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EISENHUT,D.G. Division of Licensing

SUBJECT: Forwards addl sections for Phase I final rept re design verification program per commitment made in 820820 ltr, Revision 4 to outline Phase 1 final rept & revision to sections previously submitted. S66 Rpt

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TECHNICAL SPECIFICATIONS

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Forwards oral sections rep ... 1 total pgs to ... verification program ... for commitment made in ... ltr. Revision 0 to outline Phase I final rept & revision to sections previously submitted.

Title: ... Canyon ... Independent Design Verification Program

... Documents.

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October 15, 1982

Mr. D. G. Eisenhut, Director
Division of Licensing
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Re: Docket No. 50-275, OL-DPR-76
Diablo Canyon Unit 1

Dear Mr. Eisenhut:

In a letter dated August 20, 1982, PGandE included a schedule for submittal of the Phase I Final Report for the design verification program. Initial sections for that report were submitted on September 1, September 17, and October 1, 1982. In accordance with the August 20 letter, enclosed are additional sections for the Report. These consist of the following sections and parts of sections in Part 2 of the Report:

2.1.3 Fuel Handling Building

2.1.5 Intake Structure

2.3.1 Mechanical Equipment

2.4 Electrical Conduit and Raceway Supports Review

Also enclosed is Revision 4 of the outline for the Phase I Final Report that was originally submitted with the August 20 letter. You will note that certain sections in Part 2, which were previously scheduled to be submitted with this submittal, have been rescheduled.

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1/40 Encls Rec'd

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PDR ADOCK 05000275
P PDR

Further, enclosed are certain minor revisions to sections submitted previously. The revisions correct certain referencing errors and assure continuity of the enclosed sections with the sections submitted earlier. In addition, Section 1.5.4.6, which was originally scheduled to have been submitted on October 1, 1982 but which was inadvertently omitted, has been included with the enclosure.

Very truly yours,

for Richard R Loelse

Philip A. Crane, Jr.

Enclosures

cc: R. H. Engelken
W. E. Cooper

Service List

REVISION NOTE:

The following are revision pages to replace or to be inserted into material submitted previously:

1. Page 1.5.4-9 (insert)

2. Page 1.8.1-3 (replacement)

3. Page 7, Appendix 1E (replacement)

1.5.4.6 Instrumentation Tubing and Tubing Supports

The review of safety-related instrumentation tubing and tubing supports consists of checking the rigidity of tubing and supports for the Hosgri event. For this review, a representative sample of 88 tubing and supports is reviewed and analyzed to determine if seismic capability is adversely affected by revisions to spectra.

The sample for review consists of all tubing and tubing supports located in the portions of the annulus structure that are affected by the revised, reoriented spectra. For instrument tubing, a worst case analysis is performed to assure that the tubing does not exceed allowable stresses.

If tubing supports for safety-related instruments are found to be adequate, no further review is performed. If supports are not found to be adequate, they are reviewed further on a case-by-case basis to determine any implications on supports outside of the sample, and whether further sampling or modifications are required.

Further discussion of the review methodology for instrumentation tubing and tubing support is found in Section 2.6 of Part 2 of this report.

In any event, it appears possible to establish three "categories of causes" which are comprised of contributing factors. Appendices 1C and 1D classify each of the IDVP or ITP open items previously classified as errors according to these three categories.

The deficiencies, discrepancies, deviations, and errors were, more than any single thing, the result of a group of interrelated factors operating in combination. As discussed below, some of these causes were perhaps more pervasive than others, but it is unlikely that any of them individually operated as the "basic cause." In any event, it appears the problem was concentrated in seismic design where there was a convergence of complex design problems, changing criteria, less advanced design controls than now exist, and an unusually large number of design interfaces.

Table E1 (Cont'd)

Item No.	Component Modified	Location	Reason	Description of Modification	EOI or Open Item
10	125-Volt dc switchgear	Inverter room, Area H, elev. 115'	Six 125 V dc breakers feeding instrument inverters were specified with the correct interrupting rating of 20,000a. However, 10,000a rated breakers were supplied and installed in the switchgear. The breaker could, under unusual circumstances, fail and disable one of the three redundant 125 V dc buses.	Replace original breakers with properly rated breakers (DC1-E-E-1345).	OI-28
11	Pipe hangers	Various	See summary Table E2		

OUTLINE
PHASE I FINAL REPORT

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* Results and qualifications for fuel loading.

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* Results and qualifications for fuel loading

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* Results and qualifications for fuel loading

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* Results and qualifications for fuel loading

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* Results and qualifications for fuel loading

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* Results and qualifications for fuel loading

		Due Date To NRC and IDVP
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(project overview, supporting information, and IDVP Reports)

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PHASE I
FINAL REPORT

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2.1.3.4 Design Review

The design review investigated the structural integrity of members, connections, and components necessary to maintain the safety and functional aspects of the fuel handling building. Governing criteria are stated in Section 2.1.3.2 and summarized in Table 2.1.3-1. Analytical procedures used for the design review are described in Section 2.1.3.3. In the following sections, the building modification stages are discussed.

2.1.3.4.1 Evaluation to Criteria

The fuel handling building was first evaluated by conducting a review of the design and vendor drawings. Concurrently, a field inspection was made to verify that the structure conformed to the as-built drawings. Differences were identified between drawings and the as-built conditions which necessitated that a study be performed to check the significance of the differences. A set of preliminary simplified static and dynamic analyses were done to achieve this. The result of this preliminary analysis indicated that several members may not meet the criteria. In order to obtain a better understanding of the behavior of the significant members, more detailed static and dynamic analyses were performed.

The following sections describe the observations made from studying field details, the results of the simplified analysis, and finally the results of the detailed analysis.

2.1.3.4.1.1 Visual Inspection and Simplified Analysis

Review of existing calculations and the report on the "Seismic Evaluation for Postulated 7.5M Hosgri Earthquake" (Reference 2.1.3-2) revealed that the structural stiffnesses of the building and the weight of the roof were not used consistently. Also, it was determined that the as-built slotted expansion joints were not considered to adequately represent the structural behavior during the seismic event.

Review of structural details with the criteria produced the following conclusions:

(1) Several connections did not meet the criteria.

(2) Slotted-hole connections at expansion joints did not allow for sufficient frame movement during seismic events, which would produce undesirable pounding at the joints during the earthquake.

(3) The short columns anchored to the concrete walls of the fan rooms did not meet the criteria at their bases because of the force transmitted to these columns by the high axial load in the east crane girder.

(4) The clip angle connections between the columns and the top and bottom chords running in the north-south direction between bents did not meet criteria.

(5) Connections of the north-south bracing may not meet criteria.

Because of these inconsistencies, a simplified analysis was performed on a single interior bent to capture behavior in the east-west direction, with the following conclusions:

(1) Knee braces and their connections did not meet criteria (Figure 2.1.3-9).

(2) The diagonal members of the main truss did not meet criteria.

(3) The roof truss is in the "flexible" frequency range, and the vertical modal response could be significantly larger than that resulting from a single degree of freedom analysis.

(4) A single degree of freedom model was considered to be adequate for horizontal modal responses; 93% of the total modal effective weight was represented in the first horizontal mode.

For the north-south direction (Figure 2.1.3-10), a simplified multiframe analysis was performed to investigate the effect of earthquake and crane loads, assuming the expansion joints to be locked. The analysis revealed, as expected, that the crane forces were shared by adjacent bays; but the magnitude of the load to the top and bottom roof truss bracing could not be readily determined by using the simplified analysis.

The conclusion at this stage of the review was that the number of members potentially not meeting the criteria could not be adequately assessed under the simplified analyses. To gain a better insight into the behavior of the structure, more detailed analyses were initiated.

2.1.3.4.1.2 Detailed Seismic Analyses

Three-dimensional models were developed, as described in Section 2.1.3.3. The models reflected the data obtained from the construction drawings, as supplemented by the steel fabrication drawings, and information acquired during field walkdowns to represent the as-built conditions. For this analysis all expansion joints were assumed to be fixed and the axial degrees of freedom were relieved at the base of the short columns and at the slotted ends of the crane runway girders. These assumptions are substantiated by the proposed modification (see Section 2.1.3.4.2.1).

Modal superposition dynamic analyses were performed with partial Models 2.1 and 2.2 (Figures 2.1.3-2 and 2.1.3-3). Equivalent static analyses were performed with Model 1.0 (Figure 2.1.3-1).

Various crane positions were investigated to produce maximum forces on the members. Crane loads were determined by using equivalent static analyses. Evaluation of member capacity was in accordance with the criteria stated in Section 2.1.3.2 and Table 2.1.3-1.

Typical mode shapes and frequencies of the partial Model 2.2 are shown on Figures 2.1.3-11 to 2.1.3-14.

The analyses showed that the lateral response was adequately represented by either the single degree of freedom or the simplified analyses, and that the simplified and more detailed vertical responses were in close agreement, (Tables 2.1.3.4-1A and 1B).

The results of the 3-D analyses showed that:

(1) The top chord single diagonal struts, which are adjacent to the applied crane loads, did not meet the criteria for axial loads. The struts were also subjected to considerable bending stress due to eccentricity about the weak axis.

(2) More bottom chord than top chord struts did not meet the criteria for the following reasons: most of the bracing is in the bottom chord; the bottom chords have twice the slenderness ratio of the top chords; and, being closer to the runway girders, the bottom chords are more strongly influenced by the crane loads.

(3) On the east side of the north and south bents, lateral movement of the short columns induced large rotations in the top chord, which would cause local yielding.

(4) Knee braces and their connections did not meet criteria. The detailed analyses reduced the amount of stress predicted by the simplified analyses. Compliance with criteria was achieved by modifying the members and their connections (see Section 2.1.3.4.2.1).

(5) Single diagonal vertical bracing on the east and west walls did not meet criteria. Member eccentricity also contributed to the reduced capacity.

(6) Local yielding was indicated at the north and south ends of the east crane runway girder, where the girder is connected to the short columns. These end columns have less movement than the other columns. The large differential lateral and torsional displacements produced high bending moments. A separate detailed crane runway girder analysis, as shown on Figure 2.1.3-15, was performed to determine if the simplified representation of the crane runway girder in the main model (Model 1.0) gave the proper stiffness and stress levels. The result showed that the crane runway girder would meet the criteria except at the north and south ends where interaction with the short columns would produce local yielding.

(7) Connections to the top and bottom chords, to knee braces, to vertical diagonal bracing, and to horizontal members framing between bents, would have some local yielding.

Part of the reason for the connections not meeting the criteria was that the bolt allowable capacity was controlled by the 7th Edition of the AISC Code. If the criteria permitted the use of the 8th Edition (Reference 2.1.3-5), 35% higher stresses in shear and up to 75% higher stresses in bearing would have been permissible, which would result in fewer modifications than those identified in Section 2.1.3.4.2.1.

To summarize, the detailed seismic analyses confirmed the conclusions reached by using the single degree of freedom and the simplified models, and demonstrated that the diagonal bracing of the top and bottom chords and certain connections did not meet criteria. The detailed seismic analysis, with the full model (Model 1.0), provided a thorough review of the significant members and their connections. The structural elements and the connections which did not meet the criteria were identified for modification.

2.1.3.4.1.3 Results of Review

Detailed review of calculations, the Hosgri Report, and construction drawings, in conjunction with the as-built verification and the subsequent analyses performed on simplified and detail models, identified that certain structural steel members and connections did not meet criteria. However, the fuel handling building, being constructed entirely of structural steel, has inherent ductility and adequate energy reserve to sustain deformation beyond the criteria requirements, and thus absorb the input demand. Considering the results of the analysis, our judgement is that although a limited number of the structural members may not meet criteria, structural failure will not occur.

2.1.3.4.2 Description of Modifications

Modifications are proposed to comply with criteria while maintaining an efficient construction program. These were achieved by the following iterative processes of analysis and design, as well as by maintaining constant communication with field engineers.

2.1.3.4.2.1 Proposed Modifications

The proposed modifications are shown on Figures 2.1.3-16 and 2.1.3-17.

(1) All bays separated by expansion joints will be modified to provide continuity. Thus, seismic pounding is avoided.

(2) The top chord single-strut diagonal members will be strengthened to increase tension and compression capacities; and eccentricity will be reduced by adding single-strut angles back to back, and welding member ends to gusset plates (Figure 2.1.3-16).

(3) Diagonal braces will be added to the center portion of the bottom chord roof truss. These will distribute to the adjacent members the high axial forces due to localized crane loads (Figure 2.1.3-16).

(4) All double-angle diagonal members of the main truss on Figure 2.1.3-13 will have filler plates installed, as required, to satisfy slenderness ratio requirements (Reference 2.1.3-4).

(5) The knee braces on Figure 2.1.3-9 will be replaced with members having greater capacity.

(6) Vertical, diagonal, and horizontal braces will be added to the east-west walls to increase lateral stability while making an

efficient load distribution, thereby decreasing axial tensile loads to columns, base anchors, and existing single-strut diagonals (Figure 2.1.3-17).

(7) Portions of the east crane runway girder, at the north and south ends, will be cut and removed, thereby relieving stresses on the short end columns (Figure 2.1.3-17).

(8) Connections which do not meet criteria will be strengthened. In most cases, this will be achieved by welding steel plates.

During structural modification some of the bracing connections will be temporarily unbolted. The sequence of modifications will be stated in the modification documents, and will be scheduled to preserve the overall safety and stability of the building throughout the modification phase.

2.1.3.6 References

2.1.3-1 Pacific Gas and Electric Company, "Final Safety Analysis Report," PGandE, San Francisco, CA (1974).

2.1.3-2 Pacific Gas and Electric Company, et al, "Seismic Evaluation for Postulated 7.5M Hosgri Earthquake," PGandE, San Francisco, CA (November 1977 with amendments).

2.1.3-3 Nuclear Regulatory Commission, "Safety Evaluation Report," Section 3.7 and Supplements 7 and 8, NRC, Washington, D.C. (1978).

2.1.3-4 American Institute of Steel Construction, "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings," AISC, Chicago, IL, (Seventh Edition).

2.1.3-5 American Institute of Steel Construction, "Specifications for the Design, Fabrication, and Erection of Structural Steel for Building," AISC, Chicago, IL, (Eighth Edition).

TABLE 2.1.3.4-1A

COMPARISON OF HORIZONTAL FUNDAMENTAL MODES

	Detailed Seismic Analysis 3-D Models				Simplified Analysis 2-D Model		
	Model 2.1		Model 2.2				
First[1] Fundamental Modal Direction	Frequency (cps)	Modal Effective Mass (%)	Frequency (cps)	Modal Effective Mass (%)	Frequency (cps)	Modal Effective Mass (%)	Single Degree of Freedom Frequency (cps)
E-W	1.6	85.0	1.6	86.0	1.6	93.0	1.5
N-S	3.1	88.0	2.7[2]	92.0	Not Computed	Not Computed	3.1

Notes

(1) Other modes have insignificant contributions, therefore not included in comparison.

(2) Frequency slightly less than those of Model 2.1 and 2-D model is because center span of Model 2.2 is longer than those of Model 2.1.

TABLE 2.1.3.4-1B

COMPARISON OF VERTICAL ROOF MODES

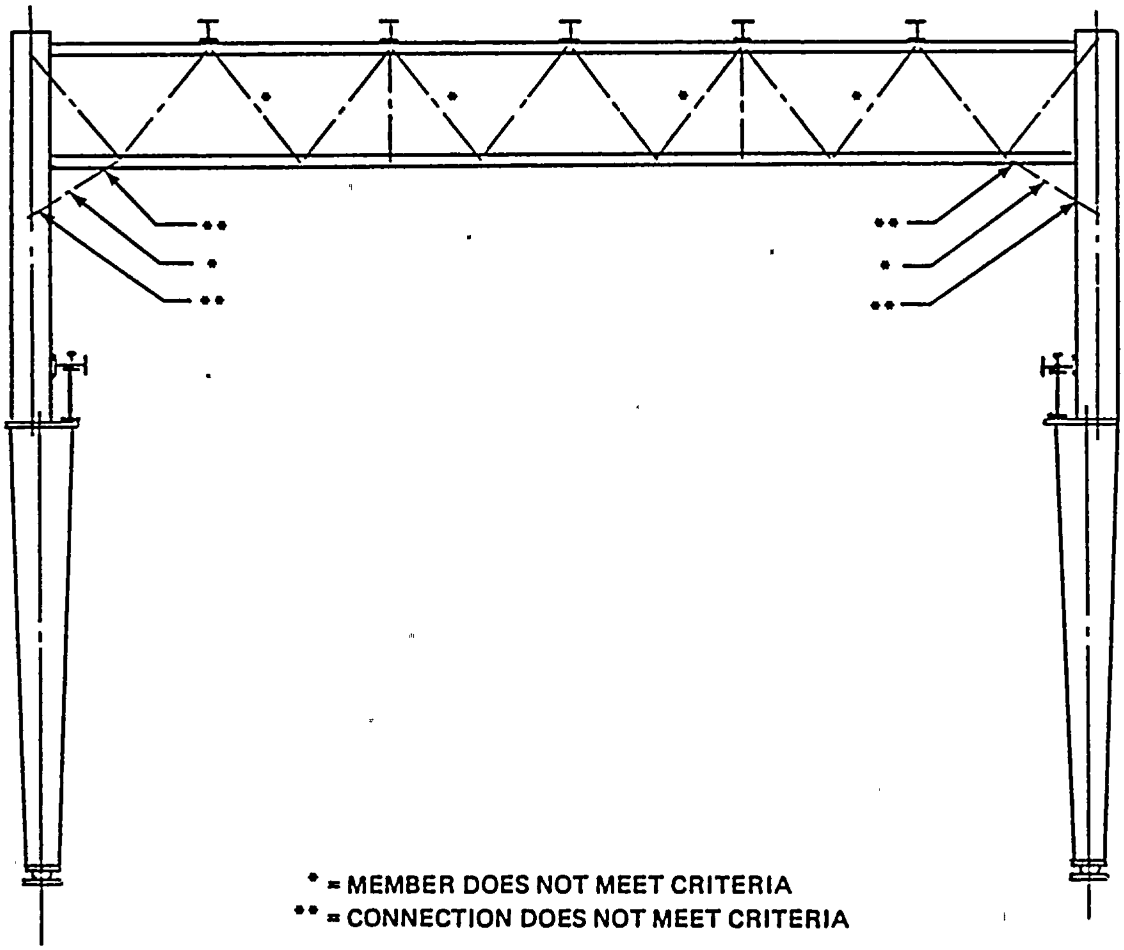
OF SIGNIFICANT CONTRIBUTION

Detailed Seismic Analysis				Simplified Analysis	
3-D Models				2-D Model	
Model 2.1		Model 2.2			
Frequency (cps	Modal Effective Mass (% of Roof)	Frequency (cps)	Modal Effective Mass (% of Roof)	Frequency (cps)	Modal Effective Mass (% of Roof)
11.3	22.0	10.8	29.0	11.6	52.0
15.7	4.0	11.6	4.0		
16.3	1.0	16.3	9.0		
17.2	9.0	17.4	2.0		
18.4	4.0	18.1	3.0		
19.6	3.0	20.7	5.0		
27.5	1.0	29.3	1.0	28.0	Negligible
31.4	Negligible	31.7	Negligible		
Σ =	44.0%		53.0%		52.0%
	(1), (2)		(1)		

Notes

(1) The vertical modes have their maximum values of various positions along the north-south direction and at the centers of the east-west spans on top and bottom roof truss. Total sum up to 33 cps compares well with the 2-D model value.

(2) Lower than Model 2.2 value due to no contribution in stiffness from short column.

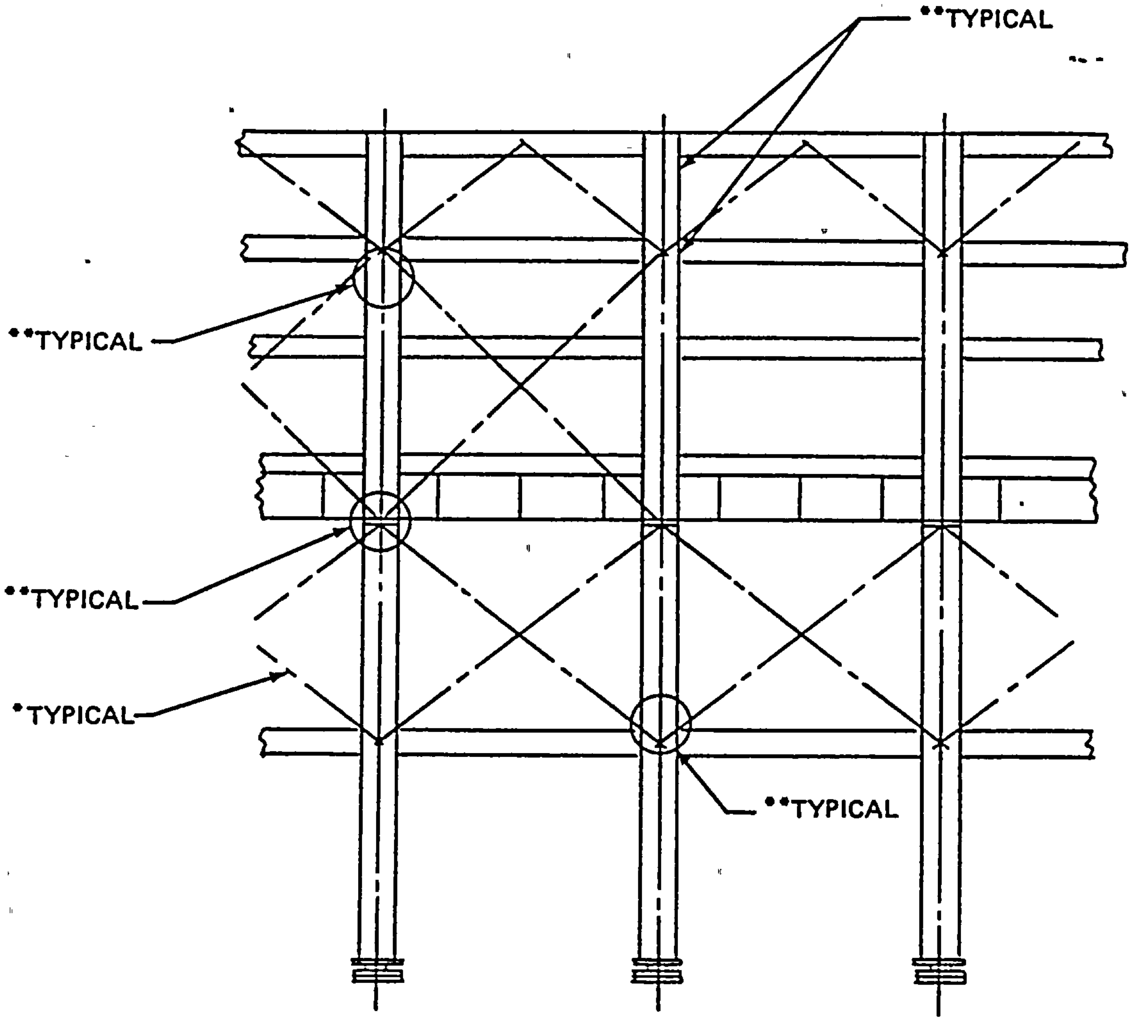


* = MEMBER DOES NOT MEET CRITERIA
** = CONNECTION DOES NOT MEET CRITERIA

FIGURE 2.1.3-9

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

FUEL HANDLING BUILDING
DESIGN REVIEW FINDING
FOR A TYPICAL INTERIOR FRAME
BASED ON SIMPLIFIED ANALYSIS

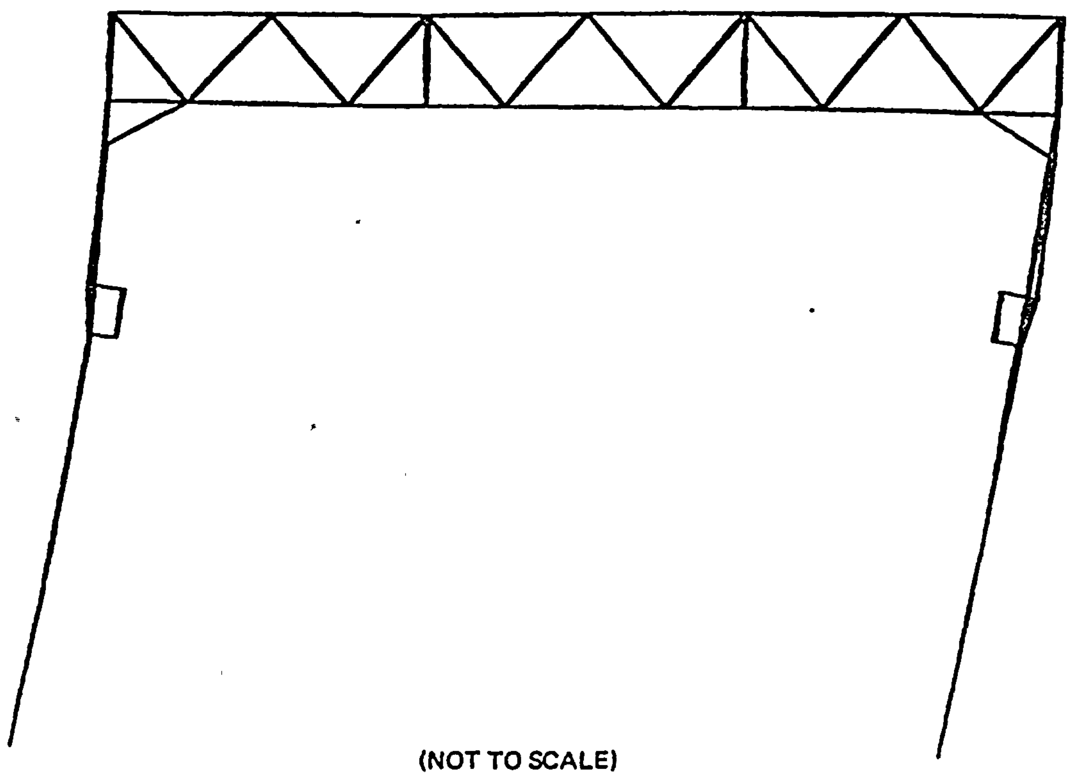


* = SINGLE ANGLE MEMBER DOES NOT MEET CRITERIA
** = CONNECTION DOES NOT MEET CRITERIA

FIGURE 2.1.3-10

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

FUEL HANDLING BUILDING
DESIGN REVIEW FINDING FOR
EAST AND WEST WALL
ELEVATION BASED ON
SINGLE DEGREE OF FREEDOM
ANALYSIS

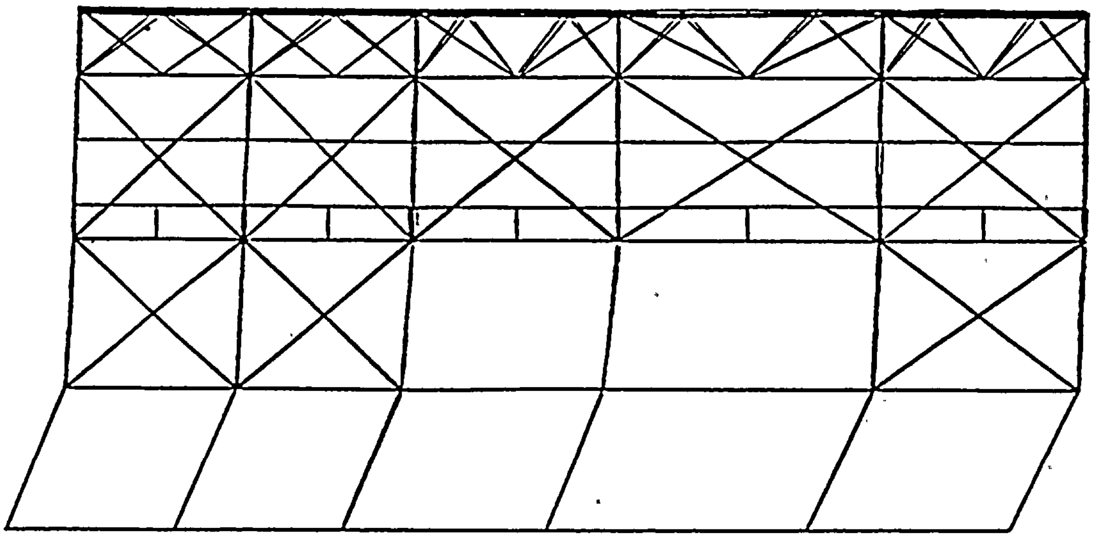


(NOT TO SCALE)

FIGURE 2.1.3-11

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

FUEL HANDLING BUILDING
PARTIAL BUILDING MODES 2.2
MODE SHAPE NO. 1, FREQ. = 1.6 Hz
EAST—WEST DIRECTION
EFFECTIVE MODAL WEIGHT≈86%

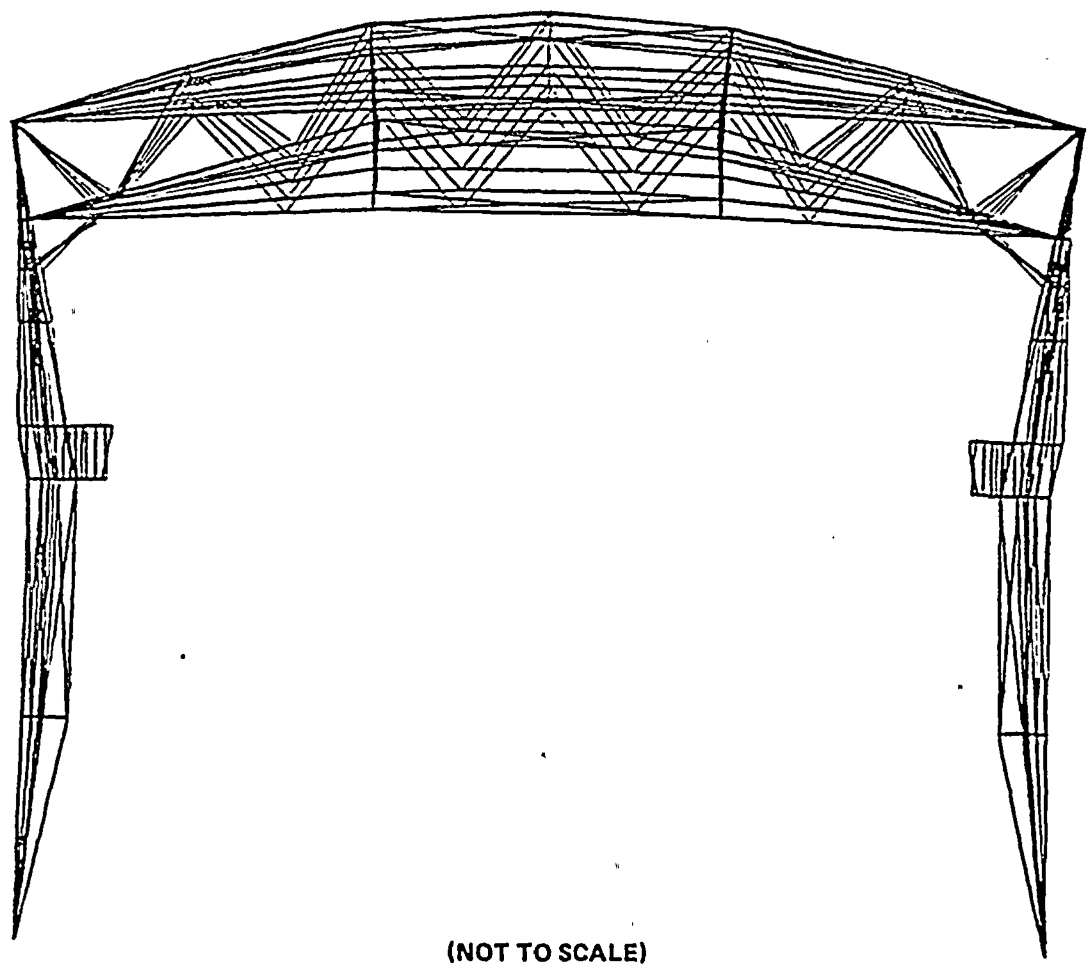


(NOT TO SCALE)

FIGURE 2.1.3-12

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

· FUEL HANDLING BUILDING
PARTIAL BUILDING MODEL 2.2
MODE SHAPE NO. 3, FREQ. = 2.7 Hz
NORTH–SOUTH DIRECTION
EFFECTIVE MODAL WEIGHT ≈ 92%

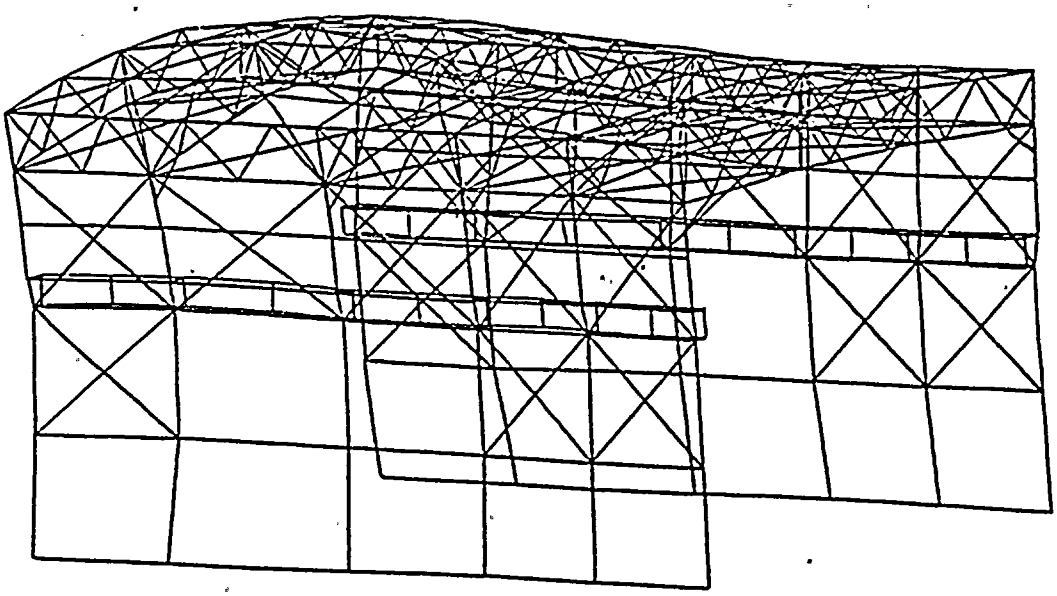


(NOT TO SCALE)

FIGURE 2.1.3-13

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

FUEL HANDLING BUILDING
PARTIAL BUILDING MODEL 2.2
MODE SHAPE NO. 5, FREQ. = 10.8 Hz
VERTICAL DIRECTION
(END VIEW)



(NOT TO SCALE)

FIGURE 2.1.3-14

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

FUEL HANDLING BUILDING
PARTIAL BUILDING MODEL 2.2
MODE SHAPE NO. 5, FREQ. = 10.8 Hz
VERTICAL DIRECTION

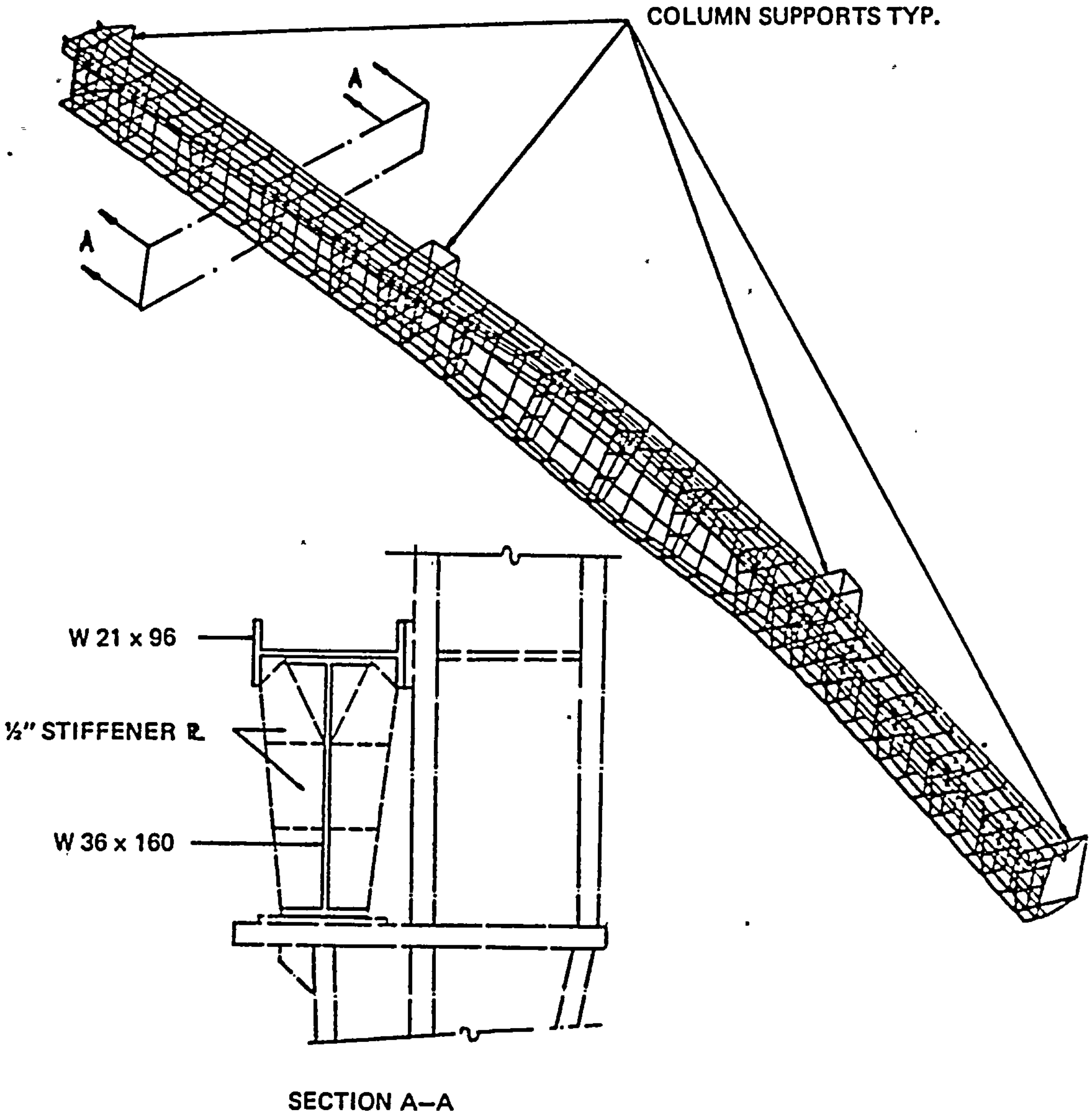


FIGURE 2.1.3-15

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

FUEL HANDLING BUILDING
DETAILED CRANE GIRDER
ANALYSIS MODEL

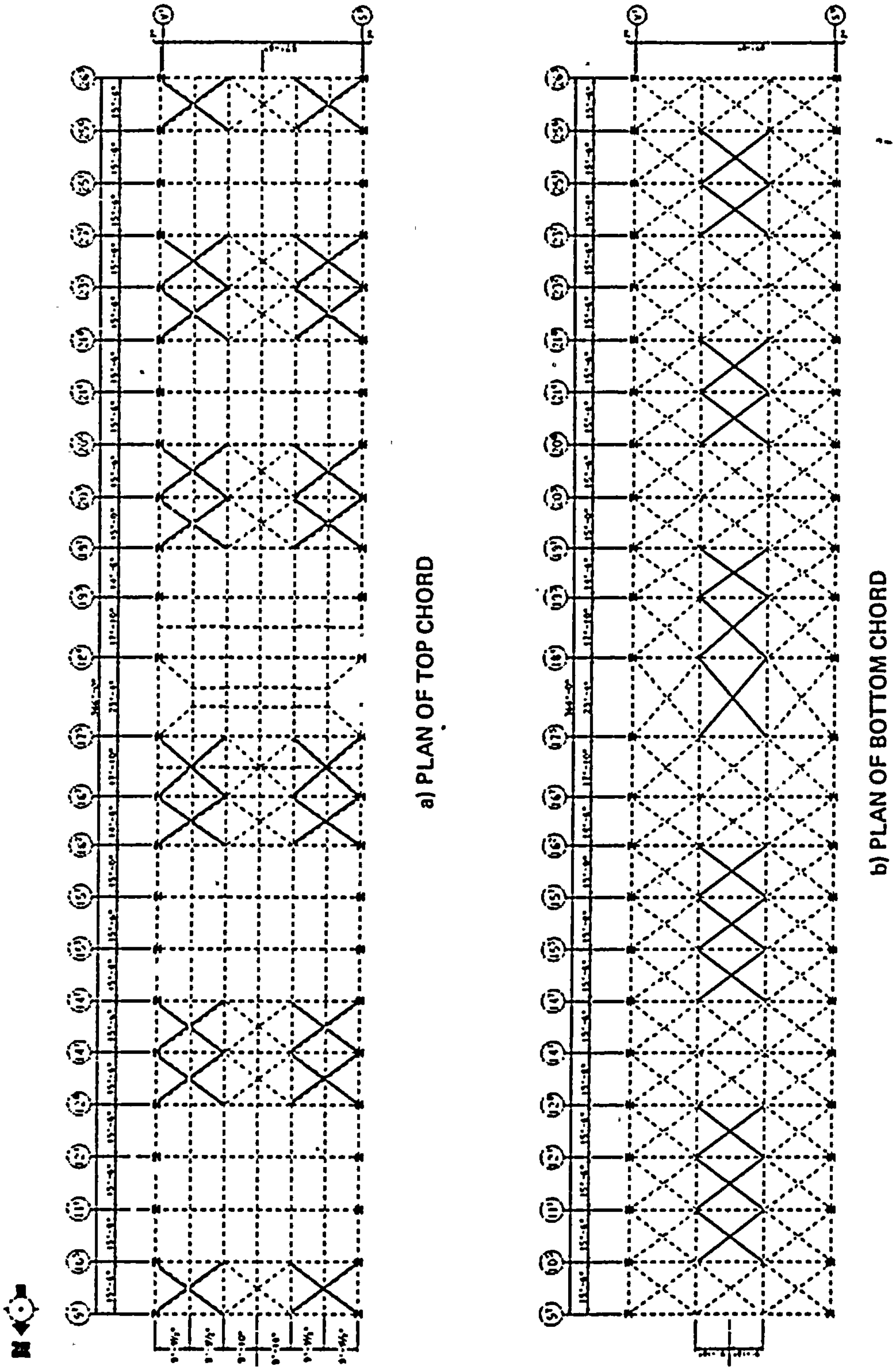


FIGURE 2.1.3-16

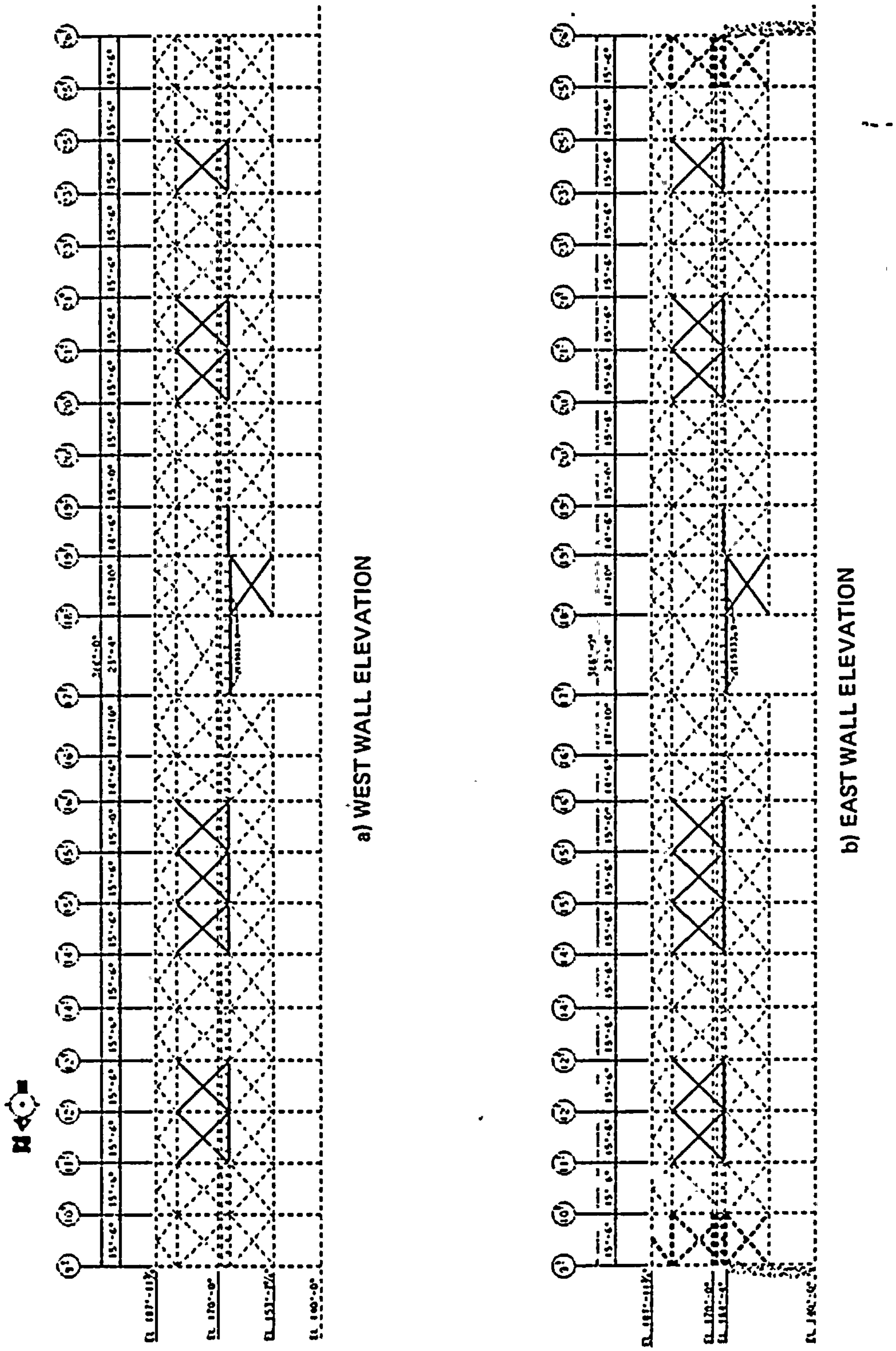
LEGEND:

- - - - EXISTING MEMBERS

——— ADDITION OF MEMBERS

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

FUEL HANDLING BUILDING
PROPOSED MODIFICATION OF
TOP AND BOTTOM DIAGONALS



LEGEND:

- ·········· EXISITING MEMBERS
- ▪▪▪▪▪▪▪▪▪ MEMBERS TO BE REMOVED
- ——— ADDITION OF MEMBERS

FIGURE 2.1.3-17

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

FUEL HANDLING BUILDING
PROPOSED MODIFICATION OF
EAST AND WEST WALL ELEVATIONS

2.1.5 INTAKE STRUCTURE

2.1.5.1 Scope

The objective of the review is to verify that the Design Class 2 intake structure is adequate for the Hosgri event, and to verify the spectra generated for the DE, DDE and Hosgri events which are developed to provide a basis for Design Class 1 equipment qualification. In addition, to address the NRC concern on the wave forces, recent wave force tests and analyses are included with this review.

The seismic review of the intake structure is performed using the criteria given in Sections 4.1 and 4.5 of the Hosgri Report (Reference 2.1.5-1). The intake structure is also reviewed for generation of response spectra for DE and DDE using the criteria given in FSAR Section 3.0 (Reference 2.1.5-2). These criteria and dynamic analysis procedures and methods used to verify the intake structure have been accepted by the NRC as stated in SER Section 3.7 and Supplements 7 and 8 (Reference 2.1.5-3) to the SER.

The seismic analysis and design of the intake structure are reviewed to assure that the models used previously for the Hosgri evaluation and the generation of DE and DDE spectra adequately represent the as-built conditions. Based on this review, and recent additional modifications to the auxiliary saltwater (ASW) vent system being made concurrently with the review, some changes of the previous models were required and the building was reanalyzed. As a result of the reanalysis, new response spectra were developed that may affect the qualification of safety-related equipment,

piping, and components supported by the structure. Reviews of these items are described in subsequent sections. The structural design of the building is reviewed for compliance with the criteria.

2.1.5.2 Criteria

The seismic review used the criteria described in the FSAR and Hosgri Report. These criteria are summarized in Table 2.1.5-1.

The design response spectra at the base of the structure, as given in the Hosgri Report, are shown on Figures 2.1.5-1 through 3. The acceleration time-histories used in the analyses are from the Hosgri Report and the FSAR. Comparisons of the response spectra computed from these time-histories and their design response spectra are shown on Figures 2.1.5-4 through 6.

2.1.5.2.1 Loading Combinations

Combinations of dead load, live load, Hosgri loads, and wave loads are considered below. For each structural member, the combination that produces the maximum stress is used for design. Stated in equation form, the load combinations are:

$$U = DL + L + HE \qquad \text{(Equation 2.1.5-1)}$$

$$U = DL + L + W_f \qquad \text{(Equation 2.1.5-2)}$$

where:

U = Strength required to resist design loads based on the methods summarized in Table 2.1.5-1 for load combinations with seismic forces. For load combinations with wave forces, strength is based on methods used in ACI 318-71 (Reference 2.1.5-4) and AISC, Seventh Edition, Part II (Reference 2.1.5-5).

D = Dead load of structure and equipment loads

L = Live load

HE = Loads due to the Hosgri event

W_f = Wave force associated with breakwater degraded to mean lower low water (MLLW).

2.1.5.3 Methodology

This section describes the intake structure and the methodology used for the DE, DDE, and Hosgri events, and wave force analyses. The analyses include forces and displacements in the structure for the Hosgri event, as well as floor response spectra for DE, DDE, and Hosgri.

2.1.5.3.1 Description of Intake Structure

The seismic Design Class 2 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 ft. The long dimension is assumed in the analysis to correspond to the north-south direction, and is parallel to the seaward face of the structure. The intake

structure is back-filled by rock on three sides, and by water on the fourth (western) side. The top deck of the structure has a maximum elevation of +17.5 ft. A concrete ventilation tower with steel coaxial ventilation pipe extends to an elevation of +49.4 ft. The structure is supported by a concrete mat foundation at elevation -31.5 ft. Figures 2.1.5-7 through 9 illustrate plans at elevations +17.5, -2.1, and -31.5 ft; Figures 2.1.5-10 through 12 illustrate representative sections through the structure.

The top level of the structure consists of an 18-in.-thick concrete slab, except for the roadway area where it is 24 in. thick. Openings, as shown on Figure 2.1.5-7, are provided in order to allow removal of pumps, screens, and gates. The pump deck floor at elevation -2.1 ft supports the four main circulating water pumps and the four seismic Design Class 1 ASW pumps. Design Class 1 ASW equipment is located in ventilated watertight compartments. The structure is symmetric about a vertical plane in the east-west direction through its centerline.

2.1.5.3.2 Description of Seismic Mathematical Model

The same three-dimensional mathematical models discussed in the Hosgri Report and shown in Figures 2.1.5-13 and 14 are used with the following improvements:

(1) The recent structural modifications in the ASW vent system are included.

(2) The nodal degrees of freedom constrained in the previous models are released. In the north-south model, the east-west and vertical translational degrees of freedom are released. In the east-west/vertical model, the north-south translational degree of freedom is released. Therefore, there are six degrees of freedom per node in both models.

2.1.5.3.3 Description of Wave Force Scale Model

A scaled, three-dimensional physical model of the cooling water intake basin, intake structure, and of a hypothetically damaged breakwater was recently constructed by Offshore Technology Corporation to examine wave effects on the intake structure. The test basin has dimensions of 120 x 80 ft with enclosing walls 4 ft high. The model scale is 1:45. This represents an area 3600 x 5400 ft in extent. The bathymetry of the intake basin was modeled to represent all apparent features of the complex topography above a depth of 100 ft below mean lower low water datum. The 1:45 scale model of the seawater intake structure was built independently of the hydraulic model and installed in the test basin. The ASW pump compartments within the intake structure and the control building were also modeled (see Figures 2.1.5-15 and 16).

2.1.5.3.4 Description of Seismic Model Properties

The intake structure is analyzed as a fixed-base model for earthquake motions. Modal damping equal to 7% of critical is used in the Hosgri evaluation and 5% of critical for the generation of DE and DDE spectra.

Values for concrete compressive strength and modulus of elasticity used in the analysis of the intake structure are 3,630 psi and 3.43×10^6 psi for the Hosgri evaluation, and 3,000 psi and 3.12×10^6 psi for DDE and DE, respectively. The Poisson's Ratio used in all analyses is 0.25.

Concrete strength (f'_c) for the Hosgri evaluation was based on the average 28-day strength of 6-in. x 12-in. cylinder samples taken from the concrete used in the construction of the intake structure. Concrete strength used for DE and DDE is the minimum specified strength. The modulus of elasticity of the concrete used, E, is taken as $E = 57,000 \sqrt{f'_c}$ (psi).

The yield strength used for the reinforcing steel in the Hosgri evaluation was 49,600 psi, corresponding to the average of the test values of the reinforcing steel in the intake structure.

The flat-plate elements modeling the intake structure walls used as-built wall thickness.

2.1.5.3.5 Analytical Methods

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 the Hosgri Report.

2.1.5.3.6 Description of Analytical Output

Maximum absolute accelerations, and maximum displacements and stress in the walls are calculated. The translational and vertical acceleration response spectra are computed at selected mass points.

2.1.5.4 Scope of Wave Force Scale Model Test

The wave effects of varying wave height, direction, breakwater configuration, and mean water surface elevation on the intake structure are evaluated. The evaluation considers the following measurements and assessments:

(1) Forces and moments on the ASW ventilation structures

(2) Wave runup and splash potential on the ASW ventilation pipes

(3) Pressures on the seaward side curtain wall

(4) Water velocities in the intake structure bays to the ASW pumps

(5) Pressures beneath the floor of the ASW pump compartments.

2.1.5.5 Results from Wave Force Scale Model Test

The evaluation of the effect of waves on the auxiliary saltwater (ASW) ventilation structure with the breakwater at mean low lower water (MLLW) elevation has been concluded. The results are reported in "The Height Limiting Effect of Sea Floor Terrain Features and of Hypothetically Extensively Reduced Breakwaters on Wave Action at Diablo Canyon Sea Water Intake" by Omar J. Lillevang, Fredric Raichlen, and Jack C. Cox, which was submitted to the NRC on July 1, 1982.

Additional tests have been completed which address pressures on the seaward curtain wall, water velocities in the intake bays to the ASW pumps, and pressures beneath the floor of the ASW pump compartment. Tests to further study the wave runup and splash potential on the ASW ventilation pipes were also conducted. Test results indicate that there are no slam pressures (high magnitude, high frequency pressures) on the outside face of the seaward curtain wall. There would be no slam pressures against the inside face of the seaward curtain, if the top deck slab was modified; nor would there be slam pressures beneath the floor of the ASW pump compartments, if the manhole covers remained in place. The top deck slab and the manholes will be modified to prevent slam pressures. With these modifications, the pressures on the front wall and on the slab of the ASW compartment are below allowable pressures. The tests also indicate that debris cannot be carried into the ASW intake bays because the velocity is very low, and water ingestion into the ventilation pipes is not a problem.

2.1.5.6 Analysis of Structure Subjected to Wave Force

The tests indicate that with the breakwater degraded to mean lower low water the Class 1 auxiliary saltwater system will be protected under the extreme wave events outlined in the July 1, 1982, submittal (Reference 2.1.5-7).

2.1.5.7 Design Review and Qualification of Structure

2.1.5.7.1 Review Procedure

The following procedure is followed in the review process:

(1) As-built Comparison With Design Drawings
Contractor-generated, as-built concrete lift drawings are compared with the latest design drawings from which the model is generated.

(2) Comparison of Design Drawings and Criteria with Seismic Models - The seismic models are compared with the design drawings to assure that the main structural elements and openings are modeled to provide an accurate seismic representation of the actual structure. Thickness, location and extent of structural elements, material properties, and boundary conditions are reviewed. The model revisions to the ASW compartment modification are also reviewed. The effective mass of water and equipment is checked.

(3) Review of Analysis

The analysis is also reviewed for compliance with criteria.

(4) Review of Output and Design

The computer output and the application of output for the analysis of structural members are reviewed.

The factors of safety against overturning and sliding, maximum allowable foundation bearing pressure, and the perimeter walls, slabs, piers, columns, as well as interior walls are reviewed for compliance with criteria.

(5) Resolution of Findings

Studies are performed where review indicates that an item may significantly affect stresses, displacements or spectra. Studies include hand calculations and computer analyses.

2.1.5.7.2 Review Results

Detailed review of seismic analysis and verification of structural members indicated that the intake structure is in compliance with the criteria. The stresses resulting from the combination of north-south, east-west, and vertical components of the Hosgri event, in conjunction with the stresses resulting from dead loads, actual live loads, and soil pressures, are within the capacity of the major portion of the structure, including the area housing the Class 1 auxiliary saltwater pumps. The only exceptions are some of the

flow straighteners which exhibit stresses beyond code values, as allowed in the Hosgri Report (Reference 2.1.5-1). However, these piers demonstrate ductility properties that would preclude structural failure of any kind and would allow only minor cracking at the base of the pier. Tables 2.1.5-2 through 2.1.5-7 present the results of the latest analysis.

The following is a list of items that are studied for possible influence:

(1) The mass of the bar racks, crane, traveling screens, and gates were assumed to have no significant effects on the seismic analysis. Detailed studies demonstrate that these masses do not significantly impact building response spectra, thus substantiating the validity of the above assumptions.

(2) The bottom of the seaward and gate walls are modeled at a higher elevation than as-built. In order to evaluate the effect of this assumption and its possible impact on the flow straightener and wall stresses, a north-south response spectra dynamic modal analysis is performed with the bottom of the walls at the as-built elevation and all the equipment and crane masses included. The bending moments and ductility values for the flow straighteners shown in Tables 2.1.5-6 and 2.1.5-7 are based on this analysis. Since the stresses in other walls are within allowable, and the change in stresses from this analysis is small, the stresses in these walls are not recalculated.

(3) Static and dynamic lateral earth pressures on the east wall of the intake structure are considered in the calculation of the in-plane shear stress for the east-west walls. The earth pressure influence is combined by the square root sum of the squares (SRSS) method with the seismic force from the dynamic analysis. The shear stresses in Table 2.1.5-5 include the influence of both lateral earth pressures and seismic response from the structure.

(4) Calculations using simplified assumptions are done to check the rigidity of the top deck slab at the locations of Class 1 conduits. The study concluded that for seismic response, slab elements are rigid.

(5) The tsunami at the intake structure would be in the form of waves with periods greater than 13 minutes. The effect on the seaward wall is like a fast tide, and the force is hydrostatic. Since there is water on both sides of the seaward wall, the net force on the wall and flow straighteners is zero. Therefore the tsunami does not have any bearing on the design of the flow straighteners.

2.1.5.7.3 Response Spectra

The review of the Hosgri seismic model as described above established that the models are adequate representations of the as-built conditions. These models were used to generate building response spectra for the Hosgri event.

DE and DDE spectra are generated for the north-south direction using a modified Hosgri model. The model is modified for damping and material properties consistent with the DE and DDE criteria and also to incorporate the items identified in the Hosgri evaluation. The predominant frequencies of the structure in the east-west direction are higher than 20 Hz; therefore the structure is considered rigid in that direction. This is consistent with the criteria stated in the FSAR (Reference 2.1.5-2).

2.1.5.9 References

2.1.5-1 Pacific Gas and Electric Company, et al, "Seismic Evaluation for Postulated 7.5M Hosgri Earthquake," PGandE, San Francisco, CA (November 1977 with amendments).

2.1.5-2 Pacific Gas and Electric Company, "Final Safety Analysis Report," PGandE, San Francisco, CA (1974).

2.1.5-3 Nuclear Regulatory Commission, "Safety Evaluation Report," Section 3.7 and Supplements 7 and 8, NRC, Washington D.C. (1978).

2.1.5-4 American Concrete Institute, "Standard Building Code Requirements for Reinforced Concrete," ACI 318-71 (1971).

2.1.5-5 American Institute of Steel Construction, "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings," AISC, New York, NY (Seventh Edition).

2.1.5-6 Seismology Committee Structural Engineers Association of California, "Recommended Lateral Force Requirements and Commentary," SEAOC (1974).

2.1.5-7 Letter from Philip A. Crane, Jr. (PGandE) to Frank J. Miraglia (NRC), July 1, 1982, "Pacific Gas and Electric Company's Interim Report on its Investigation of Breakwater Damage at Diablo Canyon."

TABLE 2.1.5-1

INTAKE STRUCTURE

SUMMARY OF CRITERIA FROM HOSGRI REPORT AND FSAR

Parameters	Hosgri	DE and DDE for Systems Qualifications
Seismic input, horizontal	Hosgri 7.5M	DE (0.20g) DDE (0.40g)
Seismic input, vertical	2/3 of 7% damped horizontal spectra with Tau = 0.0	Not applicable
Vertical analysis	Dynamic amplification considered	Not applicable
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	Allowed in some areas	Not applicable
Material properties	Based on test values	Minimum specified values
Response spectra broadening (based on frequency)	+5%, -15%	Structural peaks clipped 10% and widened by ±10%
Allowable stresses	SEAOC (1974) for concrete shear walls and ACI 318-71 for other concrete members AISC Part II, 7th Edition	Not applicable

TABLE 2.1.5-2

PERIODS OF VIBRATION AND PARTICIPATION FACTORS

	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
1	0.092	22.6	0.083	0.0	0.2
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
8	0.072	10.2	0.071	8.0	1.9
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
16	0.063	2.2	0.063	0.2	15.7
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
20	0.050	8.6	0.050	3.1	1.6
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
26	0.042	3.6	0.043	40.1	20.2
27	0.041	0.8	0.041	4.8	4.5
28	0.040	0.4	0.040	0.9	0.5
29	0.040	0.9	0.040	0.1	0.1

TABLE 2.1.5-3

MAXIMUM DISPLACEMENTS

Nodal Point*	Elevation (ft)	North-South Displacement (in.) Blume-Hosgri	North-South Displacement (in.) Newmark-Hosgri	East-West Displacement (in.) Blume-Hosgri	East-West Displacement (in.) Newmark-Hosgri	Vertical Displacement (in.) Blume-Hosgri	Vertical Displacement (in.) Newmark-Hosgri
330	+32.0	0.025	0.024	0.042	0.044	0.009	0.010
312	+24.4	0.016	0.016	0.029	0.029	0.008	0.008
71	+17.5	0.115	0.120	0.011	0.011	0.010	0.010
72	+17.5	0.111	0.116	0.011	0.011	0.009	0.009
73	+17.5	0.063	0.065	0.011	0.011	0.007	0.007
74	+17.5	0.055	0.058	0.011	0.011	0.005	0.005
75	+17.5	0.040	0.042	0.011	0.011	0.002	0.002
76	+17.5	0.015	0.016	0.012	0.012	0.002	0.002
209	+17.5	0.014	0.014	0.016	0.016	0.007	0.007
284	+17.5	0.009	0.008	0.019	0.019	0.007	0.007
363	+11.0	0.008	0.008	0.012	0.012	0.003	0.003
80	-2.1	0.127	0.133	0.007	0.006	0.010	0.010
81	-2.1	0.100	0.104	0.006	0.006	0.007	0.007
82	-2.1	0.086	0.089	0.006	0.005	0.006	0.006
83	-2.1	0.063	0.066	0.005	0.005	0.006	0.006
84	-2.1	0.009	0.009	0.002	0.002	0.001	0.001
87	-16.8	0.241	0.251	0.003	0.003	0.005	0.005
88	-16.8	0.209	0.218	0.003	0.003	0.004	0.004
89	-16.8	0.048	0.050	0.002	0.002	0.003	0.003
90	-16.8	0.012	0.012	0.001	0.001	0.001	0.001
91	-16.8	0.013	0.013	0.002	0.002	0.001	0.001

*See Figure 2.1.5-14

TABLE 2.1.5-4

MAXIMUM ABSOLUTE ACCELERATIONS

Nodal Point*	Elevation (ft)	North-South Acceleration (g)		East-West Acceleration (g)		Vertical Acceleration (g)	
		Blume-Hosgri	Newmark-Hosgri	Blume-Hosgri	Newmark-Hosgri	Blume-Hosgri	Newmark-Hosgri
330	+32.0	2.36	2.18	2.15	2.13	0.65	0.65
312	+24.4	1.50	1.39	1.55	1.55	0.65	0.64
71	+17.5	1.52	1.58	0.66	0.64	0.61	0.61
72	+17.5	1.49	1.54	0.66	0.64	0.58	0.58
73	+17.5	0.92	0.94	0.66	0.64	0.54	0.54
74	+17.5	0.83	0.85	0.66	0.65	0.51	0.51
75	+17.5	0.54	0.56	0.66	0.65	0.50	0.50
76	+17.5	0.56	0.53	0.66	0.65	0.53	0.53
209	+17.5	0.61	0.57	0.88	0.88	0.64	0.64
284	+17.5	0.64	0.60	1.00	1.00	0.62	0.62
363	+11.0	0.64	0.59	0.87	0.85	0.54	0.54
80	-2.1	1.64	1.71	0.70	0.65	0.59	0.59
81	-2.1	1.40	1.44	0.70	0.65	0.53	0.53
82	-2.1	1.24	1.27	0.70	0.65	0.53	0.53
83	-2.1	0.95	0.98	0.71	0.65	0.52	0.52
84	-2.1	0.68	0.63	0.73	0.66	0.50	0.50
87	-16.8	3.00	3.14	0.72	0.66	0.51	0.52
88	-16.8	2.68	2.80	0.73	0.66	0.50	0.50
89	-16.8	1.07	1.12	0.73	0.66	0.50	0.50
90	-16.8	0.63	0.64	0.74	0.66	0.50	0.50
91	-16.8	0.75	0.75	0.73	0.66	0.50	0.50

*See Figure 2.1.5-14

TABLE 2.1.5-5

MAXIMUM SHEAR STRESSES

Element	Shear Stress (psi) Blume-Hosgri	Newmark-Hosgri	Allowable
Upper Walls			
Seaward Wall	64	66	394
Gate Guide Wall	59	61	248
Screen Wall	103	106	394
Auxiliary Pump Wall	105	105	408
East Wall	26	25	298
North Wall	127	128	298
Flow Straighteners	41	42	372
Lower Walls			
Center Pier	261	270	394
End Wall	49	51	248
Flow Straighteners	47	48	406
Upper Slab (Roof)	207	208	408
Lower Slabs			
Slab at Center Pier	251	261	394
North Slab	178	194	298

TABLE 2.1.5-6

MAXIMUM BENDING MOMENTS OF FLOW STRAIGHTENERS

Pier Number*	Bending Moment (kip-in./in.) Blume-Hosgri	Newmark-Hosgri	Code Value
1	106	111	128
2	144	150	128
3	157	165	128
4	165	173	128
5	174	182	128
6	177	186	128
7	164	171	128

*See Figure 2.1.5-9

TABLE 2.1.5-7

DUCTILITY RATIOS OF FLOW STRAIGHTENERS

	Blume-Hosgri		Newmark-Hosgri	
Pier Number*	Displacement Ductility	Curvature Ductility	Displacement Ductility	Curvature Ductility
1				
2	1.04	1.07	1.09	1.17
3	1.15	1.28	1.21	1.39
4	1.22	1.41	1.29	1.53
5	1.29	1.53	1.37	1.68
6	1.33	1.61	1.40	1.74
7	1.20	1.37	1.27	1.50

*See Figure 2.1.5-9

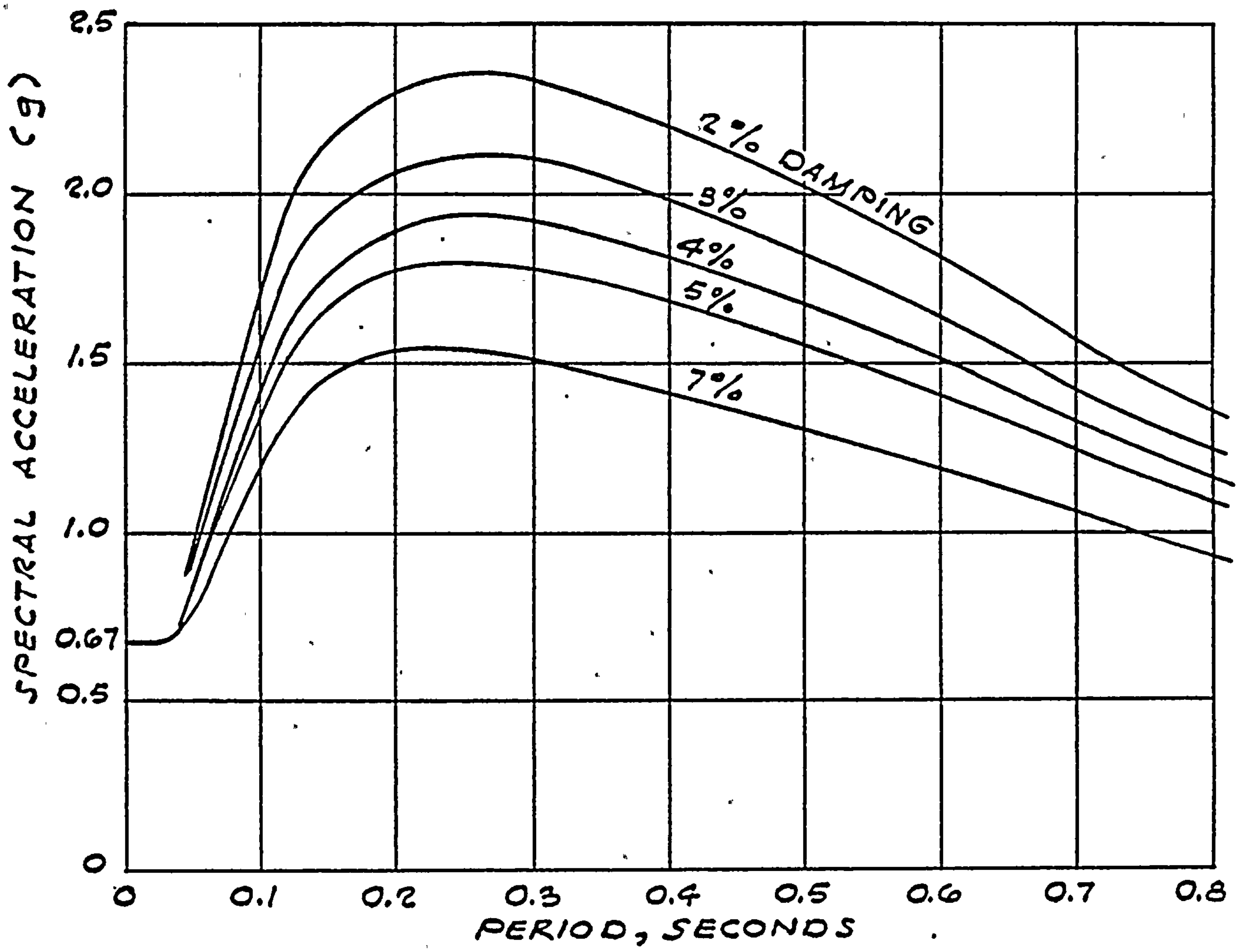


FIGURE 2.1.5-1

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE HORIZONTAL
SPECTRA-BLUME 7.5 HOSGRI
$\tau = 0.04$

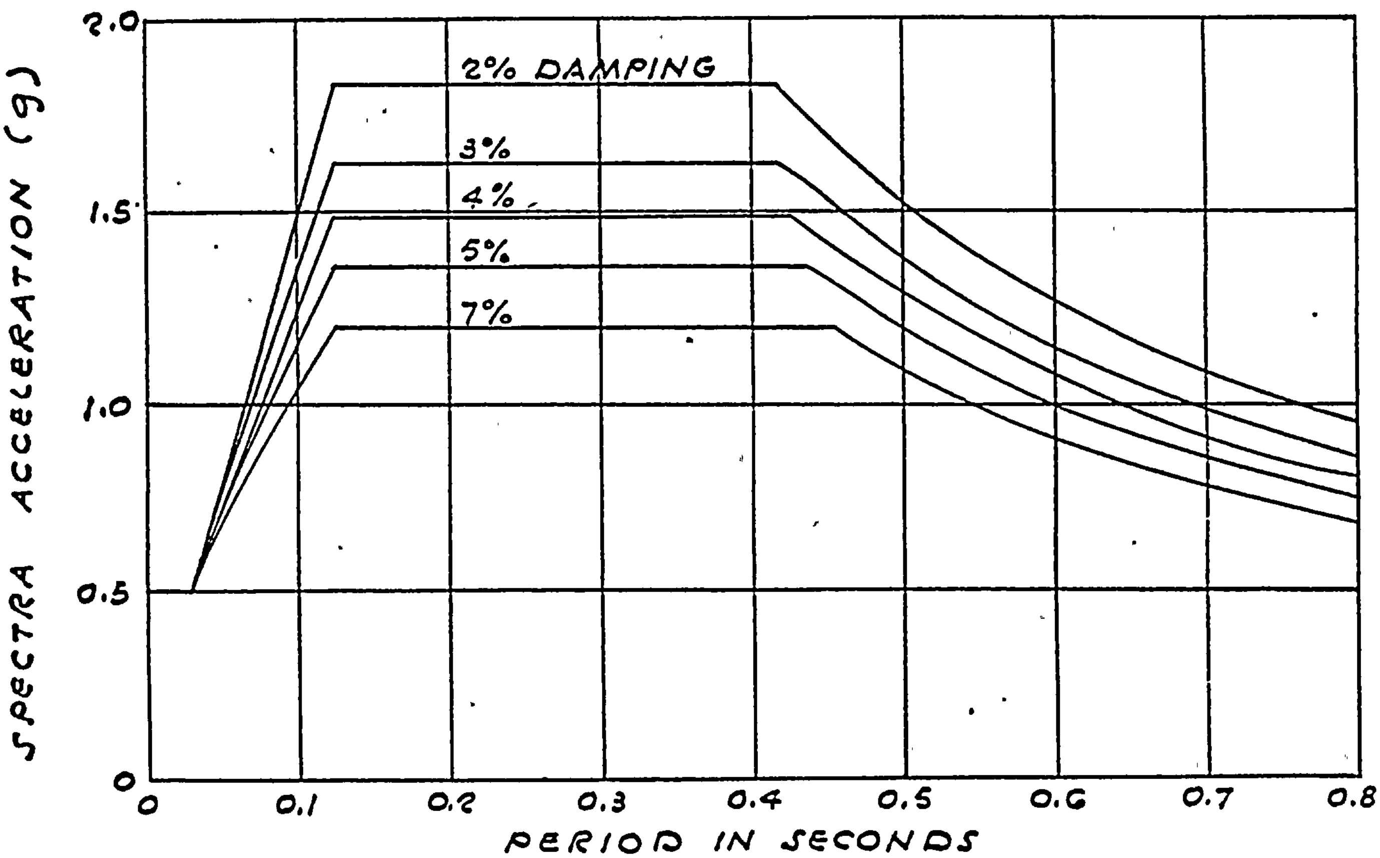


FIGURE 2.1.5-2

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE VERTICAL
SPECTRA-NEWMARK 7.5M HOSGRI

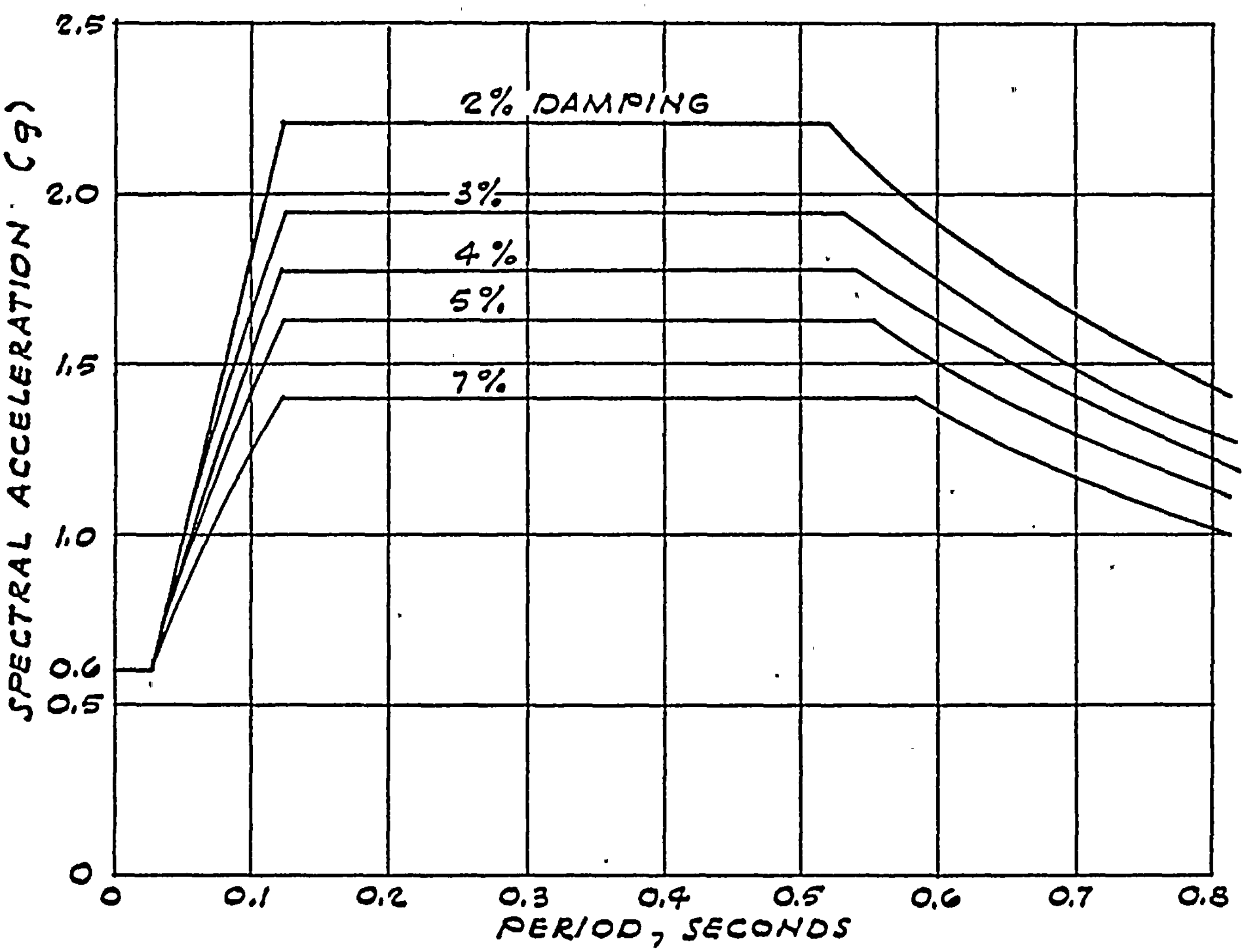


FIGURE 2.1.5-3

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE HORIZONTAL
SPECTRA-NEWMARK & HOSGRI 7.5M
$\tau = .04$

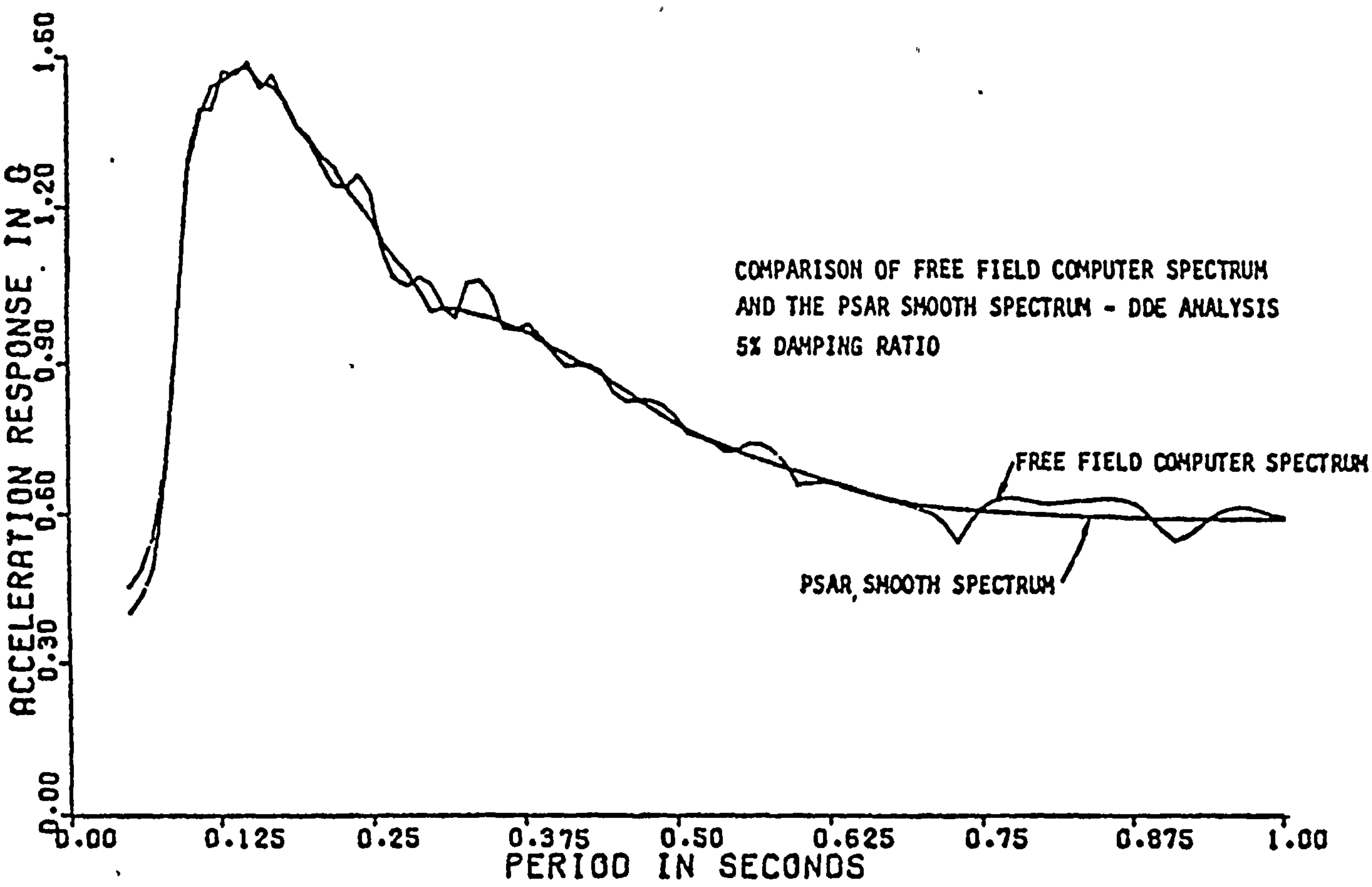


FIGURE 2.1.5-4

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

COMPARISON OF SPECTRA
DDE ANALYSIS

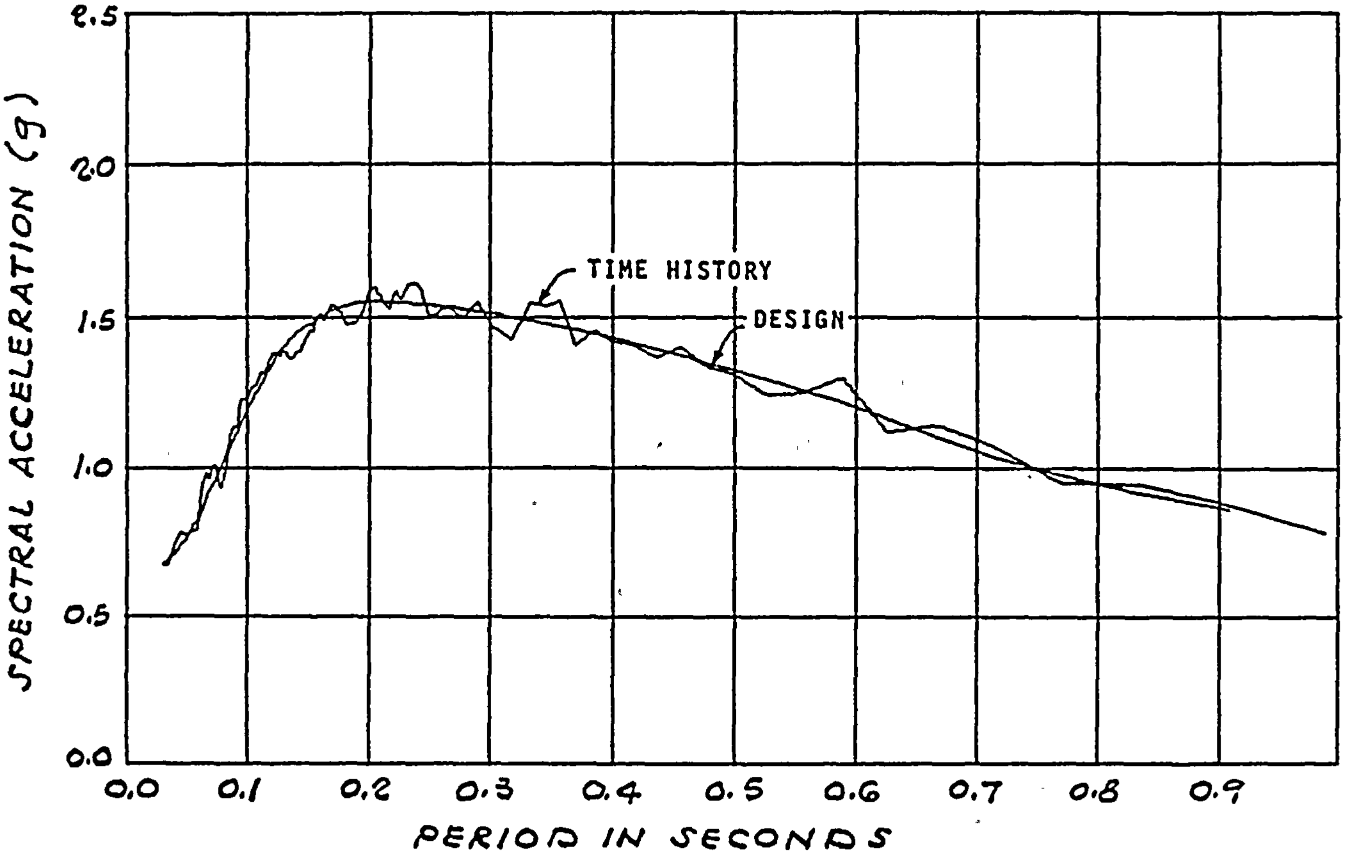


FIGURE 2.1.5-5

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

COMPARISON OF BLUME 7.5 M HOSGRI TIME HISTORY AND SMOOTH DESIGN SPECTRA FOR INTAKE STRUCTURES 7% DAMPING, $\tau = 0.04$

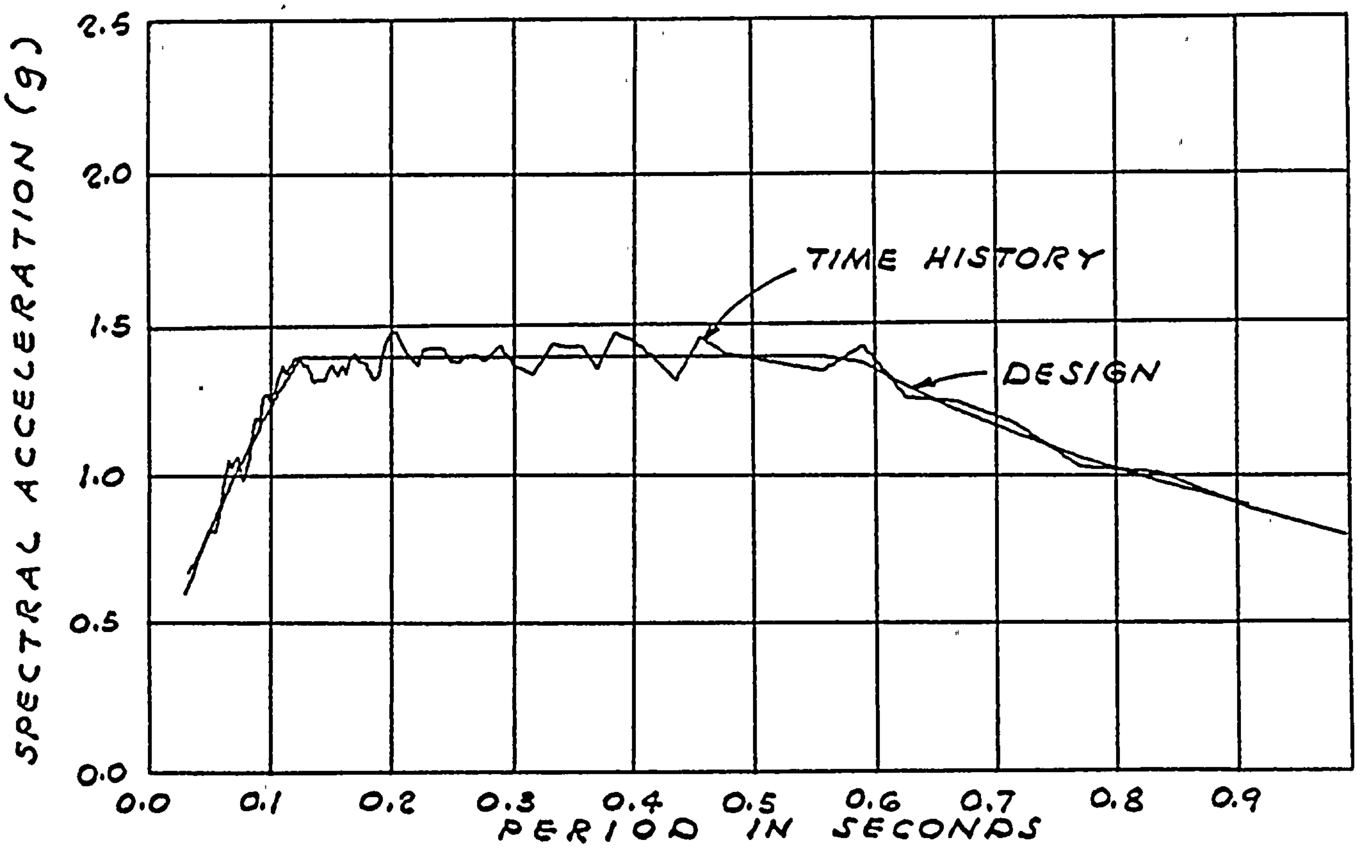


FIGURE 2.1.5-6

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

COMPARISON OF NEWMARK 7.5M HOSGRI TIME HISTORY AND SMOOTH DESIGN SPECTRA FOR INTAKE STRUCTURES 7% DAMPING , τ = 0.04

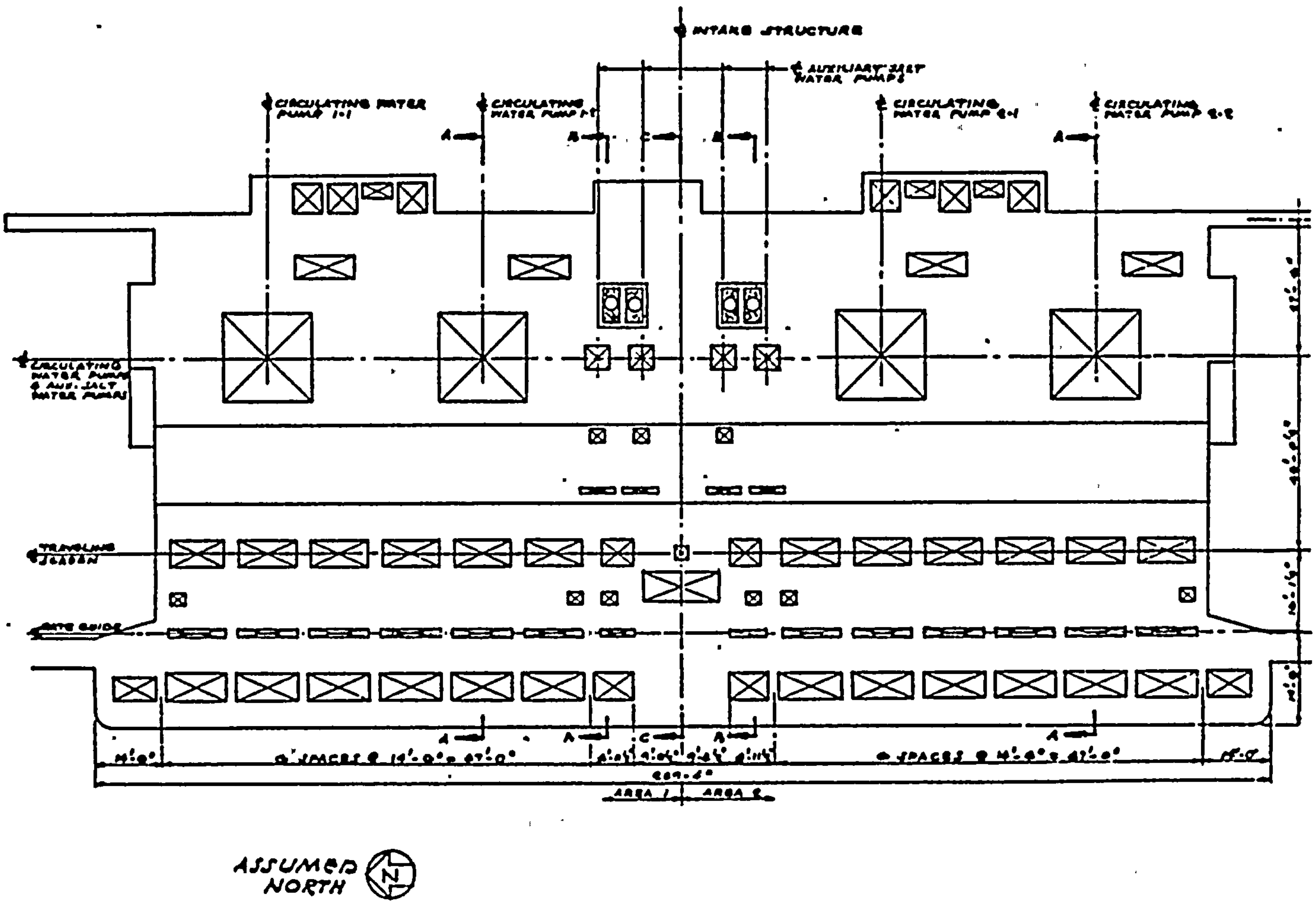


FIGURE 2.1.5-7

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE
PLAN AT TOP DECK, EL +17.5 FT

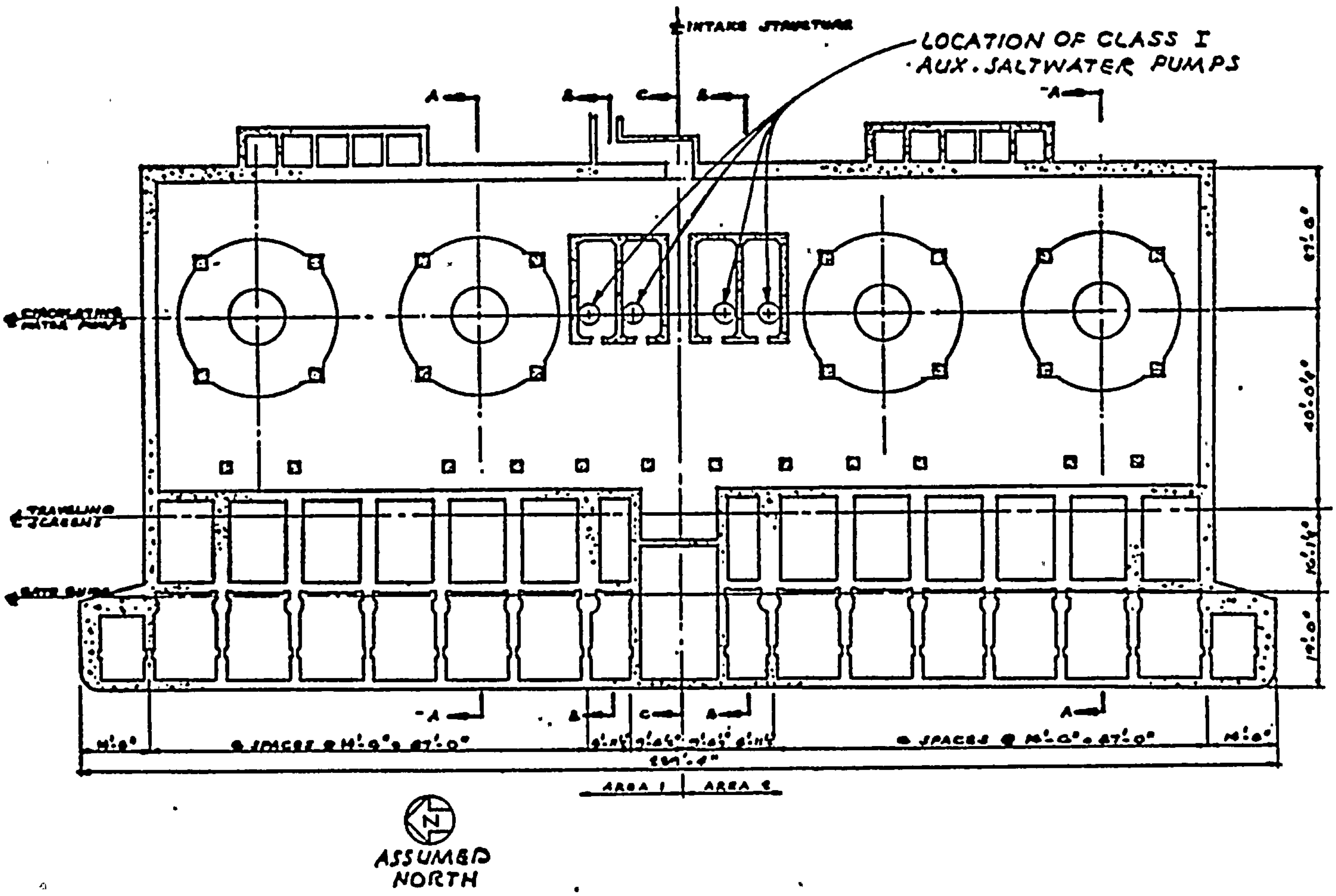


FIGURE 2.1.5-8

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE
PLAN AT PUMP DECK,
EL –2.1 FT

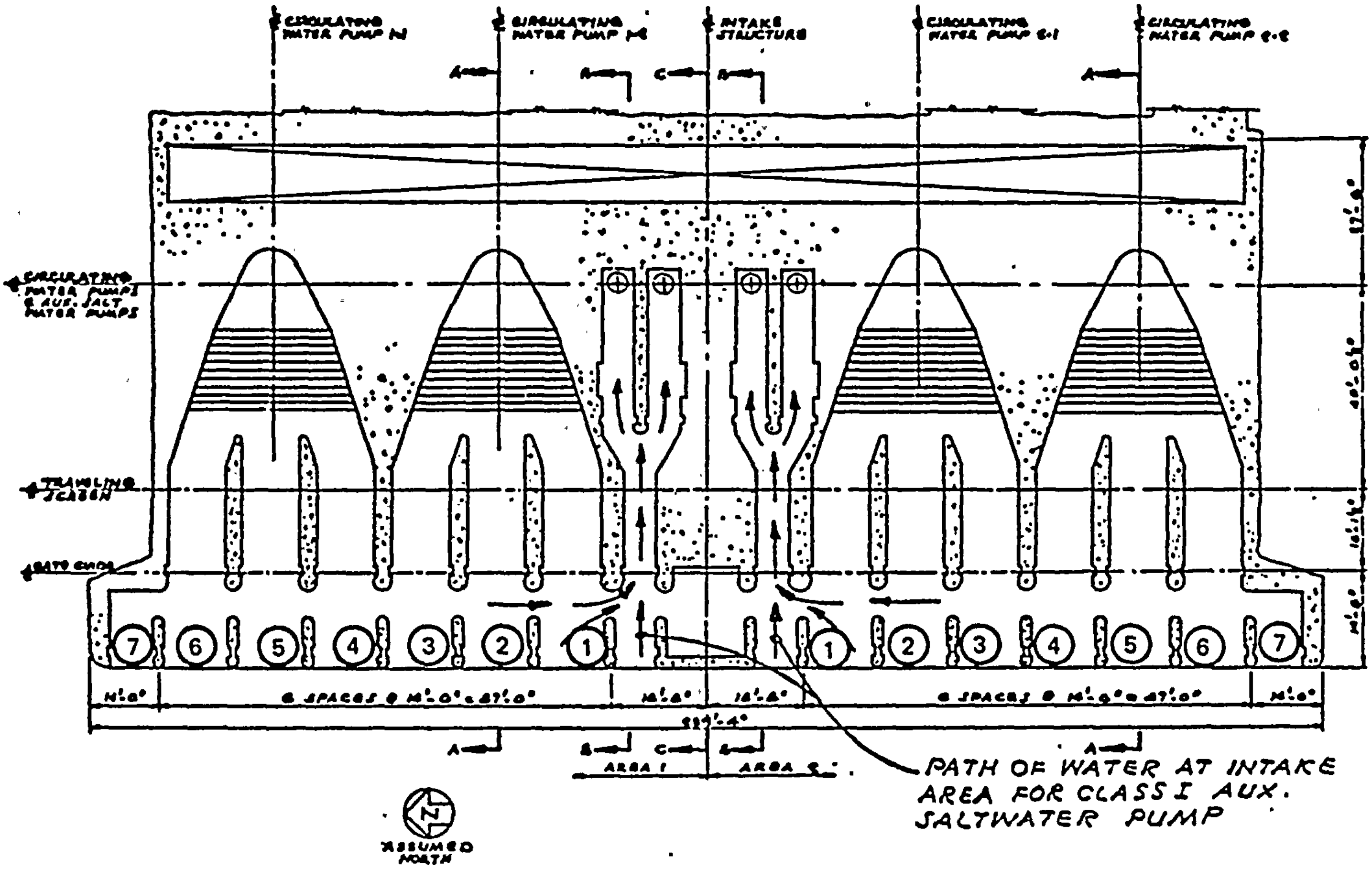


FIGURE 2.1.5-9

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE
PLAN AT INVERT AREA, EL –31.5 FT

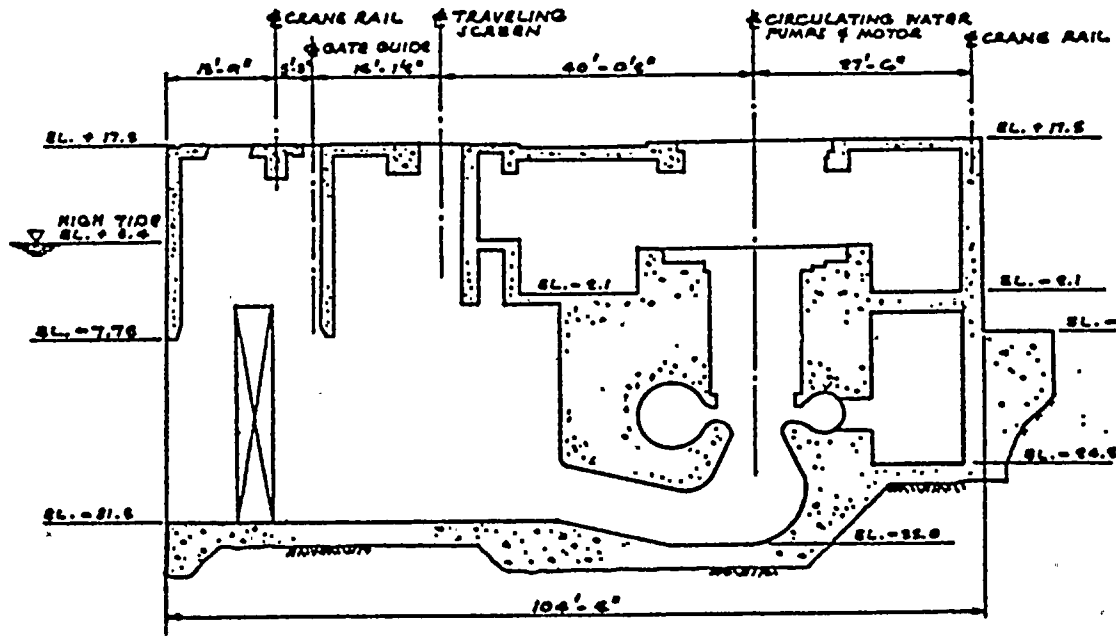


FIGURE 2.1.5-10

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE
TRANSVERSE SECTION A

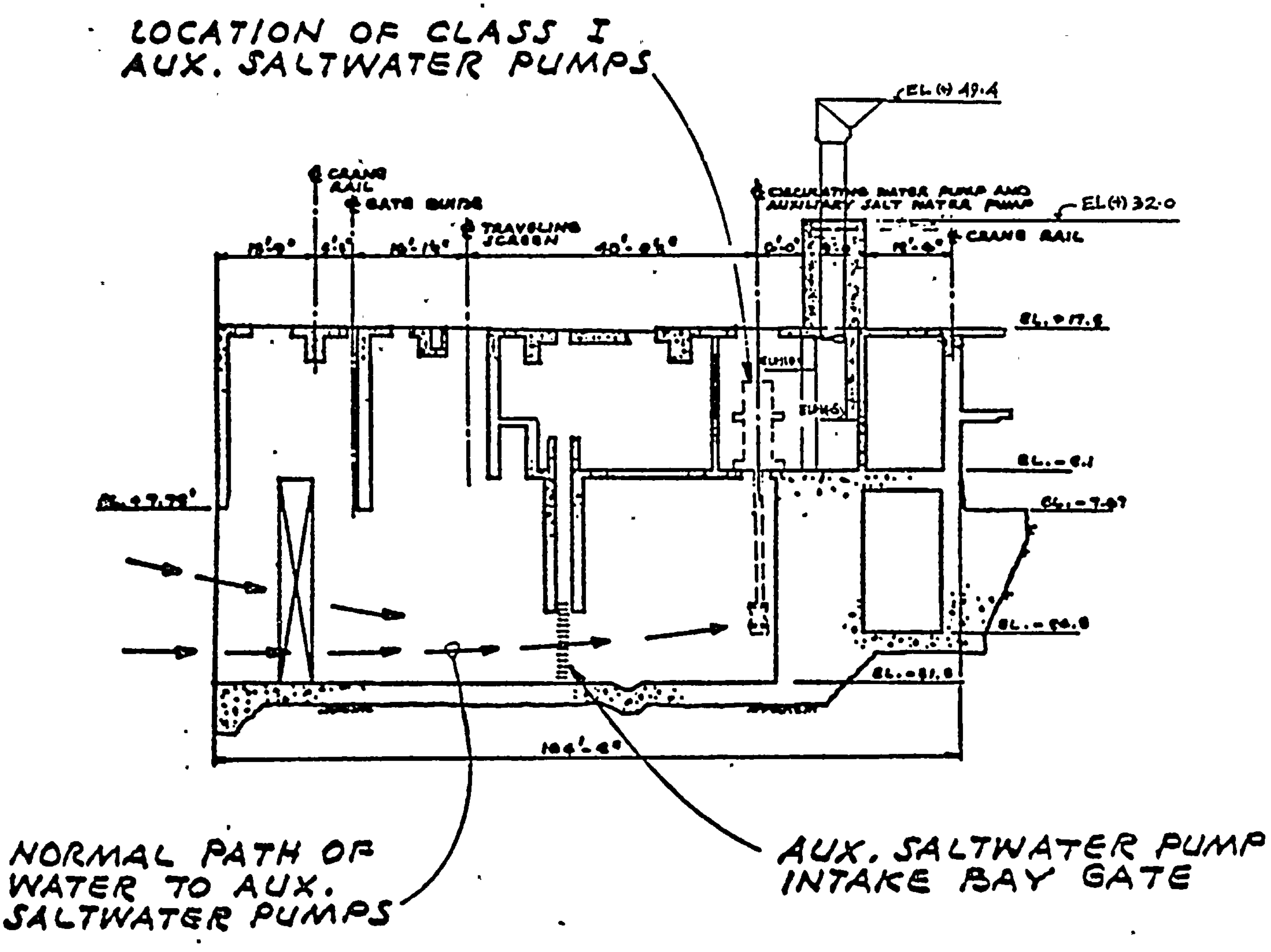


FIGURE 2.1.5-11

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE
TRANSVERSE SECTION B

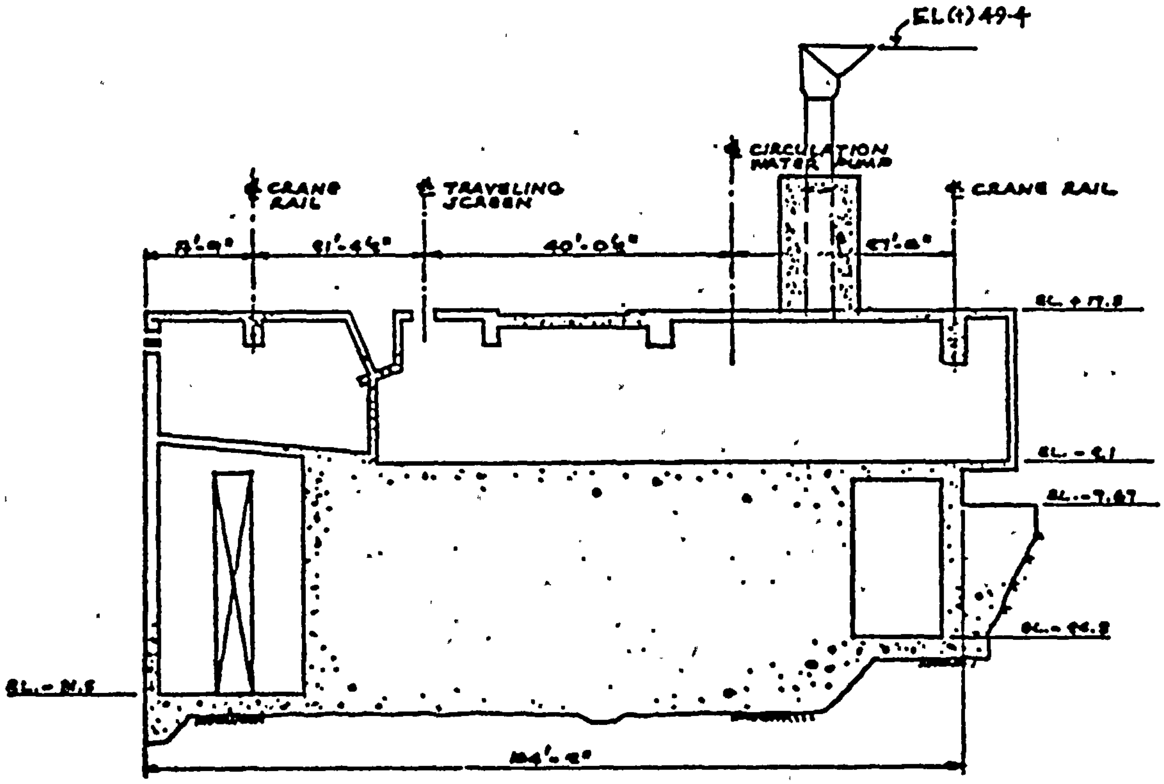


FIGURE 2.1.5-12

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE
TRANSVERSE SECTION C

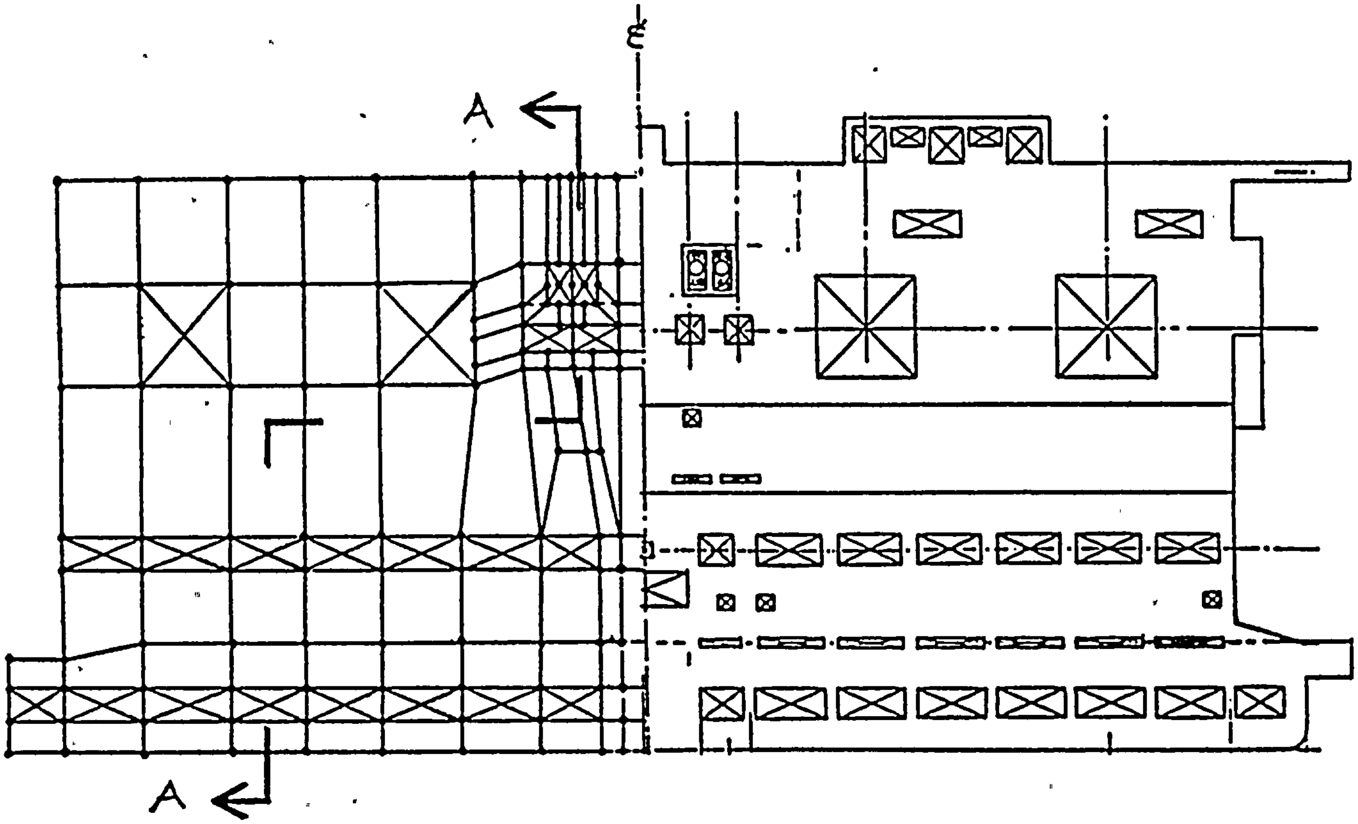
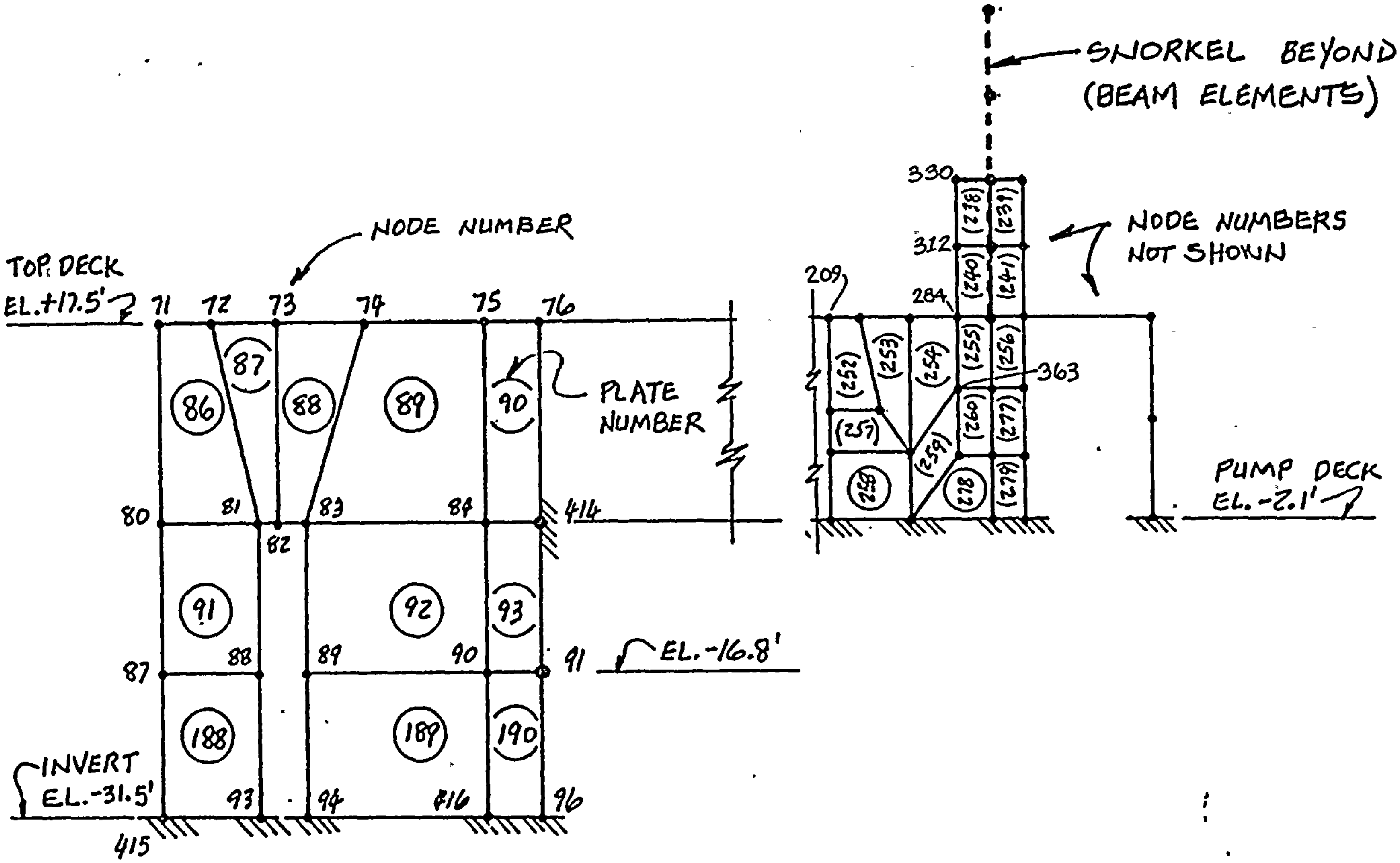


FIGURE 2.1.5-13

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE
TOP DECK MATHEMATICAL MODEL,
EL +17.5 FT



INTAKE STRUCTURE
TRANSVERSE SECTION
MATHEMATICAL MODEL

DIABLO CANYON POWER PLANT
UNITS 1 & 2

FIGURE 2.1.5-14

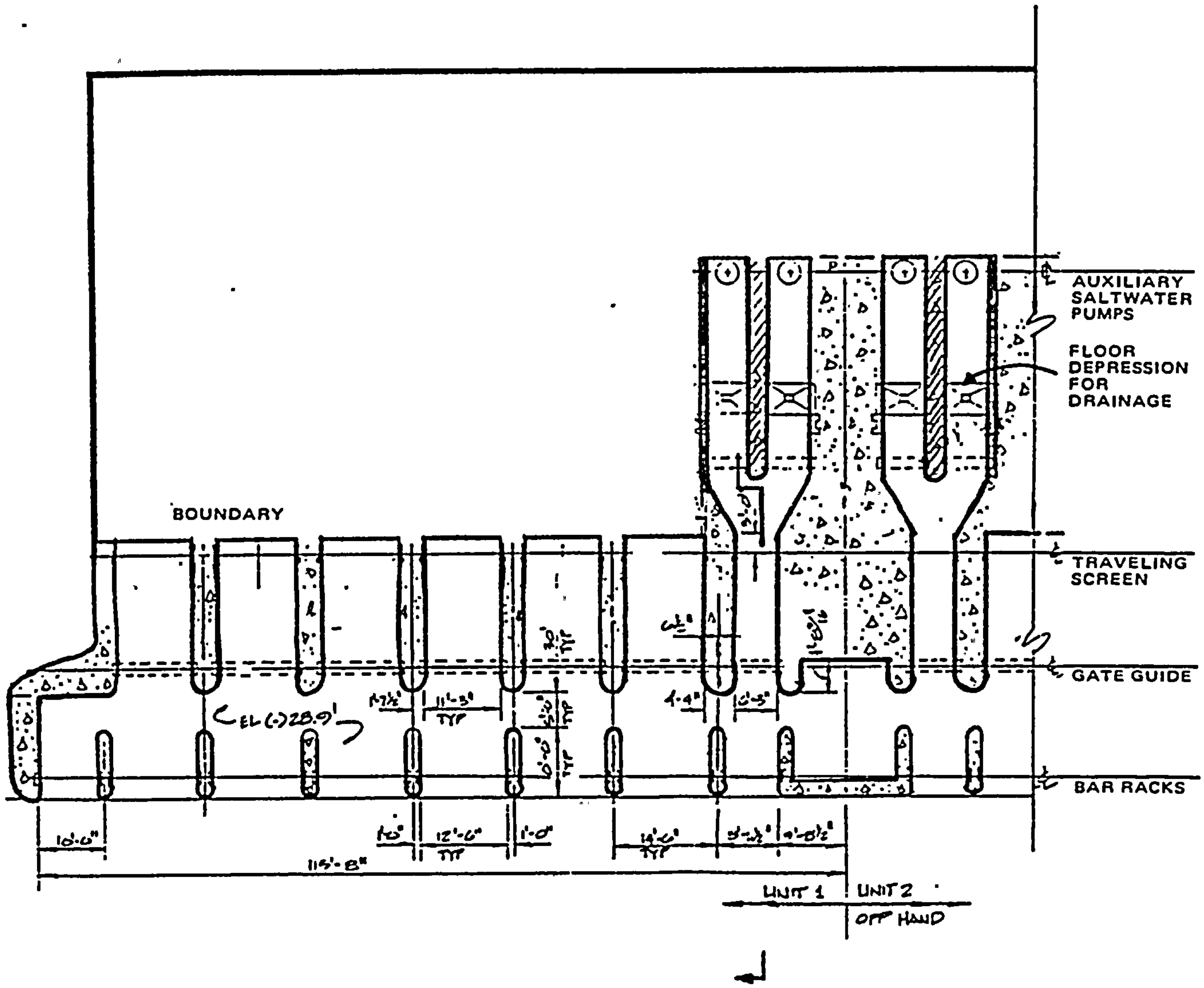


FIGURE 2.1.5-15

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE
PLAN-INVERT (UNIT 1)
NTS

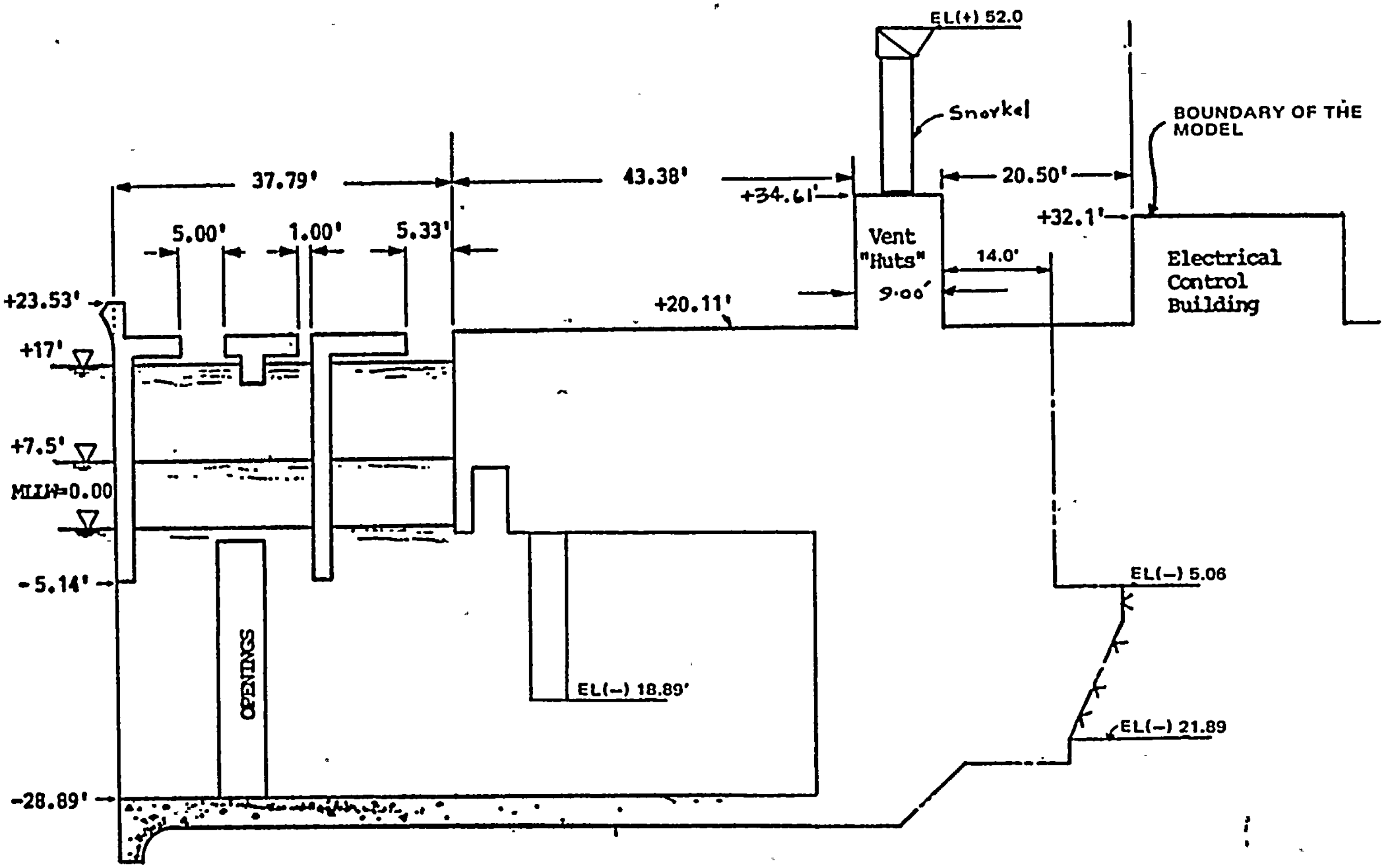


FIGURE 2.1.5-16

DIABLO CANYON POWER PLANT
UNITS 1 & 2

INTAKE STRUCTURE
WAVE SCALE MODEL
TRANSVERSE SECTION D
(All Elevations Refer to Mean Lower Low Water Datum)

2.3.1.4 Results

Each of the 24 equipment items provided by PGandE, which are listed in Tables 7-5, 7-5A, and 7-6 of the Hosgri Report, have been reviewed for compliance with the requirements of the Hosgri Report when subjected to the DCM-C-17, Revision 1, Hosgri loadings (see Table 2.3.1.4-1). The other listed equipment is provided and qualified by Westinghouse. PGandE has met with Westinghouse to discuss a program that it has been actively pursuing, for verifying seismic qualification of equipment. Westinghouse has current, controlled copies of DCM-C17, C25 and C30 (Hosgri, DE, and DDE), which contain all of the latest spectra. The Westinghouse program is scheduled to be completed by November 30, 1982. The list in Table 2.3.1.4-1 includes Class 1 equipment identified to date and indicates the qualification status of equipment with respect to the DE and DDE loadings.

Of the 24 items checked in Table 2.3.1.4-1, 14 had spectra changes. Of these 14 items, four had spectra with lower g levels, three items were qualified to high enough g levels to still be qualified with the new higher spectra, three items have been reanalyzed, and four are to be reanalyzed. As yet, no physical changes to the equipment and supports have been required.

TABLE 2.3.1.4-1

DIABLO CANYON UNIT 1
CLASS 1 EQUIPMENT SEISMIC QUALIFICATION STATUS

COMPONENT		DCM C-17 SPECTRA Rev 0			DCM C-17 SPECTRA Rev 1		
			Not Identical to Hosgri Report Spectra — Reanalysis Required?			Not Identical to Rev 0 Spectra — Reanalysis Required?	
Identification	Elev/Bldg	Identical To Hosgri Report Spectra	No (State Reason)	Yes (Give Rev No. & Date)	Identical To Rev 0 Spectra	No (State Reason)	Yes (Give Rev No. & Date)
Diesel-Generators	85'/TB	✓			✓		
Diesel-Generators Starting Air Receivers	85'/TB	✓			✓		
Diesel-Generators Fuel Oil Filter	77'/MSS	✓			✓		
Diesel-Generators Fuel Oil Premium Tank	85'/TB	✓			✓		
Diesel-Generators Fuel Oil Strainer	77'/TB	✓			✓		
Diesel-Generators Fuel Oil Transfer Pump	77'/MSS	✓			✓		
Component Cooling Water Heat Exchanger	85'/TB	✓			✓		
Auxiliary Saltwater Pumps	-2'/IS	✓			✓		

Aux = Auxiliary Building
IS = Intake Structure
TB = Turbine Building
MSS = Misc. Small Structures

TABLE 2.3.1.4-1 (cont'd)

COMPONENT		DCM C-17 SPECTRA Rev 0			DCM C-17 SPECTRA Rev 1		
			Not Identical to Hosgri Report Spectra			Not Identical to Rev 0 Spectra	
			Reanalysis Required?			Reanalysis Required?	
Identification	Elev/Bldg	Identical To Hosgri Report Spectra	No (State Reason)	Yes (Give Rev No. & Date)	Identical To Rev 0 Spectra	No (State Reason)	Yes (Give Rev No. & Date)
Auxiliary Feedwater Pump (Turbine-driven)	100'/Aux			See Calculation M-145 (7-16-82)	✓		
Auxiliary Feedwater Pump Turbine	100'/Aux		Acceleration reduced 0.05 g		✓		
Auxiliary Feedwater Pump (Motor-driven)	100'/Aux		Acceleration reduced 0.01 g		✓		
Auxiliary Feedwater Pump Motor	100'/Aux		Acceleration reduced 0.05 g		✓		
Component Cooling Water Pumps	73'/Aux	✓			✓		
Boric Acid Tanks	115'/Aux			See Calculation M-146 (7-16-82)	✓		
Fire Pump	115'/Aux			See Calculation M-143 Rev 1 (7-8-82)	✓		
Fire Pump Motor Pumps	115'/Aux			Analysis in Progress	✓		

Aux = Auxiliary building   IS = Intake structure   CS = Containment structure
TB = Turbine building   MSS = Misc small structures

TABLE 2.3.1.4-1 (cont'd)

COMPONENT		DCM C-17 SPECTRA Rev 0			DCM C-17 SPECTRA Rev 1		
			Not Identical to Hosgri Report Spectra Reanalysis Required?			Not Identical to Rev 0 Spectra Reanalysis Required?	
Identification	Elev/Bldg	Identical To Hosgri Report Spectra	No (State Reason)	Yes (Give Rev No. & Date)	Identical To Rev 0 Spectra	No (State Reason)	Yes (Give Rev No. & Date)
Component Cooling Water Surge Tank	163'/Aux		Acceleration reduced 0.49 g		√		
Auxiliary Feedwater Pump Turbine Missile Barrier (Unit 1 only)	100'/Aux			Qualified to C-17 Rev 0 on 6-11-82	√		
Spent Fuel Pit Heat Exchanger	100'/Aux			Analysis in Progress	√		
Containment Fancoolers	140'/CS		See Calculation Rev 2 (7-7-82)		√		
Compressed Breathing Air Bottle Support Structures	85'/TB			Analysis in Progress	√		
H_2 and N_2 Bottle Support Structures	115' MSS						
Component Cooling Water Pump Motor	73'/Aux			Analysis in Progress	√		
Post-LOCA Hydrogen Recombiners-Heaters	140'/CS		See Calculation M-18 Rev 3 (7-8-82)		√		
Post-LOCA Hydrogen Recombiners-Panel	100'/Aux		See Calculation M-18 Rev 3 (7-7-82)		√		

Aux = Auxiliary building
TB = Turbine building
IS = Intake structure
MSS = Misc small structures
CS = Containment structure

2.4.5 DESIGN REVIEW

2.4.5.1 Evaluation to Criteria

The electrical raceway systems are analyzed for seismic loading in the transverse, longitudinal, and vertical directions. Each of the 436 support types are evaluated, based on either its generic condition or its as-built condition, against the acceptance criteria. Most of the supports have been found acceptable. Only a small percentage of the supports do not meet acceptance limit. However, this does not imply that they would fail. They could be shown to have sufficient capacity, through inelastic action, to enable them to perform their function, and thus pose no threat to the safety of the plant. Rather than demonstrating this, which would be time consuming, modifications are made so that all supports will meet the acceptance criteria. Results of the support evaluation to date are summarized in Table 2.4-1.

2.4.5.2 Description of Modifications

The modifications required to date for raceway supports are limited to adding a simple bracing made of 1-3/4 in. X 1-3/4 in. angle irons, or additional welding around angle fittings, so that support members can develop additional moment capacity. Sample modifications are shown on Figures 2.4-1 and 2.4-2.

TABLE 2.4-1

ELECTRICAL RACEWAY SUPPORTS
EVALUATION SUMMARY

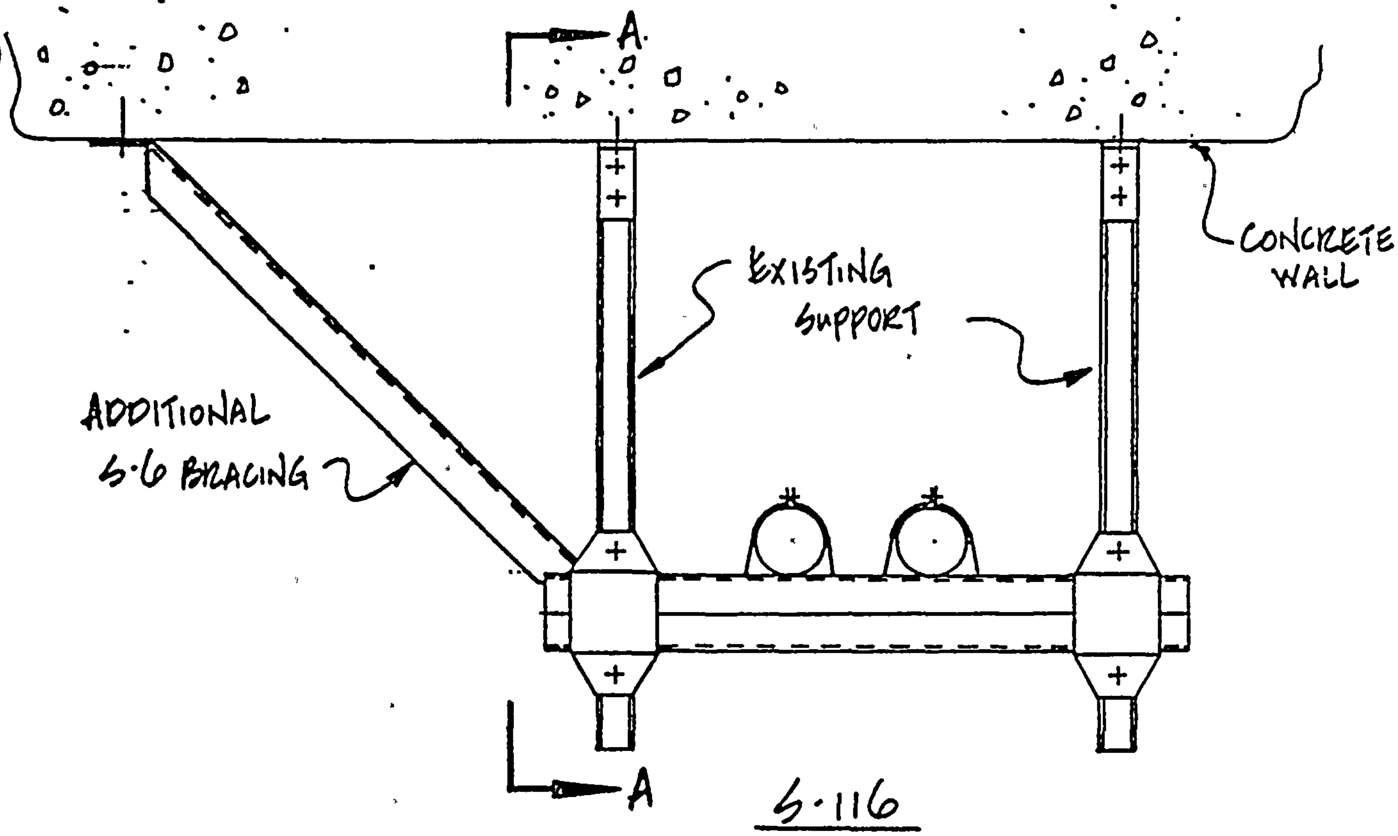
Support Type	Population	STATUS: Qualified Generic	STATUS: Qualified As-built	STATUS: Required Modification	Description of Modification
S-2A	1		X		
S-5B	23	X			
S-12B	1		X		
S-15B	3		X		
S-23	72	X			
S-40	572	X			
S-60A	9		X		
S-60B	5		X		
S-80A	29		X		
S-80B	7		X		
S-81	1	X			
S-86	8		X		
S-91	11	X			
S-93	25		X		
S-94	3	X			
S-116	21			X	Add a S-6 bracing to each support (see Fig. 2.4.1)
S-126	1			X	Add a new member to support
S-138	2		X		
S-145	15		X		
S-146	7		X		
S-154	2		X		
S-159	1	X			
S-166	1	X			
S-179	1	X			
S-182	9	X			
S-188	5		X		
S-200	13	X			
S-210	2		X		
S-214	112	X			
S-230	1		X		
S-233	6		X		
S-240	14	X			
S-248	11	X			
S-253	4	X			
S-260	1			X	Additional welding
S-262	7		X		
S-268	3	X			
S-271	1	X			
S-285	1		X		

TABLE 2.4-1 (Continued)

Support Type	Population	STATUS Qualified Generic	Qualified As-built	Required Modification	Description of Modification
S-293	6	X			
S-305	13	X			
S-314	7	X			
S-315	4		X		
S-317	4		X		
S-318	2		X		
S-320	2	X			
S-321	1	X			
S-326	5		X		
S-328	8	X			
S-330	1		X		
S-331	5		X		
S-335	2	X			
S-345	31		X		
S-349	5	X			
S-354	10		X		
S-356	10	X			
S-362	3		X		
S-363	1		X		
S-365	3	X			
S-368	3	X			
S-385	4		X		
S-389	4			X	Add clip and weld corners (see Fig. 2.4.2)
S-390	3			X	Add S-6 bracing to each support
S-391	1		X		
S-393	4	X			
S-394	3	X			
S-412	1	X			
S-424	1		X		
S-448	1		X		
S-455	1	X			
S-479	13	X			
S-482	1		X		
S-549	1			X	Add S-6 Bracing

TABLE 2.4-1 (Continued)

Support Type	Population	STATUS: Qualified Generic	STATUS: Qualified As-built	STATUS: Required Modification	Description of Modification
S-562	8	X			
S-567	10	X			
S-577	3	X			
S-599	1		X		
S-608	78	X			
S-611	1	X			
S-616	1		X		
S-625	1	X			
S-626	1	X			



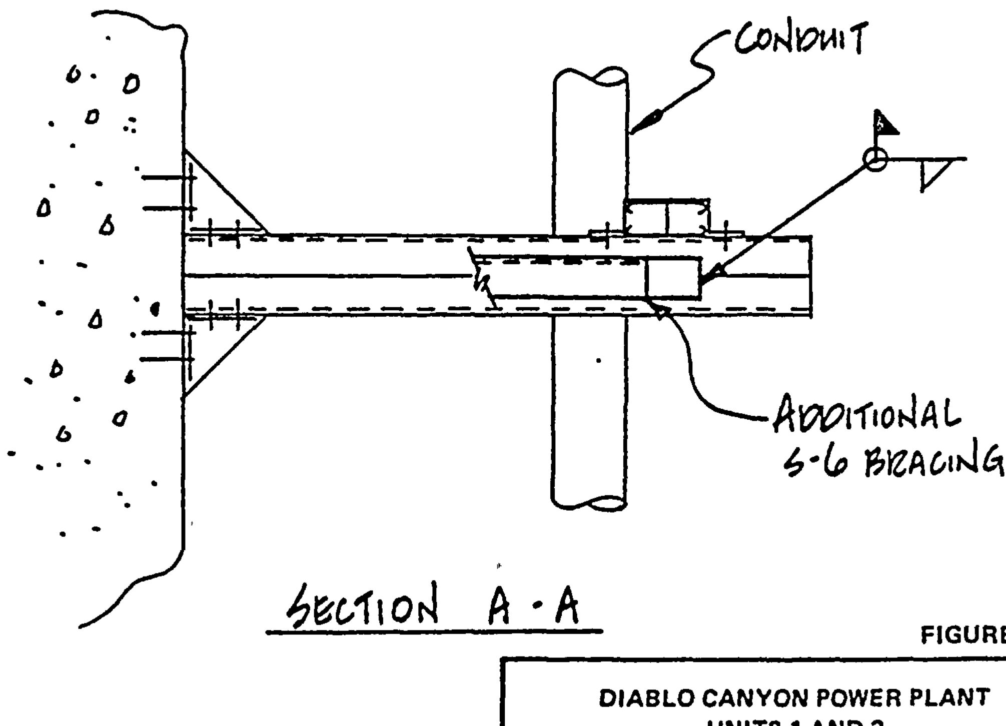
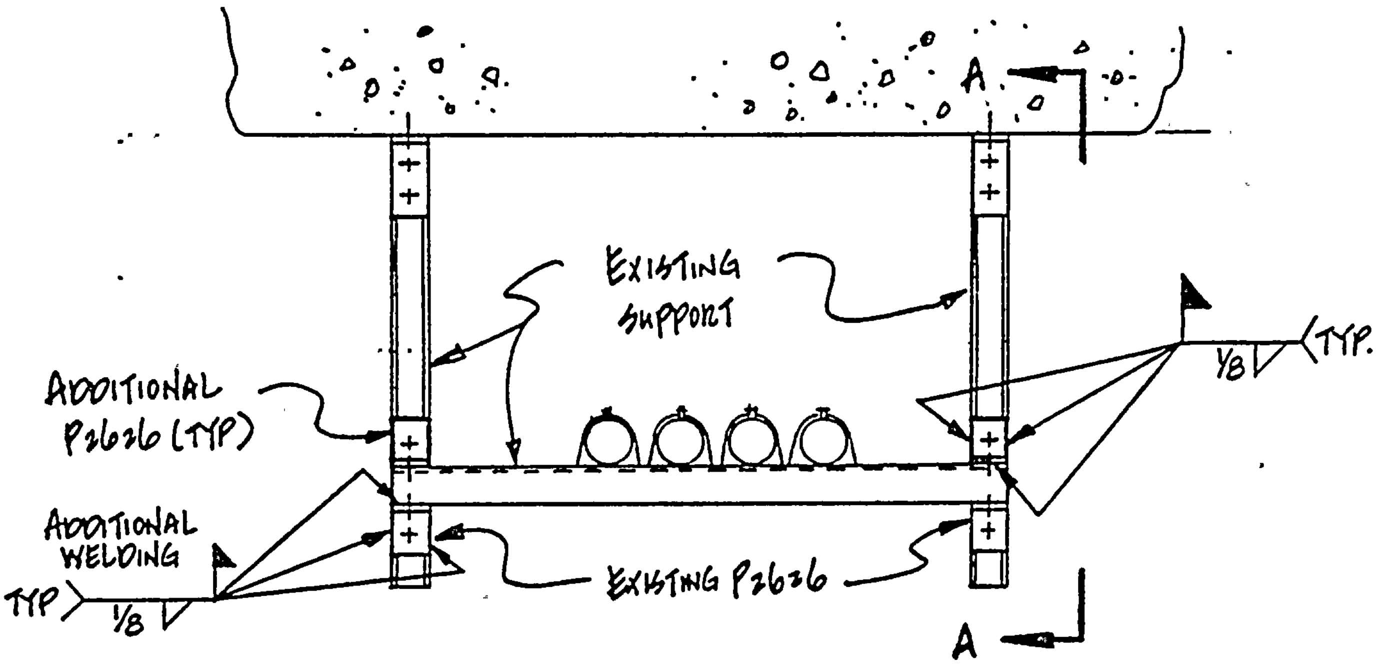


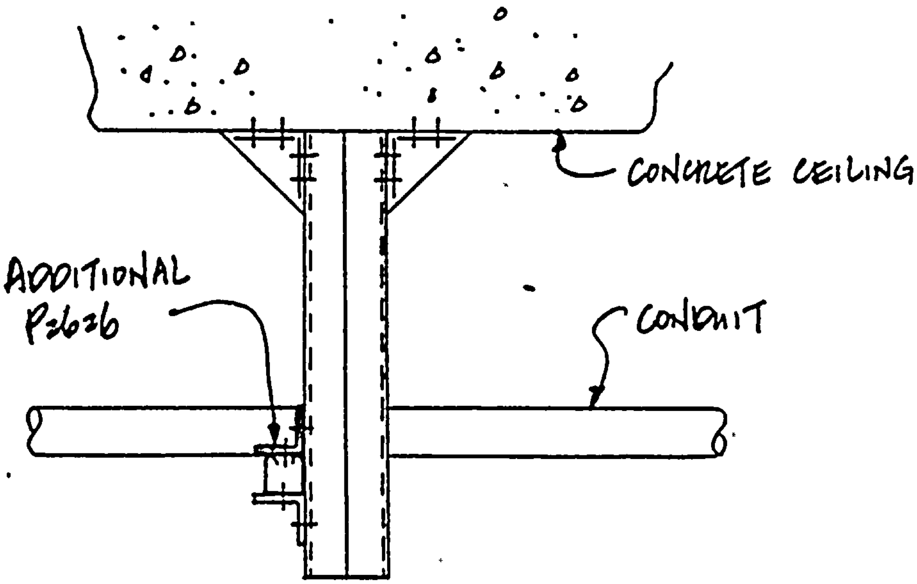
FIGURE 2.4-1

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

MODIFICATION OF SUPPORT S-116



S-389



SECTION A-A

FIGURE 2.4-2

DIABLO CANYON POWER PLANT
UNITS 1 AND 2

MODIFICATION OF SUPPORT S-389