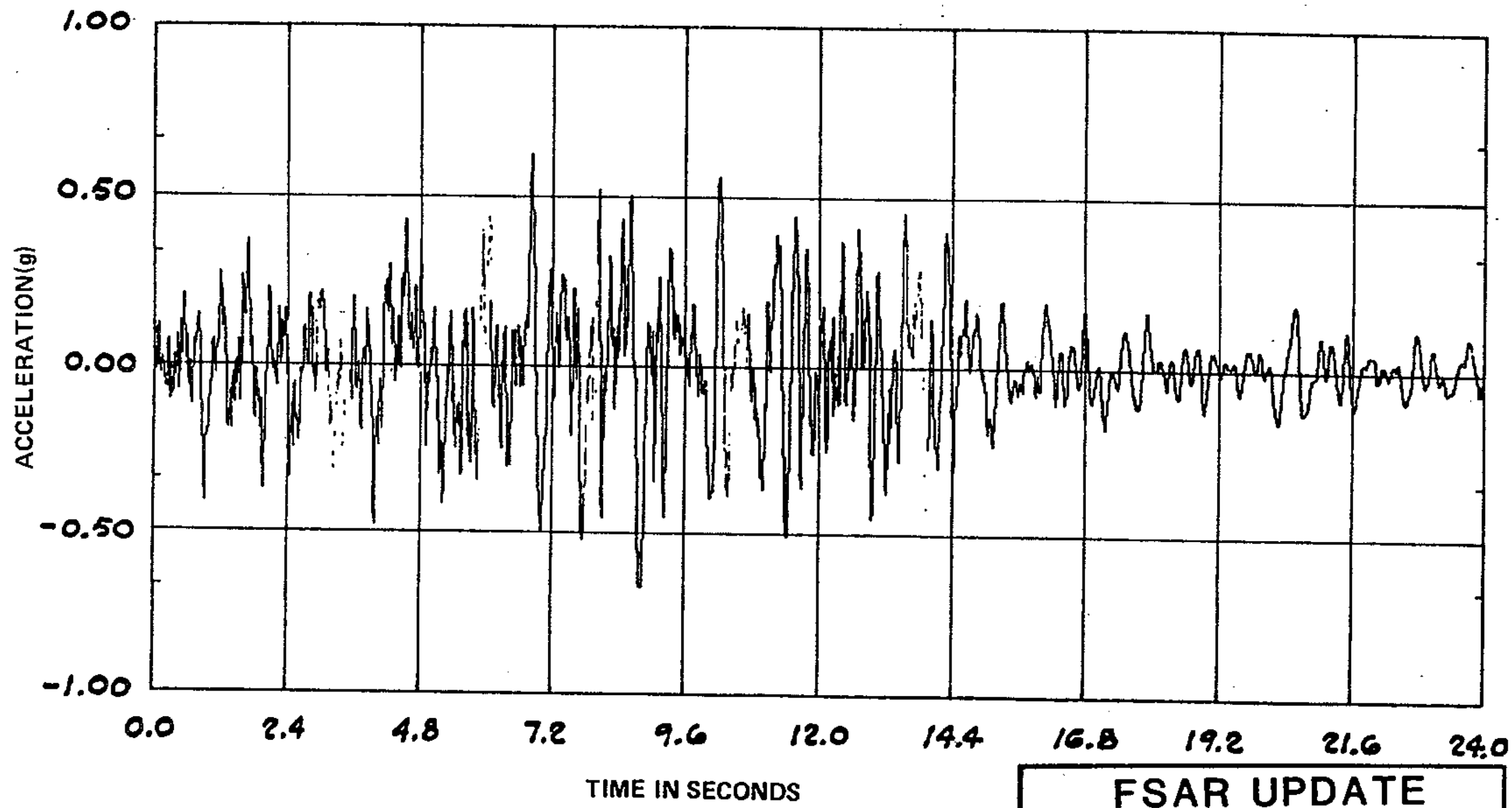
FIGURE 3.7 - 4 E
AUXILIARY BUILDING
DESIGN RESPONSE SPECTRA
HORIZONTAL
HOSGRI 7.5 M/NEWMARK
TAU= 0.052



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

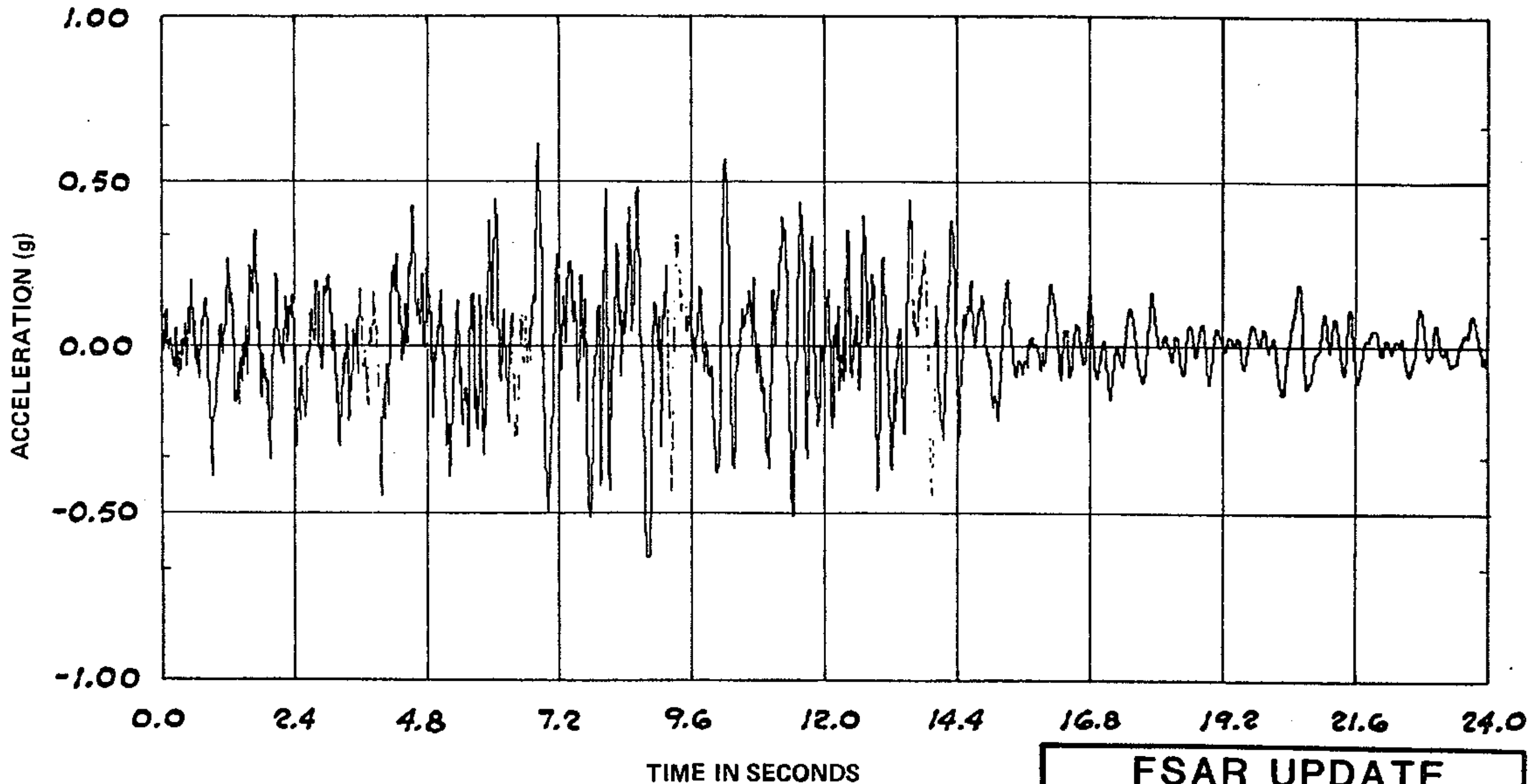
FIGURE 3.7 - 4 F
TURBINE BUILDING
DESIGN RESPONSE SPECTRA
HORIZONTAL
HOSGRI 7.5 M/NEWMARK
TAU= 0.08



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

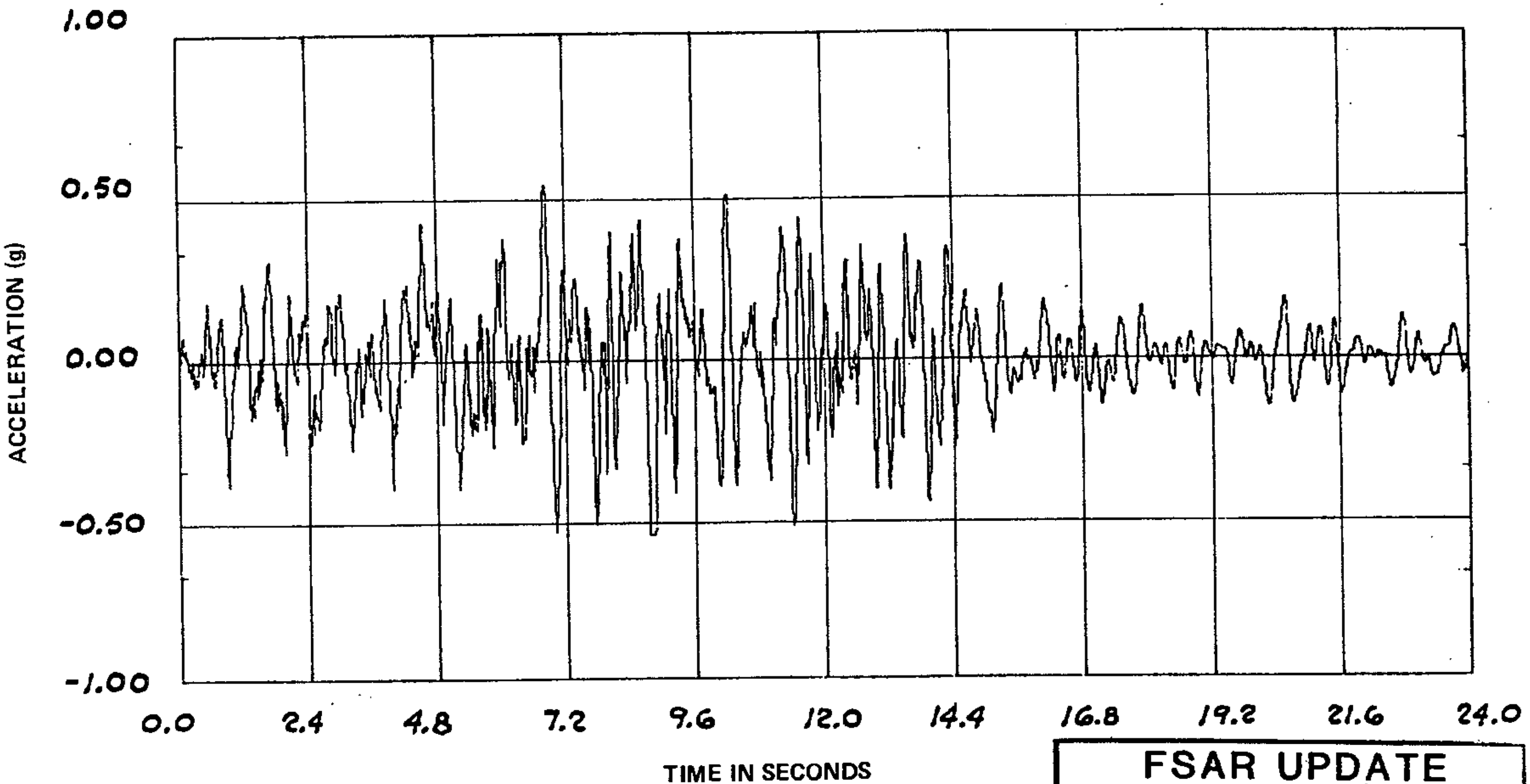
FIGURE 3.7 - 4 G
CONTAINMENT & INTAKE STRUCTURES
HORIZONTAL TIME - HISTORY
HOSGRI 7.5M/BLUME
TAU = 0.04



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

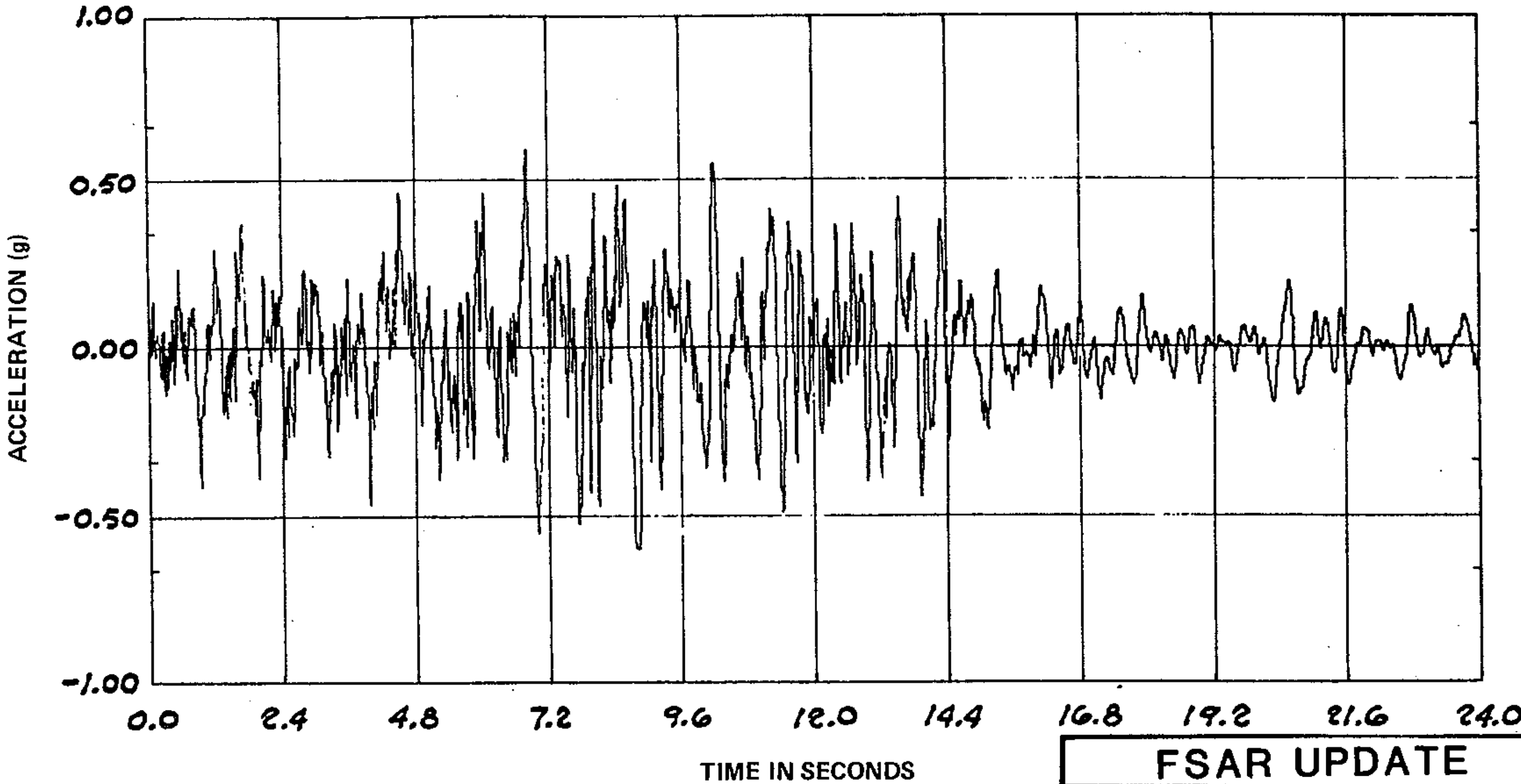
FIGURE 3.7 - 4 H
AUXILIARY BUILDING
HORIZONTAL TIME - HISTORY
HOSGRI 7.5 M/BLUME
TAU= 0.052



FSAR UPDATE

**UNITS 1 AND 2
DIABLO CANYON SITE**

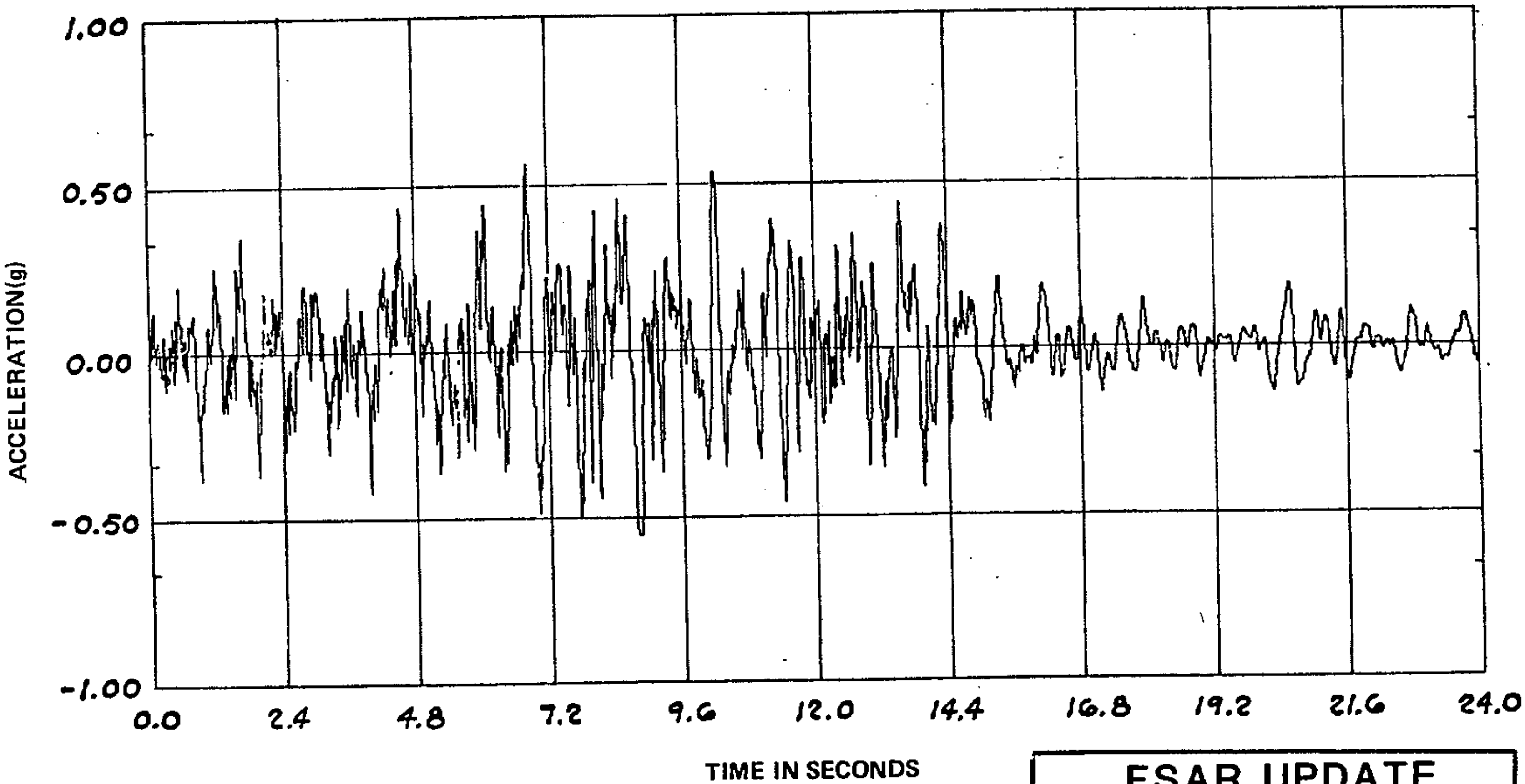
FIGURE 3.7 - 4 I
TURBINE BUILDING
HORIZONTAL TIME - HISTORY
HOSGRI 7.5 M/BLUME
TAU= 0.080



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

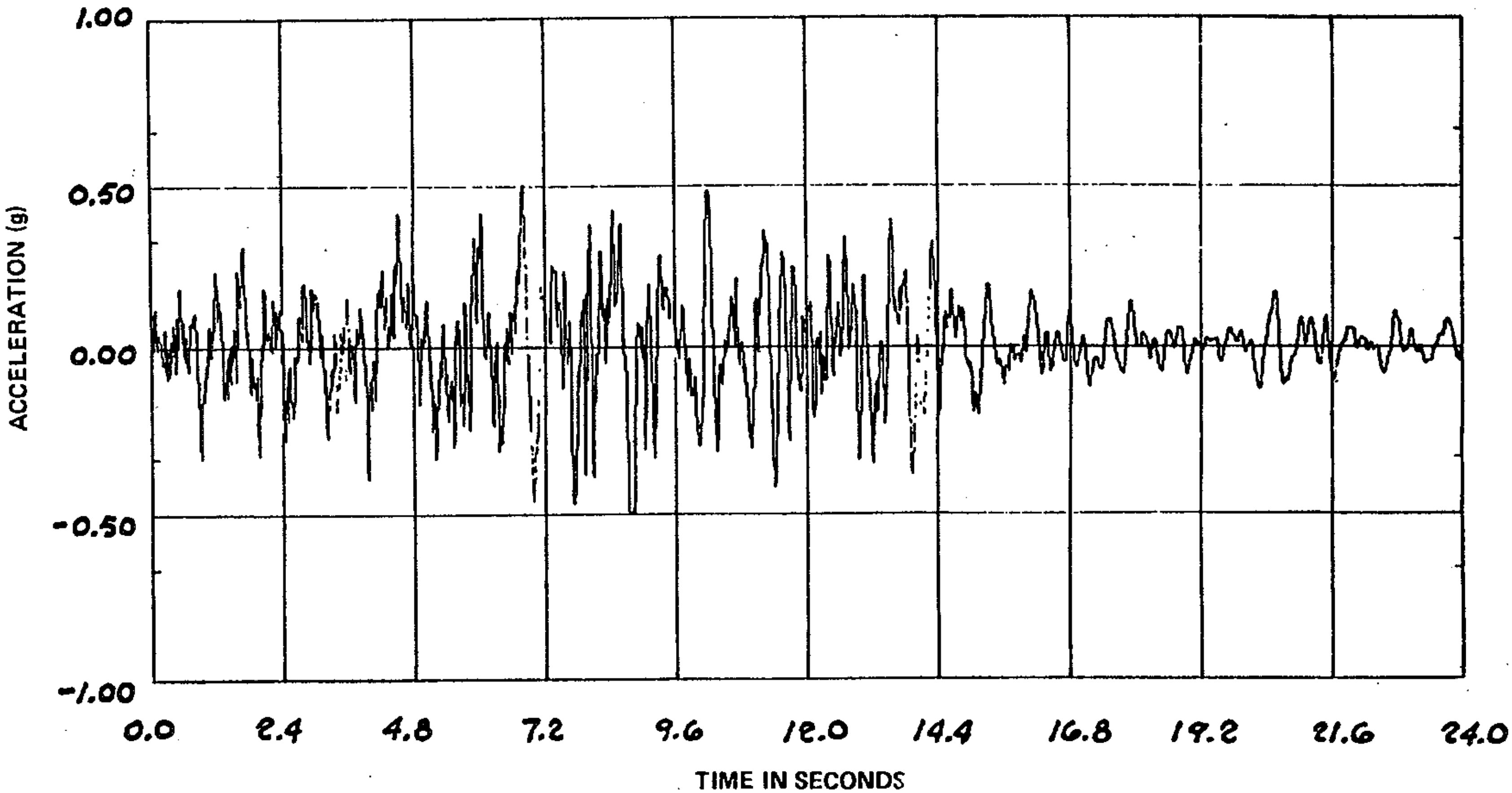
FIGURE 3.7 - 4 J
CONTAINMENT & INTAKE STRUCTURES
HORIZONTAL TIME - HISTORY
HOSGRI 7.5M/NEWMARK
TAU = 0.04



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

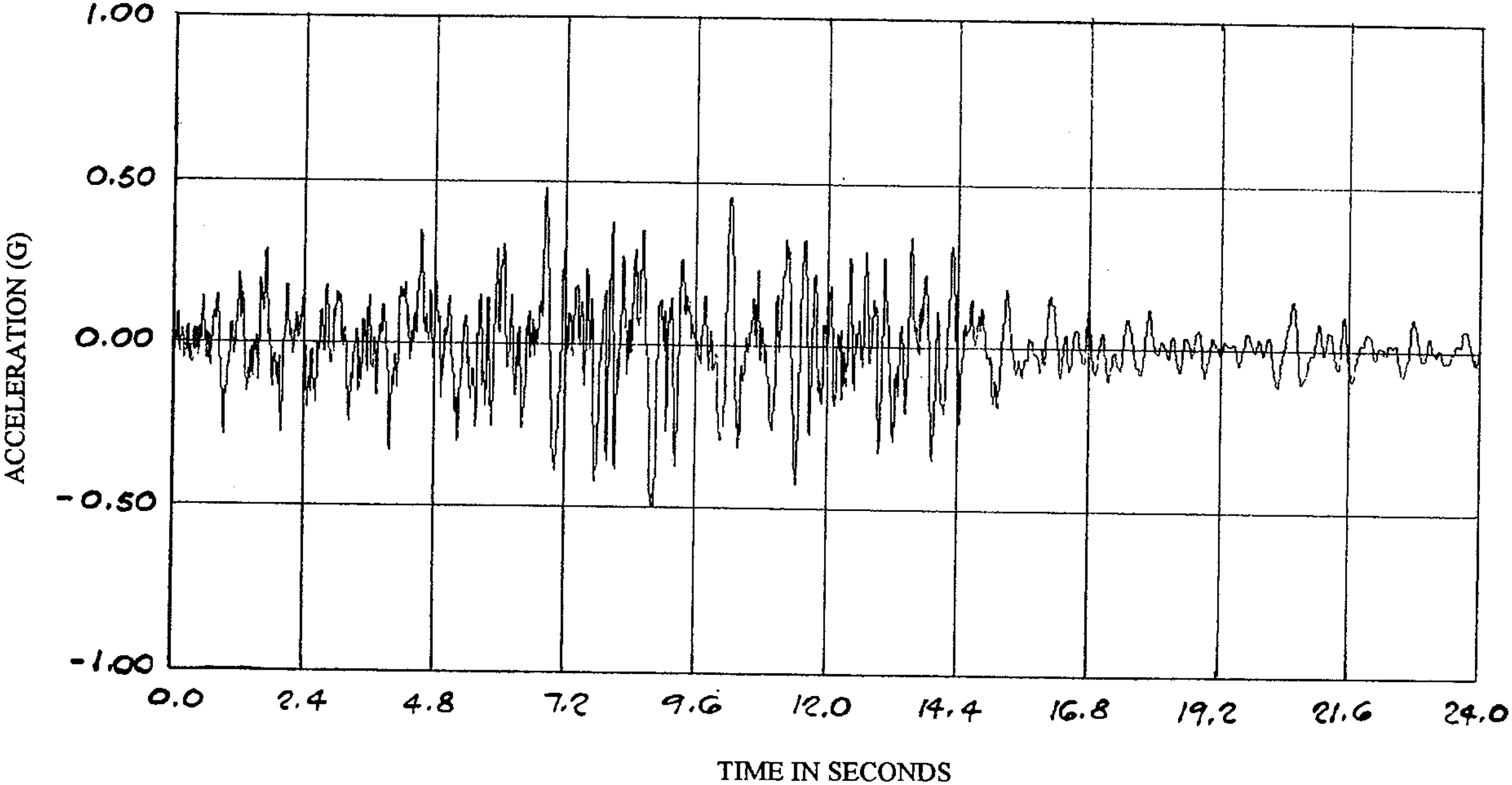
FIGURE 3.7 - 4 K
AUXILIARY BUILDING
HORIZONTAL TIME - HISTORY
HOSGRI 7.5M/NEWMARK
TAU= 0.052



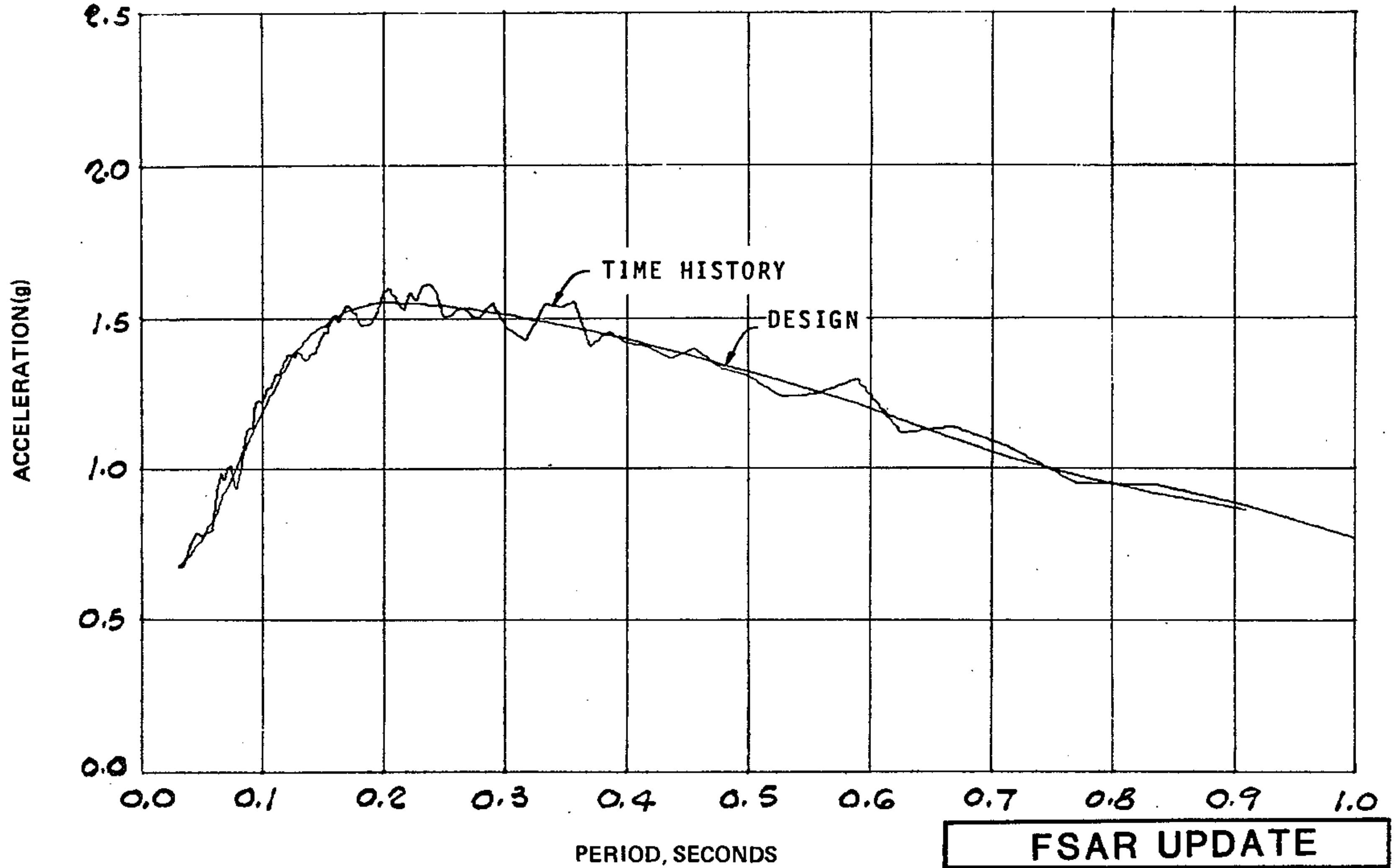
FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 3.7 - 4 L
TURBINE BUILDING
HORIZONTAL TIME - HISTORY
HOSGRI 7.5M/NEWMARK
TAU = 0.067



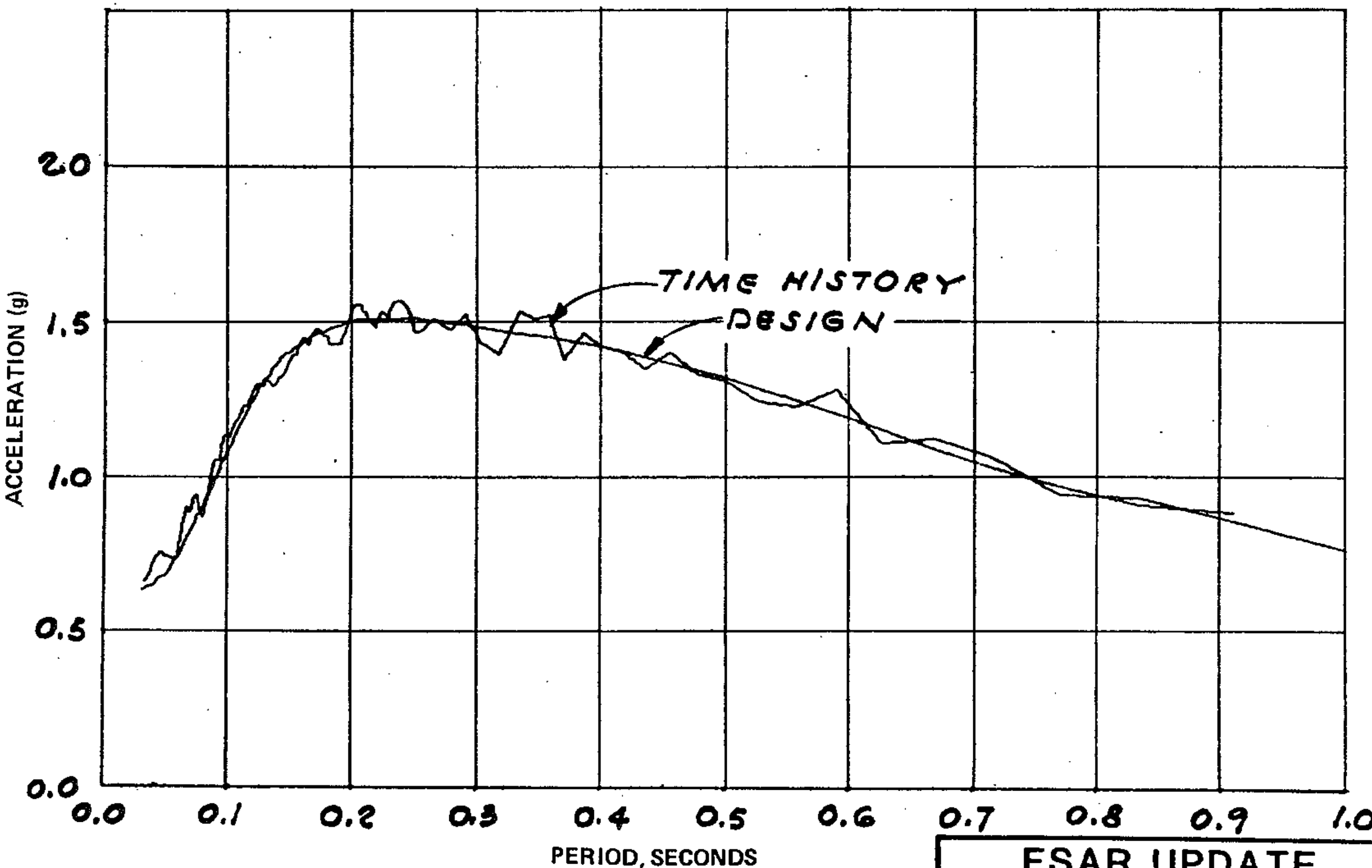
FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 3.7 - 4 M VERTICAL TIME HISTORY HOSGRI 7.5M/NEWMARK TAU = 0.0



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

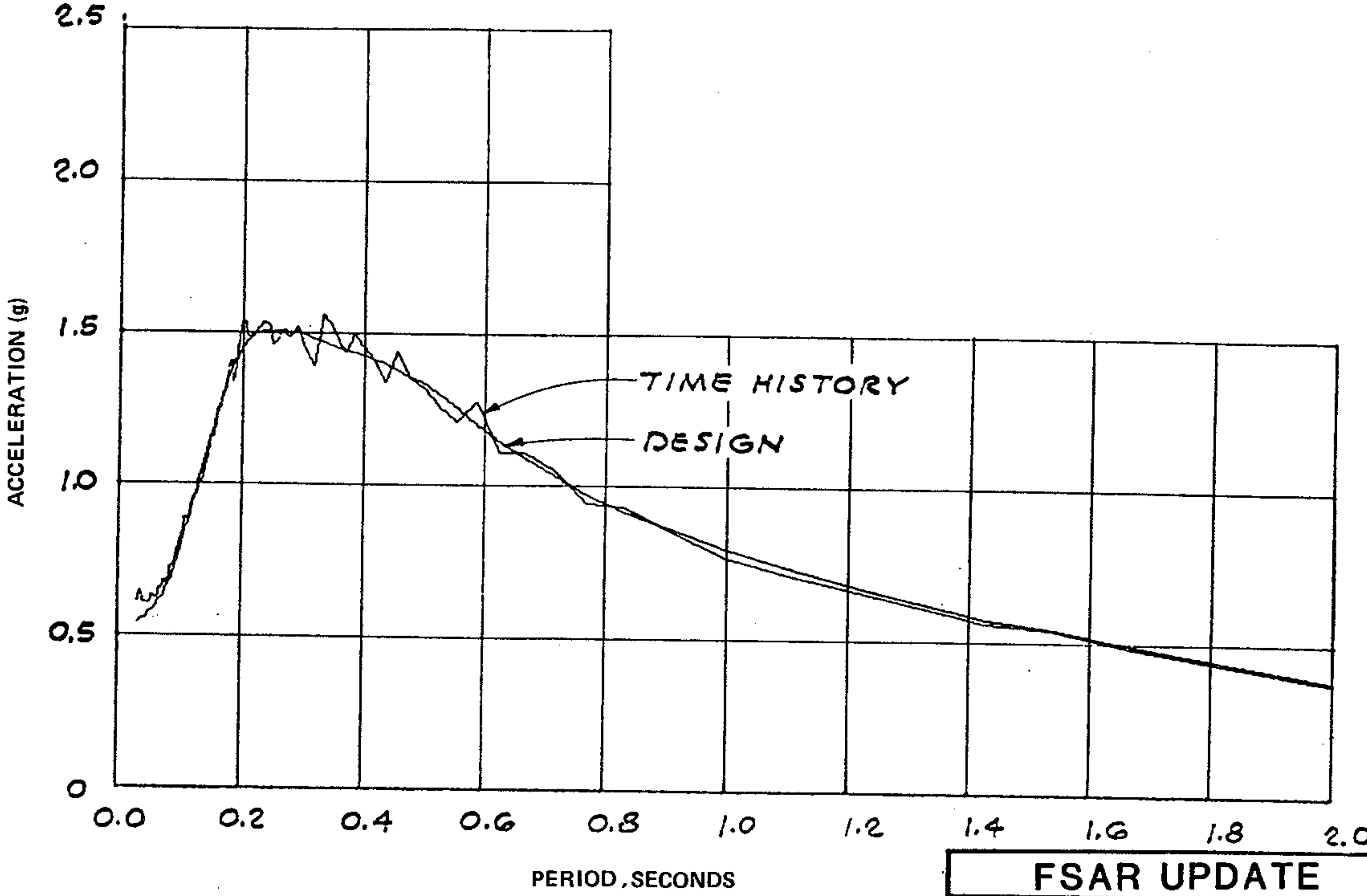
FIGURE 3.7 - 4 N
CONTAINMENT & INTAKE STRUCTURES
COMPARISON OF SPECTRA
HORIZONTAL
HOSGRI 7.5 M/BLUME
TAU= 0.04, 7% DAMPING



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

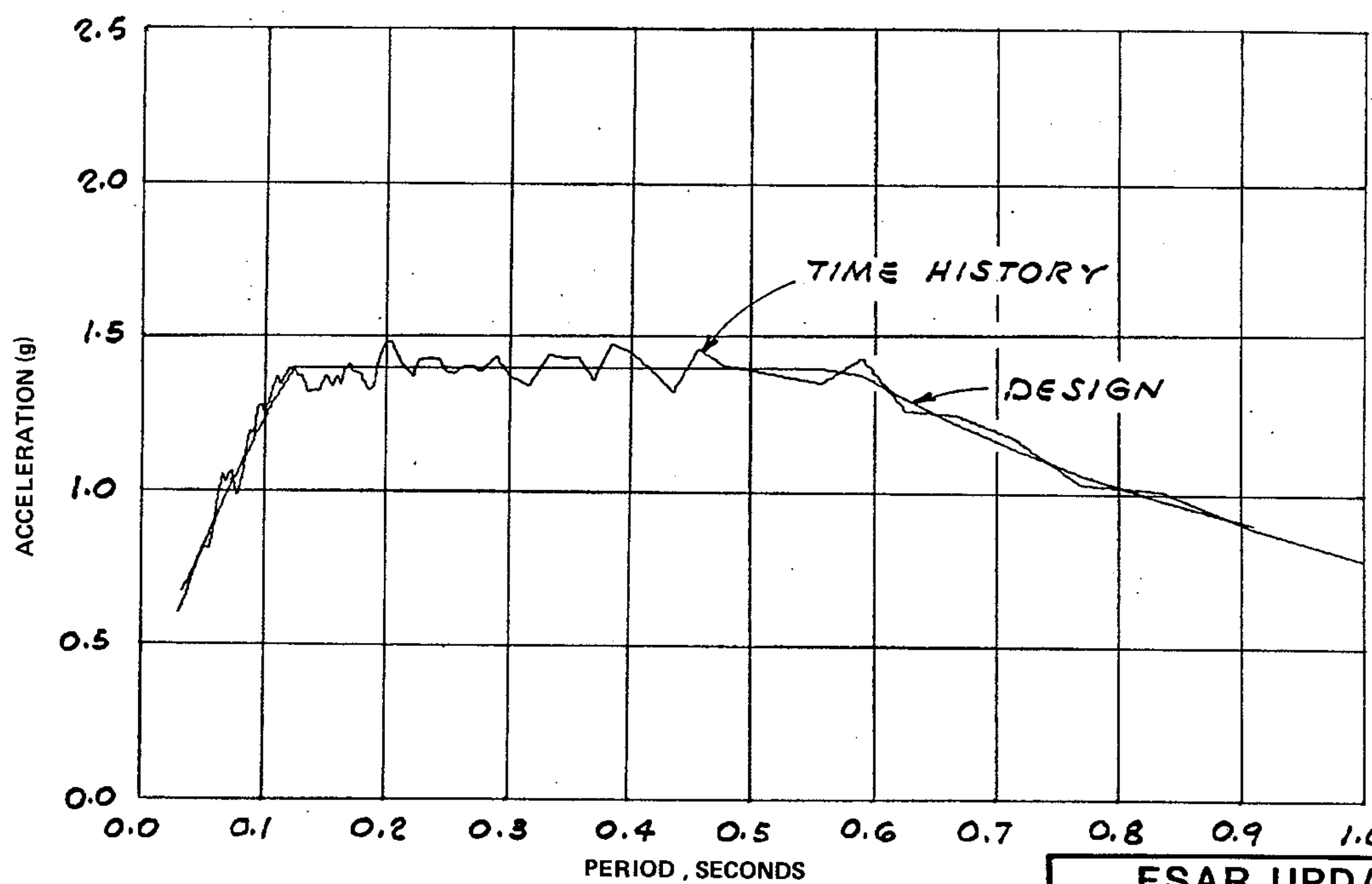
FIGURE 3.7-4 O
AUXILIARY BUILDING
COMPARISON OF SPECTRA
HORIZONTAL
HOSGRI 7.5M/BLUME
TAU= 0.052, 7% DAMPING



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

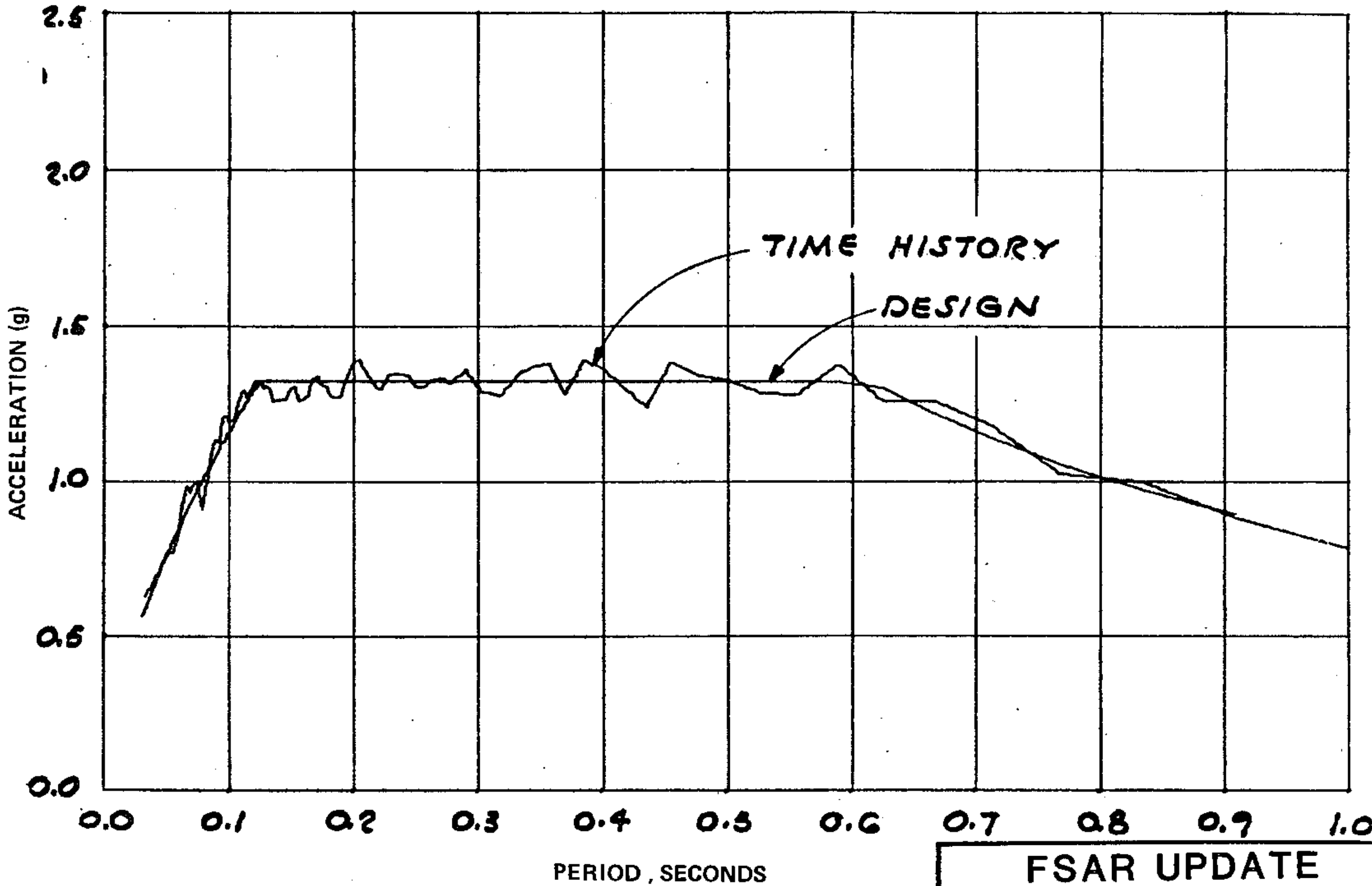
FIGURE 3.7 - 4 P
TURBINE BUILDING
COMPARISON OF SPECTRA
HORIZONTAL
HOSGRI 7.5 M/BLUME
TAU = 0.080, 7% DAMPING



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

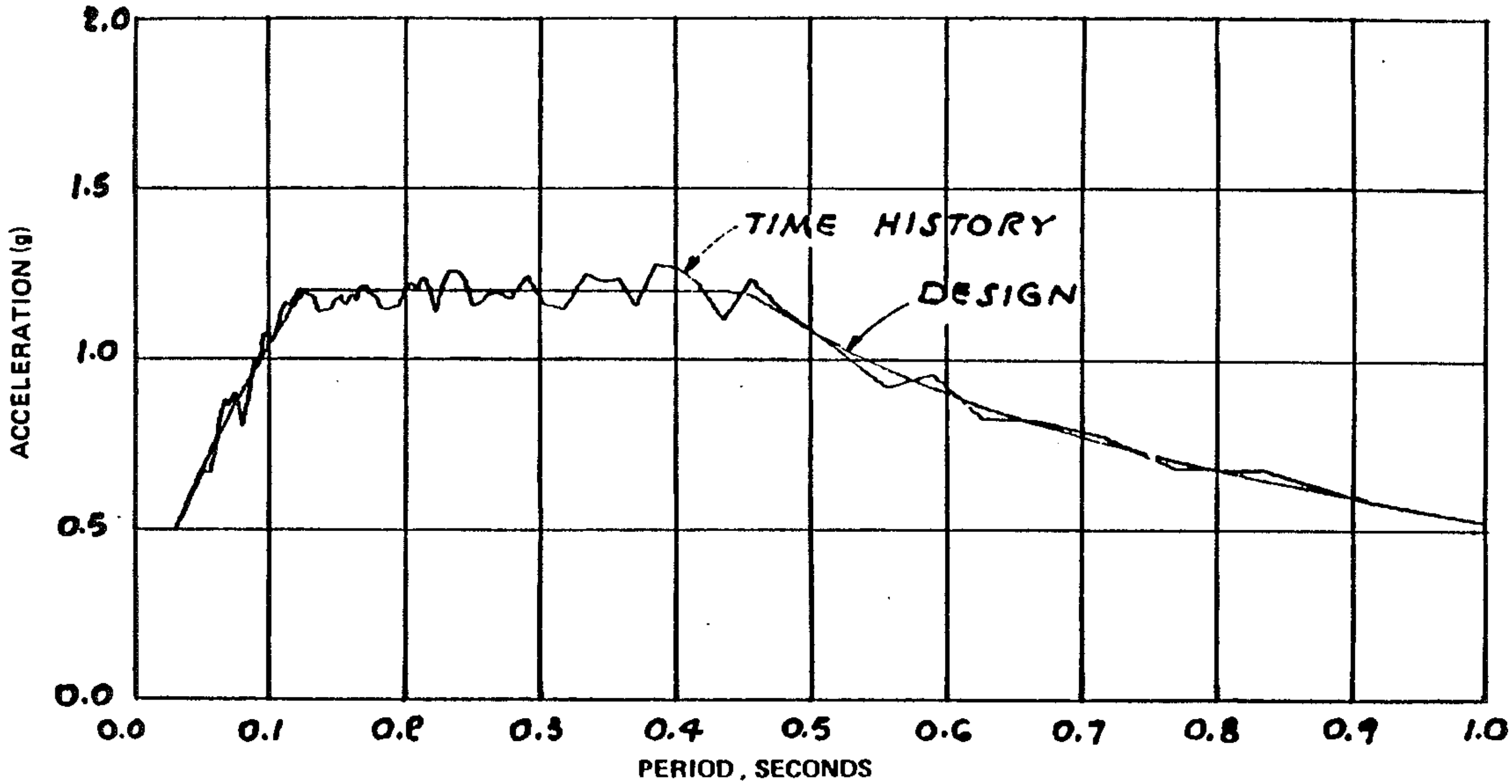
FIGURE 3.7 - 4 Q
CONTAINMENT AND INTAKE
STRUCURES
COMPARISON OF SPECTRA
HORIZONTAL
HOSGRI 7.5 M/NEWMARK
TAU = 0.04, 7% DAMPING



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

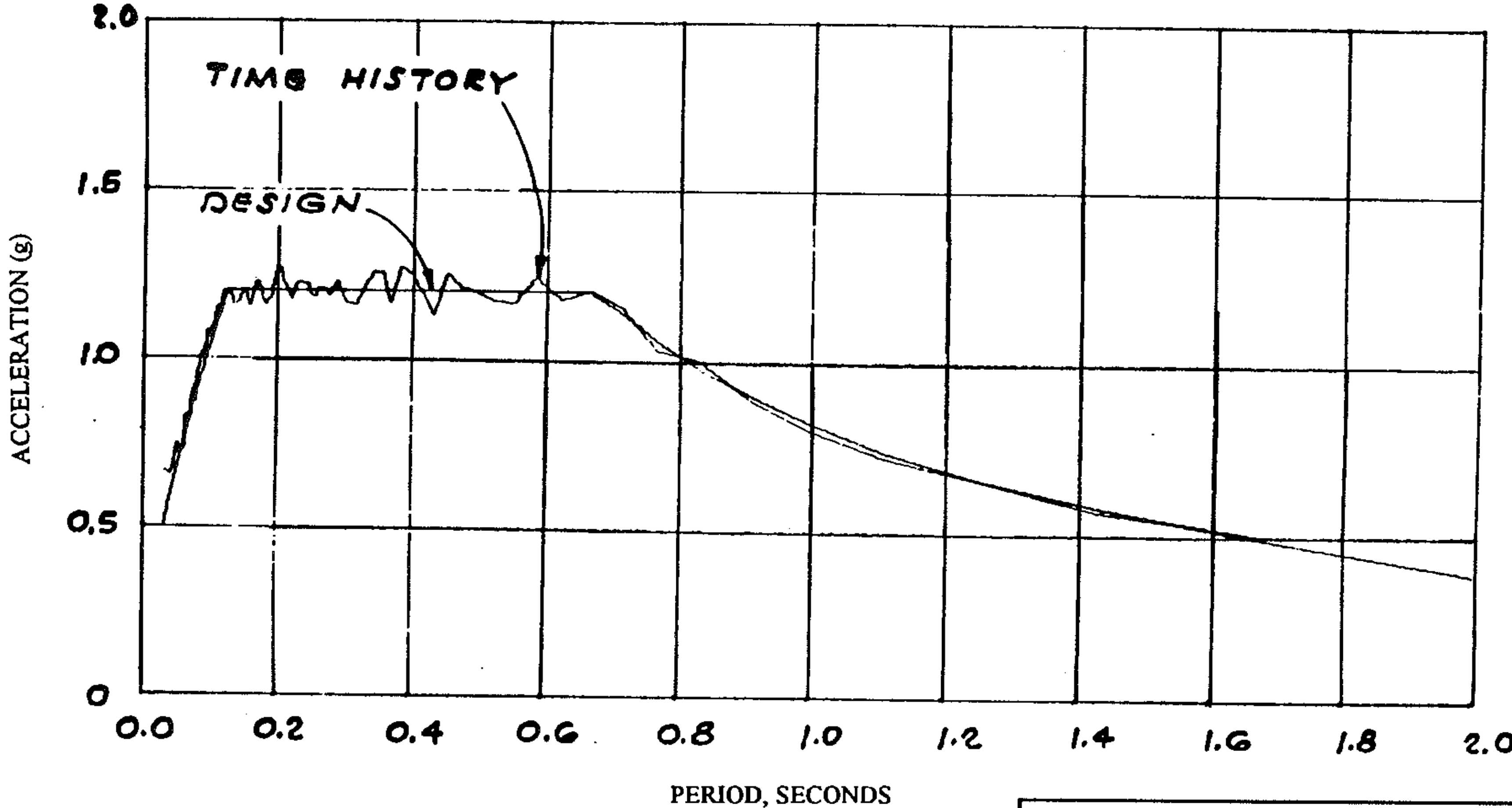
FIGURE 3.7 - 4 R
AUXILIARY BUILDING
COMPARISON OF SPECTRA
HORIZONTAL
HOSGRI 7.5 M/NEWMARK
TAU= 0.052, 7% DAMPING



FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 3.7 - 4S
COMPARISON OF SPECTRA
VERTICAL
HOSGRI 7.5 M/NEWMARK
TAU = 0.0, 7% DAMPING



FSAR UPDATE
UNIT 1 DIABLO CANYON SITE
FIGURE 3.7 - 4 T TURBINE BUILDING COMPARISON OF SPECTRA HORIZONTAL HOSGRI 7.5M/NEWMARK TAU = 0.067, 7% DAMPING