Millstone Power Station Unit 2
Safety Analysis Report

Chapter 5: Structures

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CHAPTER 5 – STRUCTURES

5.1 GENERAL

The design bases for structures required for the normal operating conditions are governed by the building design codes and specifications listed in Section 5.1.2. The basic design criterion for the design basis accident and seismic conditions specified that there shall be no loss of any function of the structures which can cause danger to the safety of the public. It should be noted that the terms "Category" and "Class" are used interchangeably throughout the MP2 FSAR in defining seismic design classifications of Structures, Systems and Components.

5.1.1 CLASSES OF STRUCTURES

Structures are grouped into two classes, depending on how their functions relate to public safety and plant operation.

5.1.1.1 Class I Structures

Class I structures are those structures whose loss of function could:

a. Cause or increase the severity of an accident.

b. Preclude establishing and maintaining safe shutdown.

c. Result in a release of radioactivity to the site boundary in excess of the l0 CFR l00 guidelines.

Class I structures are designed to withstand the appropriate seismic and other applicable loads without loss of function. These Class I structures are sufficiently isolated or protected from Class II structures to ensure that their integrities are maintained at all times.

The following are Class I structures:

a. The containment shell and internals

b. The enclosure building

c. The auxiliary building

d. The warehouse (eastern portion of the auxiliary building)

e. The turbine building, except turbine pedestals

f. The intake structure

g. The supports for all Class I system components

Note: See Table 1.4-1 for complete listing of Class I structures, systems, and components that follow the guidance of Regulatory Guide 1.29, Seismic Design Classification.

5.1.1.2 Class II Structures

Class II structures are those whose failure would not result in the release of radioactivity beyond the site boundary in excess of the l0 CFR 20 annual limits and would not prevent safe shutdown of the reactor. The failure of Class II structures, however, may interrupt power generation.

All structures that are not listed under Class I are Class II structures.

5.1.2 CODES AND SPECIFICATIONS

The following codes and specifications, where applicable, were used as the bases for the design and construction of all structures. Modifications to these codes and specifications are noted in the appropriate sections that describe the details of the structures, materials, and construction practices. Later editions of the AISC Manual of Steel Construction (7th, 8th, and 9th edition - Allowable Stress Design) and ACI 318 Code (1971, 1977, 1983, 1989, and 1995) were used for the design and construction of new and modified portions of structures.

Subsequent editions of the AISC Manual of Steel Construction and the ACI 318 Code, as governed by the plant design change process, may be used for the design and construction of new and modified portions of structures.

a. Uniform Building Code (l967 Edition)

b. Building Code Requirements for Reinforced Concrete (ACI 3l8-63)

c. Specifications for Structural Concrete for Buildings (ACI 30l-66)

d. Manual of Steel Construction (AISC, 6th Edition, 1963)

e. State of Connecticut Building Code

f. ASME Boiler and Pressure Vessel Code (1968 Edition)

g. ASTM Standards – Materials and testing procedures used are referenced in the appropriate sections. Whenever possible, ASTM or ASME material and testing procedures are used. Materials and testing procedures for new and modified portions of structures conform to ASTM or ASME Standards as implemented by plant design change documents.

5.1.3 REGULATORY GUIDES

The following Regulatory Guides that were in effect at the time of application for an Operating License were used, where applicable, to establish the bases for the design and construction of all structures. Every effort was made to follow the guidance of the documents. Any areas where the guidance was not followed are detailed in the specified subsections.

Regulatory Guide Number	Title	Subsection
1.10	Mechanical (Cadwell) Splices in Reinforcing Bars of Concrete Structures	5.9.3.2.3
1.11	Instrumentation Lines Penetrating Primary Reactor Containment	5.2.7.2.1
1.13	Fuel Storage Facility Design Basis	5.4.1.1.3
1.15	Testing of Reinforcing Bars for Concrete Structures	5.9.3.2.2
1.19	Nondestructive Examination of Primary Containment Liners	5.9.3.5.3
1.35	In-service Surveillance of Ungrouted Tendons in Prestressed Concrete Containment Structures	5.2.8.4
1.12	Instrumentation for Earthquakes	5.8.6
1.18	Structural Acceptance Test for Concrete Primary Reactor Containments	5.2.8.2

5.2 CONTAINMENT GENERAL DESCRIPTION

The containment system used for Millstone Unit 2 consists of a concrete cylindrical structure, hereinafter referred to as the containment, and a steel framed structure called the enclosure building, which completely surrounds the containment. The spaces between the enclosure building and the containment, together with selected areas of the auxiliary building such as the penetration rooms and rooms containing the engineered safety features, are referred to as the enclosure building filtration region (EBFR). In the event of a loss-of-coolant accident (LOCA), the EBFR is maintained at a slight vacuum by the enclosure building filtration system. Air from the EBFR is processed through charcoal filters and released through the Millstone stack.

The containment consists of a prestressed, reinforced concrete cylinder and dome connected to and supported by a massive reinforced concrete foundation slab. The cylindrical portion is prestressed by a post-tensioning system composed of horizontal and vertical tendons, with the horizontal tendons placed in three 240 degree systems using three buttresses as supports for the anchorages. The dome has a three-way post-tensioning system. The concrete foundation slab is conventionally reinforced with high strength reinforcing steel. A continuous access gallery is provided beneath the base slab for installation of vertical tendons. A one-quarter inch thick welded steel liner is attached to the inside surface of the concrete shell to ensure a high degree of leak-tightness. The floor liner is installed on top of the structural slab and is then covered with concrete.

The containment completely encloses the reactor, reactor coolant system, and portions of the auxiliary and engineered safety features systems. It ensures that an acceptable upper limit for leakage of radioactive materials to the environment will not be exceeded even if gross failure of the reactor coolant system occurs.

Principal nominal dimensions of the containment are as follows:

Inside diameter (feet)	130
Inside height (feet)	175
Cylindrical wall thickness (feet)	3.75
Dome thickness (feet)	3.25
Foundation slab thickness (feet)	8.5
Liner plate thickness (inches)	0.25
Internal free volume (cubic feet)	1,899,000

The containment is shown in Figures 5.2–1 and 5.2–2.

5.2.1 CONSTRUCTION MATERIALS

The following materials are used in the construction of the containment.

a. Structural and Miscellaneous Steel

Rolled shapes, plates, and bars	ASTM A-36
Crane rails	ASTM A-1
High strength bolts	ASTM A-325 or ASTM A-490
Stainless steel	ASTM A-240, Type 304

b. Concrete

Base slab (psi)	5000
Cylindrical wall and dome (psi)	5000
Tendon access gallery (psi)	3000
Floor slabs above floor liner (psi)	3000
Primary shield walls and steam generator pedestals (psi)	5000
All other internal structures (psi)	4000

c. Reinforcing Steel

Deformed bars	ASTM A-615, Grade 60
Spiral bars	ASTM A-82

d. Prestressing Steel Tendons, Anchorage, and Sheaths

Wires	ASTM A-421, Type BA
Bearing plates	Armco VNT
Stressing washers	ASTM A-4330
Shims	Armco VNT
Sheaths	Galvanized corrugated steel tubing, 22 Gauge

e. Containment Steel Liner Plate

one-quarter inch plates	ASTM A-285, Grade A
Insert plates	ASTM A-516, Grade 60

Penetration sleeves:

Pipes	ASME SA-333, Grade 6
Plates	ASME SA-516, Grade 60

Exception	ASME SA-36 for Equipment Hatch External Bolting Attachments

f. Interior Coating (Original Construction)

Steel liner plate

Primer	Carbo-Zinc 11
Finish coat	Phenoline Number 305

Concrete and masonry surfaces

Surfacer	Keeler & Long, Number 6548 epoxy block filler
Primer	Keeler & Long Number 7107 epoxy white primer
Finish coat	Keeler & Long, epoxy enamel

g. Interior Maintenance Coatings (First implemented in Mid Cycle 13, 1997)

All coating materials applied to surfaces inside or to be installed in the reactor containment are epoxy materials tested to withstand Millstone Unit 2 design basis loss of coolant accident (DBA-LOCA) conditions. The coating materials and their application comply with the requirements of Regulatory Guide 1.54. Each coating was tested in accordance with ASTM D3911, "Evaluating Coatings Used in Light-Water Nuclear Power Plants at Simulated Design Basis Accident (DBA) Conditions," to the DBA conditions represented by the pressure (70 psig) and temperature (340°F) curve of Figure 1, therein. Prior to exposure to the simulated DBA conditions, each coating was irradiated to an accumulated dose of at least $1x10^9$ Rads in accordance with ASTM D4082, "Effect of Gamma Radiation on Coatings for Use in Light-Water Nuclear Power Plants."

h. Waterproofing Membrane

The waterproofing membrane that was installed during construction of the containment is a continuous plain sheet of polyvinyl chloride applied to the concrete surface with an adhesive. The membrane was applied after the forms were stripped. The membrane is composed of an elastomeric material having thickness of 40 mils (minimum), a minimum tensile strength of 2,000 psi, a minimum elongation of 200 percent at 75-80°F, and the water absorption is less than 0.1%. The extent of the waterproofing membrane is shown in Figures 5.3–2 through 5.3–4 of the FSAR. All joints are lapped and the adhesive is applied continuously to the contact surface.

5.2.1.1 Corrosive Protection

The reinforcing steel and tendon sheaths are cast in the concrete walls and base slab, which is a corrosion inhibitive environment. The steel liner is in direct contact with, and anchored to, the inner surface of the concrete wall. Concrete will passivate the steel surfaces, thereby lowering the galvanic potential and making it compatible with any buried copper ground conductors. Measurements taken in the area show that the earth has a very high resistivity. This further diminishes the possibility of galvanic corrosion. It is therefore believed that there is no need for a cathodic protection system for these steel members.

A refined petroleum oil based product is used as a protective compound for the tendons. The electrical resistivity of this compound is relatively high, which makes it a poor electrolyte. This prevents the possibility of galvanic corrosion that could be detrimental to the tendons.

Note: For description of water intrusion into the tendon gallery during construction and methods of repair, see Appendix 5.F.

5.2.2 DESIGN BASES

The design of the containment structure provides the required features as outlined in Criteria 1, 2, 3, 4, 5, 16, 50, 51, 52, 53, 54, 55, 56, 57, 60, 61, Appendix A of 10 CFR Part 50.

5.2.2.1 Bases for Design Loads

The containment is designed for all credible loads and load combinations. These load combinations consist of loads under normal operation, loads during a LOCA, test loads, and loads due to adverse environmental conditions. The following loads are considered:

a. Dead loads

b. Live loads

c. Loads caused by the pressure and temperature transients of a LOCA

d. Thermal loads

e. Earthquake loads

f. Wind and tornado loads

g. Uplift loads due to buoyant forces

h. External pressure loads

i. Prestressing loads

j. Test loads

5.2.2.1.1 Dead Loads

Dead loads consist of the weight of the containment wall, dome, base mat, interior framing and slabs, and all interior structures and equipment. Equipment dead loads are those specified on the drawings which are supplied by the manufacturers of the equipment installed within the containment.

5.2.2.1.2 Live Loads

Live loads in the containment include design floor loads, equipment live loads, and all loads transmitted through the supports of the enclosure building. A snow load of 60 psf is used for the roof of the enclosure building.

The interior floors and slabs have the following live loads:

a. Floor grating 250 psf

b. Concrete floor slabs 1000 psf

c. Equipment live loads As specified on drawings supplied by the manufacturers of the various pieces of equipment

5.2.2.1.3 Loss-of-Coolant Accident Loads

The design pressure and temperature of the containment are greater than the peak pressure and temperature that would result from a postulated complete blowdown of the reactor coolant. This could cover a rupture of the reactor coolant system up to and including the severance of the largest reactor coolant pipe.

The supports for the reactor coolant system are designed to withstand the blowdown forces and to restrict the structural deformations associated with the sudden severance of the reactor coolant piping.

Transient pressures and corresponding temperatures resulting from a LOCA or main steam line break accident are presented in Section 14. These serve as the basis for a containment design pressure of 54 psig.

The variations of temperature with time and the forces resulting from the expansion of the liner plate with the temperature associated with a LOCA are considered.

5.2.2.1.4 Thermal Loads

The spaces between the containment and the enclosure building are maintained at a minimum temperature between 55 and 70°F by unit heaters, as discussed in Section 9.9.2. Under normal operating conditions, a temperature gradient exists when the exterior structure of the concrete cylindrical wall is at 55 to 70°F, while the interior surface is at an operating ambient temperature of 120°F.

However, to be conservative, a design temperature of 20°F (average minimum temperature at the site) is applied at the exterior surfaces of the concrete cylindrical wall in analyzing the temperature gradient under the normal operating conditions.

The temperature gradients through the cylindrical wall of the containment during normal operating and LOCA conditions are shown in Figure 5.2–3.

5.2.2.1.5 Earthquake Loads

Earthquake loads are predicated on an operating basis earthquake (OBE) at the site having a horizontal ground surface acceleration of 0.09 g. In addition, a design basis earthquake (DBE), having a horizontal ground surface acceleration of 0.17 g, is used to check the design to ensure that loss of structural functions would not occur. The seismic design spectrum curves are given in Section 5.8.1.1. A vertical component two-thirds of the magnitude of the horizontal component at the ground surface is applied simultaneously as a static coefficient throughout the height of the structure. A dynamic analysis, utilizing the response spectrum technique, is used to obtain the earthquake loads for design.

5.2.2.1.6 Wind and Tornado Loads

Wind loads for the containment are determined on the basis of the ASCE Paper 3269, “Wind Forces on Structures,” using the highest wind velocity at the site for a 100 year recurrence period. The ASCE Paper 3269 is used mainly to determine the shape factors. Based upon the site location and the structure classification, the design wind velocity is taken to be 115 mph with gusts up to 140 mph.

The containment has been analyzed for tornado loads (not coincident with a LOCA or earthquake) on the following basis:

a. Differential bursting pressure between the interior and exterior of the containment is assumed to be 3 psi pressure occurring in the three seconds (1 psi/sec), followed by a calm for two seconds and a re-pressurization.

b. Lateral loads on the containment are based on a tornado funnel which is conservatively assumed to have a peripheral tangential velocity of 300 mph and a forward translation of 60 mph. These velocities are added together, resulting in a design basis tornado wind velocity of 360 mph. The applicable portions of the wind design methods described in the ASCE Paper 3269 are used, particularly for the shape factors. The provisions in the paper for gust factors and variations in wind velocity with respect to height are not applied. The wind velocity is assumed to be uniformly distributed over the height of the structure.

c. A tornado-borne missile as defined in Section 5.2.5.1.2.

With the exception of the missile impact area, the allowable stresses necessary to resist the effects of tornadoes are 90 percent of the yield strength of the reinforcing steel and 85 percent of the ultimate strength of the concrete.

A discussion of the probability of tornado occurrence is presented in Section 2.3 of the Millstone Unit 3 FSAR (Reference 5.2-13).

5.2.2.1.7 Hydrostatic Loads

Buoyant forces resulting from the displacement of ground or flood water by the structure are accounted for in the design of the structure.

The following water levels are considered:

Ground water Elevation	(+) 5-0
Flood water Elevation	(+) 18-1

5.2.2.1.8 External Pressure Loads

An external design pressure, equivalent to a barometric pressure rise to 31 inches of mercury after the containment is sealed at 29 inches of mercury, is considered. For this condition, a differential pressure of 2 psi from the exterior to the interior of the containment is assumed and applied as an external pressure on the containment.

This external design pressure is also adequate to permit the containment to be cooled to 50°F from an initial maximum operating temperature of 120°F.

5.2.2.1.9 Prestressing Loads

Where applicable, prestressing forces are considered in the loading combinations. These include the axisymmetric loads of normal compressive forces in the containment wall and dome, and the local effects at the anchorage zones from stressing and shimming the tendons.

5.2.2.1.10 Test Loads

At the completion of construction, the containment and its penetrations are tested at 115 percent of the design pressure. This test pressure is considered in the load combinations to ensure the structural integrity of the containment.

5.2.2.2 Load Combinations

To ensure the structural integrity, both the working stress method and the ultimate strength method are used in the design of the containment for various loading combinations. The containment is examined with respect to strength, the nature, and the amount of cracking, the magnitude of deformation, and the extent of corrosion so as to ensure proper performance. The structure is designed to meet the performance and strength requirements under the following conditions:

a. Prior to prestressing

b. At transfer of prestress

c. Under sustained prestress

d. At design loads

e. At factored loads

All design criteria are in accordance with ACI-318-63 unless stated otherwise herein. Members subject to stresses produced by temperature forces combined with other loads in design load combinations may be proportioned for reinforcing steel stresses 33-1/3% greater than those specified.

5.2.2.2.1 Load Prior to Prestressing

Under this condition, the structure is designed as conventionally reinforced concrete. It is designed for dead loads, live loads (including construction loads), and wind loads. Allowable stresses are in accordance with ACI-318-63.

5.2.2.2.2 Loads at Transfer of Prestress

The containment is checked for prestress loads and the resulting stresses are compared with those allowed by ACI-318-63, with the following exception:

> The membrane compressive stresses are limited to 0.30 f'_{ci} whereas in combination with flexural compressive stresses, the maximum allowable compressive stresses are limited to 0.60 f'_{ci} in accordance with the ACI Code.

For local stress concentrations with nonlinear stress distribution, as predicted by the finite element analysis, a compressive stress of 0.75 f'_c is permitted when reinforcing steel is utilized to distribute and control these localized strains. These high stresses are allowed since they occur only in small localized areas which are confined by material at lower stresses, and would have to be considerably greater than the allowable stresses of the material before significant local plastic yielding takes place.

Membrane and flexural tensile stresses in concrete are permitted provided that they do not jeopardize the integrity of the steel liner plate. Membrane tensile stresses in concrete are permitted to occur during the post-tensioning sequence, but are limited to 1.0 f'_{ci}. When there are flexural tensile stresses but no membrane tensile stresses, the section is designed in accordance with Section 2605(a) and the ACI Code. The stresses in the liner plate due to the combined membrane tensile stress and flexural tensile stresses are limited to 0.5 f_y. The effects of the prestressing sequence are considered.

Design criteria for shear are in accordance with ACI-318-63, Chapter 26, as modified by the equations shown in Section 5.2.2.2.5. For ultimate strength design, a load factor of 1.5 is used.

5.2.2.2.3 Loads Under Sustained Prestress

The design conditions and the allowable stresses for this case are the same as those stated in Section 5.2.2.2.2 except that the allowable tensile stresses in nonprestressed reinforcing are limited to 0.5 f_y and no membrane tensile stresses in concrete are permitted. When membrane stresses are combined with flexural stresses, tension is permitted provided it does not jeopardize the integrity of the liner plate. Where the flexural tensile stresses exist, the section is designed in accordance with Section 2605(b) of the ACI Code.

5.2.2.2.4 At Design Loads

The containment is designed by the working stress method for the following loading combinations:

a.	$D + F + L$	Construction case
b.	$D + F + L + T_o + E$	Operating case
c.	$D + F + L + P + T_i$	Design incident case
d.	$D + F + L + T_s + E$	Prolonged shutdown case
e.	$D + F + L + 1.15\ P$	Test case

where:

D = dead loads

L = live loads

F = prestressing loads

P = design pressure

T_i = thermal loads due to the loss-of-coolant incident

T_o = thermal loads due to the operating temperature

T_s = thermal loads due to transient wall temperature over a prolonged shutdown (20°F at exterior face, 70°F at center, 50°F at interior face)

E = operating basis earthquake loads (0.09 g)

Long term effects of creep and shrinkage in concrete are considered in all loading combinations.

When determining the value of the modulus of elasticity of concrete to be used in the containment analysis, instantaneous loads such as internal pressure are differentiated from sustained loads such as dead, prestress, and thermal loads. This distinction is necessary to evaluate the effects of creep and shrinkage deformations on the concrete.

The following equation is used to include the effect of creep and shrinkage in the modulus of elasticity:

$$E_{cs} = E_{ci} \times \frac{E_i}{E_s + E_i}$$

where:

E_{cs} - sustained concrete modulus of elasticity

E_{ci} - instantaneous concrete modulus of elasticity

E_s - concrete strain from sustained loads

E_i - concrete strain from instantaneous loads

The modifications described above are used in the analysis of the containment shell for both the design and factored load conditions.

No modification is made to Poisson's ratio for concrete for either sustained or instantaneous loads.

Sufficient prestressing is provided in the cylindrical and dome portions of the containment to eliminate membrane tensile stresses under design load combinations. Flexural tensile cracking is permitted, but is controlled by unprestressed reinforcing steel.

Under the design load combinations, the same performance criteria as specified in Section 5.2.2.2.2 are applied with the following exceptions:

a. When the net membrane compressive stresses are below 100 psi, they are neglected and a cracked section is assumed in the concrete for the computation of unprestressed reinforcing steel for flexural tension. Flexural tensile stress of $0.5f_y$ in unprestressed reinforcing steel are allowed.

b. When the maximum flexural tensile stress does not exceed $6\sqrt{f'_c}$ and the extent of the tension zone is no more than one-third the depth of the section under consideration, unprestressed reinforcing steel is provided to carry the entire tensile forces in the tension block. Otherwise, a cracked section is assumed in the design of the unprestressed reinforcing steel. When the tensile stresses due to the bending moment are additive to the thermal tensile stresses, the allowable tensile stresses in the unprestressed reinforcing steel is $0.5\ f_y$.

c. The problems of shear and diagonal tension in a prestressed concrete structure are considered in two parts: membrane principal tension and flexural principal tension. Since sufficient prestressing is used to eliminate membrane tensile stresses at design loads, membrane principal tension is not a critical design case. Membrane principal tension due to combined membrane tension and membrane shear is discussed in Section 5.2.2.2.5.

Flexural principal tension is associated with bending in planes perpendicular to the surface of the shell and shear acting along these planes (radial shear). The present ACI-318-63 provisions of Chapter 26 for shear are adequate for design purposes with the modifications as discussed in Section 5.2.2.2.5.

Crack control in concrete is accomplished through the use of reinforcing steel in accordance with the ACI-ASCE Code Committee Standards. These criteria are based on recommendations of the Prestressed Concrete Institute. The minimum reinforcing provided in terms of the gross concrete cross-sectional area is as follows:

a. 0.25 percent at the tension face for small members.

b. 0.20 percent for medium size members.

c. 0.15 percent for large members.

d. 0.25 percent at the exterior faces of the containment.

5.2.2.2.5 At Factored Loads

Load factors are the coefficients by which loads are multiplied for analysis purposes to ensure that the load-deformation behavior of the structure is elastic with low strain. The load factor approach is used in the design as a means of making a rational evaluation of the isolated loads which must be considered to assure an adequate safety margin for the structure. This approach permits the designer to place the greatest conservatism on those loads which have the most variation and which most affect the overall safety of the structure. Consequently, a smaller load factor is used for the fixed gravity loads and larger load factors are employed for the LOCA, earthquake, and tornado loads. Justification of the load factors is discussed in Appendix 5.B.

The design of the containment satisfies the following load factors and load combinations:

$C = 1/\phi\ [(1.00 \pm 0.05)\ (D) + 1.5P + 1.0T_I + 1.0F]$ Eq (1)

$C = 1/\phi\ [(1.00 \pm 0.05)\ (D) + 1.25P + 1.0T_I + 1.25H + 1.25E + 1.0F]$ Eq (2)

$C = 1/\phi\ [(1.00 \pm 0.05)\ (D) + 1.25H + 1.0R + 1.0F + 1.25E + 1.0T_o]$ Eq (3)

$C = 1/\phi\ [(1.00 \pm 0.05)\ (D) + 1.25H + 1.0F + 1.25W + 1.0T_o]$ Eq (4)

$C = 1/\phi\ [(1.00 \pm 0.05)\ (D) + 1.0P + 1.0T_I + 1.0H + 1.0E' + 1.0F]$ Eq (5)

$C = 1/\phi\ [(1.00 \pm 0.05)\ (D) + 1.0H + 1.0R + 1.0E' + 1.0F + 1.0T_o]$ Eq (6)

where:

C = required capacity of the structure to resist factored loads

ϕ = capacity reduction factor

D = dead loads of structures and equipment plus any other permanent loads which contribute stresses, such as hydrostatic pressure or soil pressure. In addition, portions of the live loads are added when these include items such as piping, cables, and trays suspended from floors. An allowance is also made for future additional permanent loads.

P = design accident pressure loads

F = final prestress loads

R = forces or pressures on structure due to the rupture of any one pipe

H = forces on structure due to the thermal expansion of pipes

T_I = thermal loads due to the temperature gradient through the walls, based on temperature corresponding to unfactored design accident pressure

T_o = thermal loads due to the normal operating temperature gradient through the walls

E = operating basis earthquake loads

E′ = design basis earthquake loads

W = normal wind or tornado loads (use 1.0 factor for tornado loads)

Equation (1) assures that the containment has the capacity to withstand pressure loadings at least 50 percent greater than those calculated for the postulated LOCA alone.

Equation (2) assures that the containment has the capacity to withstand loadings at least 25 percent greater than those calculated for the postulated LOCA with a coincident operating basis earthquake.

Equation (3) assures that the containment has the capacity to withstand earthquake loadings 25 percent greater than those calculated for the operating basis earthquake coincident with the associated rupture of any one of the attached piping.

Equation (4) assures that the containment has the capacity to withstand wind loadings at least 25 percent greater than the design wind loadings or full tornado loadings.

Equations (5) and (6) assure that the containment has the capacity to withstand either a postulated LOCA or the rupture of any one of the attached piping coincident with the design basis earthquake.

The yield strength of the structure is defined as the upper limit of the proportional stress-strain behavior of the effective load carrying capacities of the structural materials. The stresses from these load combinations, with the load factors given therein, are less than the yield strength of the structure. For steel (both prestressed and unprestressed), the upper limit is taken to be the guaranteed minimum yield strength, as specified in the appropriate ASTM specifications. For concrete, it is the ultimate values of shear (as a measure of diagonal tension) and bond as specified by ACI-318-63 Code, as well as the 28 day ultimate compressive strength for concrete in flexure (f'_c).

The peak strain in the concrete due to secondary moments, membrane loads, local loads, and thermal loads is limited to 0.003 inch/inch.

The predicted strain in the steel liner plate for all loading combinations does not exceed 0.005 inch/inch.

Principal concrete tension due to the combined membrane tension and membrane shear, excluding flexural tension due to bending moments or thermal gradients, is limited to $3((f'_c)^{1/2})$.

Principal concrete tension due to the combined membrane tension, membrane shear, and flexural tension caused by the bending moments or thermal gradients is limited to $6((f'_c)^{1/2})$. When the principal concrete tension exceeds the limit of $6(f'_c)^{1/2}$, reinforcing steel is provided in the following manner:

a. Thermal flexural tension – Reinforcing steel is provided in accordance with ACI-505. The minimum area of steel provided is 0.25 percent of the gross concrete cross-sectional area in each direction.

b. Bending moment tension – Sufficient reinforcing steel is provided to resist the moment on the basis of the cracked section theory. The exception is when the bending moment tension is additive to the thermal tension, the allowable tensile stress in the reinforcing steel is 0.5 f_y.

Shear stress limits and reinforcing for radial shear are in accordance with Chapter 26 of ACI-318 with the following exceptions:

a. Formula 26-12 of the Code is replaced by:

$$V_{ci} = Kb'd(f'_c)^{1/2} + M_{cr}(V/M') + V_i$$

where:

$K = 1.75 - (0.036/np') + 4.0\ np'$ but not less than 0.6 for $p' > 0.003$.

For $p' < 0.003$, the value of K is zero.

$$n = (505/(f'_c)^{1/2})$$

$p' = (A'\ s/bd)$

$M_{cr} = (1/y)(6((f'_c)^{1/2}) + f_{pe} + f_n + f_i)$

and

f_{pe} = compressive stresses in concrete due to prestress only applied normal to the cross-section after all losses (including the stresses induced by any secondary moments), at the extreme fiber of the section at which tensile stresses are caused by live loads. (f_{pe} is negative for tensile stresses; positive for compressive stresses)

f_n = stresses due to applied axial loads (f_n is negative for tensile stresses; positive for compressive stresses)

f_i = stresses due to the initial loads at the extreme fiber of a section at which tensile stresses are caused by the applied loads (including the stresses induced by any secondary moments, f_i is negative for tensile stresses; positive for compressive stresses)

V = shear at the section under consideration due to the applied loads

M′ = moment at the critical section under consideration, in the direction of decreasing moment, due to the applied loads. The critical section is considered to be at a distance of 1/2 (d) from the support, where d is the effective depth of the concrete section.

V_i = shear due to the initial loads (positive when the initial shear is in the same direction as the shear due to the applied loads).

The lower limit given by the ACI-318-63 on V_{ci} as $1.7b'd(f'_c)^{1/2}$ is not applied.

b. Formula 26-13 of the Code is replaced by:

$$V_{cw} = 3.5b'd(f'_c)^{1/2} \left[1 + (fpe + fn)/(3.5(f'_c)^{1/2})\right]^{1/2} + V_p \qquad \text{Eq (8)}$$

where V_p = radial shear component of effective prestress due to the curvature of tendon at the section considered, and the term, f_n, is as defined in Equation (7). All other notations are in accordance with Chapter 26 of ACI-318-63.

Equation (7) is based on test and work done by Dr. A. H. Mattock of the University of Washington. Equation (8) is based on the commentary for the Proposed Redraft of Section 2610 of ACI-318 by Dr. A. H. Mattock, dated December 1962.

When these equations indicate that the allowed shear stress in concrete is zero, radial shear ties are provided to resist all the calculated shear.

5.2.2.2.6 Prestress Losses

In accordance with ACI-318-63, the design provides for prestress losses caused by the following effects:

a. Seating of anchorage

b. Elastic shortening of concrete

c. Creep of concrete

d. Shrinkage of concrete

e. Stress relaxation of steel

f. Frictional losses due to intended or unintended curvature in the tendon

All of these losses have been predicted with a reasonable degree of accuracy.

In the present application, the environment of the prestressing system and concrete is not appreciably different from that found in conventional prestressed concrete structures such as bridges and buildings. Data from research and practical experience with this type construction have made it possible to evaluate conservatively the allowances for all the prestress loss due to various causes.

5.2.2.2.7 Capacity Reduction Factors

The capacity of all load-carrying structural elements is reduced by a capacity reduction factor (ϕ) as given below. The justification for these numerical values is given in Appendix 5.C. These factors provide for the possibility that small adverse variations in material strengths, workmanship, dimensions, control, and degree of supervision, while individually within required tolerances and the limits of good practice, may occasionally combine to result in undercapacity.

The capacity reduction factors used are as follows:

ϕ = 0.90 for concrete in flexure

ϕ = 0.85 for tension, shear, bond, and anchorage in concrete

ϕ = 0.75 for spirally reinforced concrete compression members

ϕ = 0.70 for tied compression members

ϕ = 0.90 for fabricated structural steel

ϕ = 0.90 for mild reinforcing steel

ϕ = 0.95 for prestressed tendons in direct tension

5.2.2.3 Structural Analysis

The containment is analyzed by a finite element computer program for individual loading cases of dead loads, live loads, winds and tornadoes, temperatures, pressures, and prestresses. A dynamic analysis for seismic loads is performed. The results of the various loadings are superimposed according to the design and factored equations as stipulated in Sections 5.2.2.2.4 and 5.2.2.2.5.

The ACI-318-63 design methods and allowable stresses are used for the concrete as well as for the prestressed and unprestressed reinforcing steel, except as noted herein.

5.2.2.3.1 Critical Areas of Analysis

The main areas for design analysis are:

- Restraints at the top and bottom of the cylinder
- Restraints at the edge of the dome
- Ring girder
- Behavior of the base slab relative to an elastic foundation
- Transient temperature gradients in the steel liner plate and concrete
- Large penetrations
- Tendon anchorage zones
- Concentrated loads
- Seismic loads

5.2.2.3.2 Analytical Techniques

The analysis of the containment consists of two parts: the axisymmetric analysis and the nonaxisymmetric analysis. The axisymmetric analysis is performed by utilizing a finite element computer program for the individual loading cases of dead loads, live loads, temperatures, pressures, and prestresses. The axisymmetric finite element representation of the containment assumes that the structure is axisymmetric and does not take into account the buttresses, penetrations, brackets, and anchors. These items, together with the lateral loads due to earthquakes, winds, tornadoes, and various concentrated loads, are considered in the nonaxisymmetric analysis.

5.2.2.3.2.1 Axisymmetric Analysis

The axisymmetric analysis is performed by using a finite element computer program developed by Dr. E. L. Wilson under the sponsorship of the National Science Foundation. Such a method of analysis is normally used for thick-walled structures where a conventional shell analysis may yield less accurate results. Good correlation has been demonstrated between the finite element analysis method and the test results for thick wall model vessels.

The finite element technique is a general method of structural analysis in which a continuous structure is replaced by a system of elements connected at a finite number of nodal points. In applying this method to an axisymmetric solid structure, such as a containment shell, the continuous structure is replaced by a system of rings of quadrilateral cross-sections which are interconnected at the circumferential joints. Based on energy principles, a set of force equilibrium equations is formulated in which the radial and axial displacements at the circumferential joints are the unknowns. The results obtained by solving these equations are the deformations of the structure under the given loading conditions.

The finite element mesh used to describe the structure is shown in Figures 5.2–16 through 5.2–18. The upper and lower portions of the containment are analyzed separately to permit use of a greater number of elements for those areas of the structure which are of major concern, e.g., the ring girder area and the haunch connecting the cylindrical shell to the base slab. The finite element mesh of the base slab is extended into the foundation to give consideration to the elastic nature of the foundation material and its effect upon the behavior of the base slab. The tendon access gallery is designed as a separate structure.

The finite element analysis produces stresses due to axisymmetric loads. The stresses from the earthquake loads, as well as wind and tornado loads, are obtained by the nonaxisymmetric analysis and then superimposed on the stresses obtained from the finite element analysis. The final summation of all the stresses is used in the design of the base slab, shell, and dome. The liner plate is considered as an integral part of the structure and is included in the finite element mesh of the containment.

Thermal loads result from temperature differentials across the cylindrical wall. The design temperature gradients for the containment are shown on Figure 5.2–3. In the finite element analysis, when temperatures are specified at every nodal point, thermal stresses are obtained at the center of each element.

The computer program used in the finite element analysis is capable of handling the following inputs:

Eight different materials

Nonlinear stress-strain curves for each material

Axisymmetric loadings of any shape

The program outputs are:

C

ϕ

D

P

An auxiliary computer program plots the isostress curves based on the outputs of the aforementioned program.

5.2.2.3.2.2 Nonaxisymmetric Analysis

The nonaxisymmetric configurations and loadings require various methods of analysis. The descriptions of the methods used, as applied to the different parts of the containment, are given in the following sections.

5.2.2.3.3 Buttress and Tendon Anchorage Zone Analyses

The containment has three buttresses. At each buttress, two out of any group of three hoop tendons are spliced by anchoring on the opposite faces of the buttress, and the third tendon is continuous through the buttress.

Between the opposite anchorages in the buttress, the compressive forces exerted by the spliced tendons are twice as great as elsewhere on the shell. This value, combined with the effect of the tendon which is not spliced, is 1.5 times the prestressing force acting outside of the buttresses. The thickness at the buttress is about 1.5 times that of the wall. Thus, the hoop stresses as well as the hoop strains and radial displacements can be considered as being nearly constant all around the structure.

The vertical stresses and strains, caused by the vertical post-tensioning, become constant a short distance from the anchorages because of the stiffness of the cylindrical shell. The stresses and strains remain nearly axisymmetric despite the presence of the buttresses. The effect of the buttresses on the overall vessel behavior is negligible, whether the structure is under dead loads or prestress loads.

The analysis of the anchorage zone stresses at the buttresses is the most critical of the various types of anchorage areas on the shell. The local stress distribution in the immediate vicinity of the bearing plates is investigated using the following procedures:

a. The Guyon's Equivalent Prism Method: This method is based on the experimental photoelastic results as well as the equilibrium considerations of homogeneous and continuous media. It also considers the relative bearing plate dimensions of the anchorages. (Reference 5.2-1)

b. The experimental test data presented by S. J. Taylor of the March 1967 London Conference of the Institute of Civil Engineers (Group H, Paper 49): these data are used to evaluate the effect of the biaxial stresses at the anchorages, including the effects of the trumpet welded to the bearing plate.

c. F. Leonhardt's formula for determining the bursting force in the anchorage zone of a prestressed concrete member (Reference 5.2-2).

d. The three-dimensional stress distribution in the anchorage zones is analyzed in sufficient detail to permit a rational evaluation of the stress concentrations. A conical wedge segment is used as the basic design element and the radial splitting tension is determined as a tangential distribution function. The summation of the splitting stresses through the entire volume of the leading zone establishes the value of the splitting force. This force is a function of the ratio of the base dimensions of the cone; i.e., a/b, and the height, h, of the cone. Several different combinations of the values are analyzed and the most critical values selected.

Note: For description of water intrusion into the tendon gallery from around the bearing plates during construction and methods of repair, see Appendix 5.F.

Transient thermal gradients are used in the analysis and the resulting stresses are superimposed on the bursting stresses obtained from the triaxial stress analysis.

The possibility of a torsional load being applied to the anchors because the tendon wires are twisted was considered. It was determined that no such load occurs since the ram pull rod is free to rotate during the stressing operation.

The design of the anchorage zone reinforcing is based on the results of these analyses, and the following considerations:

a. Bechtel Topical Report BC-Top-7, "Full Scale Buttress Test for Prestressed Nuclear Containment Structures."

b. Design of similar anchorages.

c. Rebar spacing determined to allow ease in placing of sound concrete behind the anchorage bearing plates.

d. Review of the reinforcing details from earlier projects undertaken by the consulting firm of T. Y. Lin, Kulka, Yang and Associates.

5.2.2.3.4 Stresses Near Large Openings

The analytical solutions for determining the state of stresses in the vicinity of large openings are based on the procedure described in the Welding Research Council (WRC) Bulletin Number 102, entitled "State of Stress in a Circular Cylindrical Shell with a Circular Hole." (Reference 5.2-3)

The analysis of the containment, as a whole, was first carried out without considering any openings. This analysis has been done by using the finite element program.

The containment, considering openings, is then analyzed as follows:

a. Formulate differential equations for the shell in a complex variable form with the center of the hole as the origin.

b. Solve the differential equations.

c. Evaluate parameters in the solution.

d. Formulate the boundary conditions based on the stresses obtained from the shell analysis without the hole.

e. Calculate membrane forces, moments, and shears around and at the edge of the opening.

f. Increase and reinforce the wall thickness around the opening to carry the higher forces, moments, and shears. The effects of stress concentration due to the thickening of the wall are considered.

g. Evaluate the effects of prestressing that are not handled in the WRC Bulletin Number 102.

h. Check the design to ensure that the strength of the reinforcement provided is adequate to replace the strength removed by the opening. This check is done to assure a good degree of compatibility between the general containment shell and the areas around the opening.

To analyze the thermal stresses around the openings the following procedure is used. At the edge of the opening a uniformly distributed moment equal but opposite to the moment existing on the rest of the shell is applied. The opening is then analyzed using the methods of the WRC Bulletin Number 102. The effects are superimposed on the stresses calculated by the finite element method.

The membrane stresses resulting from the seismic loads around the openings are modified by appropriate stress concentration factors.

Typical details of reinforcing around the equipment hatch and personnel lock are shown on Figures 5.2–4 and 5.2–5.

5.2.2.3.5 Seismic Analysis

The seismic loads on the containment are determined from a dynamic analysis of the structure. The method of analysis is presented in Section 5.8.3.

5.2.2.3.6 Wind and Tornado Analyses

The design wind loads on the containment are a function of the kinetic energy per volume of the moving air mass. The product of one-half of the air density and the square of the resultant design velocity results in a pressure corresponding to the design wind.

Determination of the design wind pressure on the containment is in accordance with the ASCE Paper 3269, “Wind Forces on Structures.”

The pressure corresponding to the standard air at 0.07651 pcf at 15°C and 760 mm of mercury in terms of the velocity at the appropriate height zone is given by:

$$q = 0.002558V^2$$

Similarly, the design pressure, including the effect of the shape coefficient, C_d, is given by:

$$p = q \times C_d = 0.002558V^2 C_d$$

The design wind velocity for the containment, with the enclosure building attached to it, is taken to be 115 mph with gusts up to 140 mph. The shape coefficient for the enclosure building is found to be:

$$C_d = 1.30$$

The design wind pressure based on the above shape coefficient and the wind velocity of 140 mph is:

$$P = 65.2 \text{ psf}$$

The design pressure is assumed to be constant throughout the height of the enclosure building and is being resisted entirely by the containment.

The design tornado loads on the containment are analyzed on the following basis.

a. Tornado loads are not coincident with an accident or earthquake.

b. Differential bursting pressure between the inside and outside of the containment is assumed to be 3 psi pressure occurring in three seconds (1 psi/second), followed by a calm for two seconds and a repressurization outside.

c. The wind force on the containment is considered as a uniform static load caused by a tornado funnel having a peripheral tangential velocity of 300 mph and a translational velocity of 60 mph. These velocities are combined, resulting in a design wind velocity of 360 mph. The applicable portions on wind design methods described in the ASCE Paper 3269 are used, particularly for shape factors. The provisions in the ASCE Paper 3269 for gust factors and variation of wind velocity with height do not apply. The wind velocity is assumed to be uniformly distributed over the height of the structure.

d. The metal sidings (but not the supporting frame) of the enclosure are designed to be blown away by the differential bursting pressure, thereby, subjecting the containment to the lateral forces resulting from the 360 mph wind.

e. Tornado-borne missiles is defined in Section 5.2.5.1.2.

f. Except for local crushing at the missile impact area, the allowable stresses available to resists the effects of tornadoes are 90 percent of the yield strength of the reinforcing steel and 85 percent of the ultimate strength of the concrete.

Based on an aspect ratio (h/d) of 1.12 and a surface smoothness (t/d) of 1.8 percent, the shape factor for the containment in the tornado analysis is found to be:

$C_d = 0.70$

where:

h = containment height above ground

t = projection of buttress

d = maximum outside diameter of the containment

The maximum pressure is found to be 274 psf (negative) and occurs at 90 degrees to the direction of wind. Since the analysis of the containment is limited to uniform pressure loading, 80 percent of the maximum pressure is assumed to act uniformly across the horizontal projection of the containment. This is believed to be a conservative approach.

5.2.2.3.7 Results of Structural Analyses

Scaled plots for the results of structural analyses on moments, shears, normal forces, and deflections of all loading conditions are shown in Figures 5.2–19 through 5.2–26. Corresponding stresses at various locations on the containment are shown in Tables 5.2-1 through 5.2-9.

Deformations are consistent with the elastic strains; i.e., design is not governed by deformation. The deformations will not affect the continued functional capability of the containment structure or any other Class I structure which might interact with it. Measurable relative displacements are expected between the containment and other components and structures due to the loads imposed. Those displacements have been accounted for by providing expansion joints or, in the case of most pipes, imposing those deflections on the interfacing components.

5.2.3 STEEL LINER PLATE AND PENETRATION SLEEVES

The containment steel liner plate and penetration sleeves are designed to serve as the primary leakage barrier for the containment. Typical details of the liner are shown on Figures 5.2–6 and 5.2–7. The design considered the composite action of the liner and the concrete structure and includes the transient effects of the liner due to temperature changes during construction, normal operation, and the LOCA. The changes in strain to be experienced by the liner due to these effects, as well as those at the pressure testing of the containment, are considered.

The stability of the liner is achieved by anchoring it to the concrete structure. At all penetration sleeves, the liner is thickened to reduce stress concentrations, based on the 1968 ASME Code, Section III, for Class B vessels. The thickened portions of the liner are then anchored to the concrete. All weldments associated with the penetration sleeves are designed to resist the full applied loads. Typical details of the penetration sleeves are shown on Figure 5.2–8.

All components of the liner which must resist the full design pressure, such as penetration sleeves, personnel lock, and equipment hatch, are designed to meet the requirements of paragraph N-1211, of Section III, Nuclear Vessels, 1968 Edition through the summer 1969 addenda of the ASME Code, except the external bolting attachments to the equipment hatch which were designed to meet the requirements of Section III, Subsection NE, 1986 Edition.

In isolated areas, the liner has an initial inward curvature due to fabrication and erection tolerances and inaccuracies. The anchors are designed to resist the forces and moments induced when a section of liner between anchors has initial inward curvature while the adjacent panels have no such imperfections. As a result, inward deformation of the liner between anchors may occur under both operating and accident conditions. The liner and the anchors are designed with sufficient ductility to undergo displacement to relieve the loads without rupturing under these conditions.

With the exception of the containment spray piping supports, an insert plate is provided to transmit the load through the liner at each location where a load is transferred to the walls, slabs, or dome of the containment. The insert plate is anchored to the concrete by appropriate anchors and shear connections. Examples of such insert plates are the polar crane brackets and the floor beam brackets at the operating deck. Typical details of these brackets are shown in Figure 5.2–9.

5.2.3.1 Construction Materials

Materials used for the construction of the steel liner plate and penetration sleeves are listed in Section 5.2.1.

5.2.3.2 Design Criteria

The design criteria applied to the containment steel liner plate to meet the specified leak rate under the operating and accident conditions are as follows:

a. The liner is protected from damage by potential missiles generated from a LOCA and main steam pipe break.

b. The liner strains are limited to those values that have been shown by past experience to result in leaktight pressure vessels and piping.

c. The liner is prevented from developing distortions sufficient to impair leak tightness.

5.2.3.3 Design Loads

The liner is designed with the capability to resist, without rupture, the compressive stresses due to the following loads:

a. Construction loads, particularly those which are applied to the liner before the concrete is placed.

b. Local thermal loads at hot process penetrations.

c. Thermal gradients.

d. Thermal shock loads due to cold sprays.

e. Local loads, such as structural supports, pipe supports, and restraints, etc.

f. Prestress loads.

g. Creep and shrinkage loads.

The fatigue analysis of the liner plate is made on the basis of the following considerations:

a. Thermal cycling due to annual outdoor temperature variations. The number of cycles for this loading is 40 for the plant life of 40 years. (Daily temperature variations do not penetrate a significant distance into the concrete shell to appreciably change the average temperature of the shell relative to the liner, and therefore, are not considered.)

b. Thermal cycling due to the containment interior temperature variations during heatup and cooldown of the reactor system. The number of cycles for this loading is assumed to be 500.

c. Thermal cycling due to the LOCA is assumed to be one cycle.

5.2.3.4 Permissible Stresses and Strains

The basis for establishing the allowable liner strains is the 1968 ASME Boiler and Pressure Vessel Code, Section III, Nuclear Vessels, Article 4.

The thermal stresses in the liner fall into the categories as considered in Article 4 of Section III. The allowable stresses in Figure N-415 (A) of the Code are for alternating stress intensity for carbon steels, with the temperature not exceeding 700°F.

To fulfill the criteria set forth in the 1968 ASME Code, Paragraph N-412 (m) 2, the liner is restrained against significant distortion of an angle grid anchor system. Materials are expected to be exposed to a maximum temperature of approximately 289°F under a LOCA condition which is well below the 700°F limit. The liner design also satisfies the criteria for limiting the strains on the basis of fatigue consideration. Figure N-415 (A) of Paragraph N-412 (n) of the ASME Code and its appropriate limitations are used as the bases for establishing the allowable strains for the liner.

Since the graph in Figure N-415 (A) does not extend below 10 cycles, 10 cycles were used for the LOCA conditions instead of one cycle.

The allowable strain from Figure N-415 (A), based on 10 significant thermal cycles on the LOCA conditions, would be approximately two percent. However, the maximum allowable membrane tensile or compressive strain is conservatively set at 0.5 percent. The maximum predicted strain in the liner during the LOCA conditions is found to be 0.25 percent in compression.

No maximum allowable compressive strain is set for the test condition because it is expected the value will be less than that experienced under the LOCA conditions. The maximum allowable tensile strain is 0.2 percent for the test conditions; the predicted value is nearly zero.

5.2.3.5 Design of Liner Plate Anchorage

The anchors are designed to preclude failure when subjected to the most severe loads or deformations. They are designed so that a missing or defective anchor will not jeopardize the overall integrity of the liner and anchorage system.

The following factors are considered in the design of the anchorage system:

a. The initial inward curvature of the liner between anchors due to fabrication and erection tolerances and inaccuracies.

b. Variations of anchor spacing.

c. Misalignment of liner seams.

d. Variations of plate thickness.

e. Variations of the yield strength of the liner plate materials.

f. Variations of the Poisson's ratio for the liner plate materials.

g. Variations of the anchor stiffness.

The anchorage system satisfies the following conditions:

a. The anchors have sufficient strength and ductility so their energy absorbing capability is sufficient to restrain the maximum force and displacement resulting from the condition where a panel with an initial outward curvature is adjacent to a panel with an inward curvature.

b. The anchors have sufficient strength to resist the bending moment that results when a panel with an initial outward curvature is adjacent to a panel with an inward curvature.

c. The anchors have sufficient strength to resist the radial pullout forces.

The proprietary topical report, "Consumer Power Company Palisades Nuclear Power Plant Containment Building Liner Plant Design Report B-TOP-1," constitutes the basic design approach used in Millstone Unit 2.

The following are the minor differences between the Millstone Unit 2 design and the one presented in the topical report.

a. The welding of the stiffeners is 3/16-6 x 12 rather than 3/16-4 x 12. This does not invalidate the analysis, because the spring constants used in the analysis are similar.

b. The stiffeners on the thickened plates are not welded with a double fillet weld as stated in the topical report. The 3/16-6 x 12 welding is used on all stiffeners. The topical report indicates that additional welding is not required to resist the loads.

c. The 1/4 inch liner material is ASTM A-285 Grade A. This plate has a specified yield strength of 24,000 psi which is lower than the values used in Topical Report, B-TOP-1. This would only tend to decrease the loads on the anchors, as stated in Section 3.4 of the report.

d. A self supporting dome is used on Millstone Unit 2. It is stiffened in two directions instead of one as stated in Section 2.2.2 of the report. Details of the dome are shown on Figure 5.2–6.

5.2.3.6 Design of Weldments

Paragraphs of UW-8 to UW-19, Subsection B, Section VIII of the 1968 ASME Code are used as a guide in the design of the weldment.

Inspection and testing of liner plate weldments during and after erection are discussed in Section 5.9.3.5.3.

Quality control of field welding electrodes are presented in Section 5.9.3.5.4.

Quality control procedures for field welding and nondestructive examination are defined in Section 5.9.4.

5.2.4 INTERIOR STRUCTURES

5.2.4.1 General

Design of the containment interior structures evolves from four basic systems: reactor coolant, main steam, engineered safety features and fuel handling.

The structures which house or support the basic systems are designed to sustain the loading cases as outlined in Sections 5.2.4.4.1 and 5.2.4.4.2.

The design bases are as follows:

a. The structures are capable of sustaining all operating loads, seismic loads, and thermal deformations.

b. Loads and deformations resulting from an LOCA and the associated effects on any of the basic systems are sustained and restricted so that propagation of the failure to any other system is prevented. In addition, a single failure in one of the cooling pipes of the nuclear steam supply system is restricted such that propagation of the failure to the other cooling piping is prevented.

Structural details for the supports for major Class I equipment such as the reactor vessel and steam generators, are shown in Figure 5.2–11 through 5.2–13. Typical details for the primary and secondary shield walls are shown on Figure 5.2–14.

5.2.4.2 Construction Materials

The following materials are used in the construction of the containment interior structures:

Concrete:

Primary shield walls	5,000 psi
Steam generator supports	5,000 psi
Secondary shield walls	4,000 psi
Refueling pool walls	4,000 psi
Reinforcing steel	ASTM A-615, Grade 60
Carbon steel plates	ASTM A-302, Grade B, A-441, and A-569
Stainless steel plates	ASTM A-240, Type 304
Stainless steel tubes	ASTM A-358, Type 304
Structural and miscellaneous steel	ASTM A-36 and A-441
Anchor bolts	ASTM A-307, A-325, and A-490

5.2.4.3 Design Loads

The following loads are considered in the design of the interior structures:

a. Dead loads

b. Live loads

c. Earthquake loads

d. Loss-of-coolant accident (LOCA) loads

e. Pipe rupture loads

5.2.4.3.1 Dead Loads

Dead loads consist of the weight of the concrete, structural steel, equipment, major piping, and electrical conductors. Equipment dead loads are those specified on the drawings supplied by the manufacturers of the various pieces of equipment. Major equipment supported by the interior structures are reactor vessel, steam generators, pressurizers, and the reactor coolant pumps.

5.2.4.3.2 Live Loads

Live loads for the design of the interior structures are:

Floor and equipment area	250 psf
Containment laydown area	1,000 psf

Equipment live loads are those specified on the drawings supplied by the manufacturers of the various pieces of equipment.

5.2.4.3.3 Earthquake Loads

Earthquake loads are predicated upon an operating base earthquake at the site having a horizontal ground acceleration of 0.09 g and a vertical acceleration of 0.06 g. In addition, a design basis earthquake having a ground acceleration of 0.17 g and a vertical acceleration of 0.11 g is used to check the design to ensure that there will be no loss of function.

Seismic response spectrum curves are given in Section 5.8.1.1, for both horizontal and vertical ground motions.

5.2.4.3.4 Loss-of-Coolant-Accident (LOCA) Loads

The maximum forces which result from a pipe rupture are based on the following:

a. A single break in any pipe under consideration is assumed to occur at one time.

b. The maximum area of the break is assumed to be an opening in the pipe equivalent to its cross-sectional area.

c. The maximum force is based on a free discharge from an open-ended pipe.

The minimum design pressure and temperature of the interior structures are equal to, respectively, the peak pressure and temperature occurring as a result of the complete blowdown of the reactor coolant due to a rupture of the reactor coolant system. This could be up to and including the hypothetical double-ended rupture of the largest pipe in the primary coolant system. The following effects associated with a LOCA are considered:

a. Thrust loads resulting from the rapid mass release from a pipe break in the reactor system or other systems, if the break occurs during a design incident.

b. Pressure build-up in locally confined areas such as the secondary shielding room, as described in Section 5.2.4.4.4.

c. Jet forces resulting from the impingement of the escaping mass upon the adjacent structure.

d. Pipe whipping following a pipe break in the reactor coolant system.

e. Rapid rise in ambient temperature and accompanying rise in ambient pressure.

f. Missiles as described in Section 5.2.5.1.1.

Leak-before-break (LBB) analyses of the reactor coolant system (RCS) main coolant loops, the pressurizer surge line, and unisolable RCS portions of the safety injection and shutdown cooling piping allows the exclusion of the dynamic effects associated with pipe ruptures in the above piping segments from the design basis. Dynamic effects of pipe rupture include the effects of pipe whipping, subcompartment pressurization and discharging fluids.

5.2.4.4 Design Criteria

In establishing the structural design criteria for the interior structures, consideration was given to a structure which would withstand the differential pressure within the cavities in the event of an accident, and to minimizing the effects of the pipe rupture force by the use of supports and restraints.

The ACI-318-63, "Building Code Requirements for Reinforced Concrete," and AISC "Manual of Steel Construction," 6th Edition, are used as design criteria for reinforced concrete and structural steel, respectively, except as noted in the following:

> Localized concrete yielding is permitted when it can be demonstrated that the yield capacity of the component is not affected, and that this small localized yielding does not generate missiles which could damage the structure. Full recognition is given to the time increments associated with these postulated failure conditions, and yield capacities are appropriately increased when a transient analysis demonstrates that the rapid strain rate justifies this approach. The walls are also designed to provide adequate protection for potential missile generation which could damage the containment liner.
>
> The yielding of concrete discussed above concerns itself with localized stress concentration under the load associated with a LOCA. The load due to pipe whipping followed a pipe break in the reactor coolant system is considered a concentrated load and is analyzed to ascertain that its local effect on the concrete surface will not result in changes in the member capacity. These high local stresses are not identified because of simplifications made in the design analysis. These high stresses are allowed because they occur in a very small percentage of the cross-section, are confined by material at lower stress, and would have to be considerably greater than the values allowed before significant local plastic yield would result.

The outline of the reactor coolant system and the primary supporting structure are presented in Figures 5.2–35 and 5.2–36. Localized concrete yielding is permitted at the point of impact resulting from any high energy pipe rupture. The high concrete stress is limited to the area of impact and is confined by concrete in compression at lower stresses, and is estimated conservatively to be not more than one inch in thickness within the secondary wall boundary. Since the area of high concrete stresses is in compression, no significant concrete cracking is expected. Pipe rupture is postulated at locations which result in the most critical conditions for designing the structures. A typical case is shown in Figures 5.2–35 and 5.2–36. Under this load, combined with other loads in the design load combination, a strip of concrete wall of width L as shown in Figure 5.2–36 was analyzed. A continuous span of secondary shield wall, supported at the buttresses and fixed at the refueling pool walls was assumed. Reinforcing steel was proportioned according to the results obtained.

The strength of the structures at working stress and overall yielding is compared to various loading combinations to ensure safety. The structures are designed to meet the performance and strength requirements under the following conditions:

a. At design loads.

b. At factored loads.

5.2.4.4.1 At Design Loads

This loading is the basic "working stress" design. The structure is designed for the following loading cases:

a. D + L

b. D + L + H

c. D + L + H + E

where:

D = dead loads

L = live loads

H = thermal loads under operating conditions

E = operating basis earthquake

5.2.4.4.2 At Factored Loads

The structure is checked for the factored loads and load combinations as follows:

a. $C = 1/\phi\ (1.25D + 1.25L + 1.0\ R + 1.25E)$

b. $C = 1/\phi\ (1.25D + 1.25L + 1.25\ H + 1.25E)$

c. $C = 1/\phi\ (1.0D + 1.0L + 1.0\ R + 1.0T_I + 1.0E')$

d. $C = 1/\phi\ (1.0D + 1.0L + 1.0\ H + 1.0E')$

e. $C = 1/\phi\ (1.0D + 1.0L + 1.0\ P + 1.OT_I + 1.0E')$

where:

C = required capacity of the structure to resist factored loads

ϕ = capacity reduction factor

D = dead loads

L = live loads

H = thermal loads under operating conditions

P = differential pressure due to a double-ended pipe rupture

T_I = loads due to thermal gradient produced by a double-ended pipe break

E = operating basis earthquake loads

E′ = design basis earthquake loads

R = loads due to pipe rupture (includes both jet and pressure differential forces)

5.2.4.4.3 Thermal Gradients

The thermal gradients in the interior structures are maintained at a low level so they have very small structural effects on the concrete walls. Nevertheless, these effects are considered in the design.

a. Primary shield walls.

Based on the reactor vessel and cooling piping heat loads and 100 percent of cooling air, the temperature within the primary shield walls is approximately 7°F higher than the air temperature, due to nuclear heating. Cooling air from the CAR coolers is supplied to the bottom of the reactor cavity, primary shield walls, reactor vessel supports and the ex-core detectors. Calculations, which support the permanent reactor cavity seal project and neutron shield modification, have estimated that the cavity cooling air limits the maximum temperature of the primary shield walls to less than 150°F.

b. Secondary shield walls, steam generator pedestals, and refueling pool walls.

The ventilation system will eliminate the temperature gradients across the secondary shield walls, the refueling pool walls, and the operating floor. The ambient temperature inside the containment varies between 105°F and 120°F under operating conditions.

5.2.4.4.4 Differential Pressures

The differential pressure-time curves across the primary shield wall and the annular space between the reactor coolant pipe and the pipe sleeve extending through the reactor cavity wall (biological shield) have been excluded from the design basis, as all high energy lines within the reactor vessel cavity have supporting NRC approved leak before break analyses. The Bechtel COPRA computer program (NS731-NE576) is used to calculate the differential pressure-time curves across the secondary shield walls. The calculations are based on conservation of mass, momentum, and energy.

The ensuing flow from the compartment follows the orifice flow relations with the entrance and friction losses included in the flow coefficient for each case.

The following is a summary for the input parameters used for the computer analysis of steam generator compartment pressurization:

Containment free volume (cubic feet)	1.899 x 10^6
Initial containment temperature (°F)	120
Initial containment pressure (psia)	14.7
Initial containment humidity (%	30
East Steam Generator Compartment	
Volume (cubic feet)	62,000
Vent Area (square feet)	2,800
West Steam Generator Compartment	
Volume (cubic feet)	54,100
Vent Area (square feet)	2,620

Neither steam line nor feedwater line breaks were analyzed in the steam generator compartments. Therefore a double-ended guillotine break in the hot leg, with a restricted break area of 10.78 square feet, is considered, which provides the bounding rate of energy and mass releases.

The Pressurizer Compartment was originally designed for a 22 psi differential pressure based on its contiguous boundary with the east steam generator cavity. However, due to a modified, semi-open blockhouse design that was implemented to support the replacement of the original pressurizer and the adoption of Leak-Before-Break methodology to eliminate pressurization effects due to the primary coolant piping, the design differential pressure for this structure is currently based on a double-ended feedwater break. Utilizing a break size comparison between the less energetic feedwater break case (2.97square feet) and the previously analyzed hot leg break case (10.78 square feet) provides a bounding estimate for the differential pressure effects from the feedwater break.

Results of compartment pressurization analyses are listed below. Conservative estimates of compartment geometries, combined with the conservative calculational model assumed in COPRA, result in differential pressures across compartment walls that are inherently larger than can actually occur. Consequently, no additional safety margins are necessary.

COMPARTMENT	PRESSURES	
	DIFFERENTIAL (PSI)	DESIGN DIFFERENTIAL (PSI)
Steam Generator Cavity (East)	8.7	22
Steam Generator Cavity (West)	9.95	22
Pressurizer	2.4	3

5.2.5 SPECIFIC DESIGN TOPICS

5.2.5.1 Missile Protection

5.2.5.1.1 Design Criteria Inside the Containment

High pressure reactor coolant system components, which could be the source of missiles, are screened either by the concrete shield walls enclosing the reactor coolant loops, by the concrete operating floors, or by special missile shields to block the passage of any missiles and prevent them from striking the wall and dome of the containment. All potential missile sources are oriented so that the missiles are intercepted by the shields or structures provided. A shield is provided over the control rod driving mechanism to block the passage of any missiles generated as a result of a postulated fracture of the nozzle.

Missile protection inside the containment is provided to comply with the following criteria:

a. The containment liner is protected from a loss of function due to damage by missiles as might be generated in a LOCA for piping break sizes up to and including the double-ended severance of the largest reactor coolant pipe.

b. The engineered safety features, systems, and components required to maintain the integrity of the containment are protected from a loss of function due to damage by the following missiles:

   1. Valve stems

   2. Valve bonnets

   3. Instrument thimbles

   4. Various types and sizes of nuts and bolts

Table 5.2-10 lists the spectrum of potential missiles from inside the containment, their kinetic energies, weights and leading cross-section configurations:

The following methods are used to implement the missile protection criteria:

a. Components of the reactor coolant system are examined to identify and classify missiles according to the sizes, shapes, and kinetic energy so as to analyze their effects.

b. Missile velocities are calculated considering both the fluid and mechanical driving forces that exist during missile generation.

c. The reactor coolant system is surrounded by reinforced concrete and steel structures which are designed to withstand the forces associated with the double-ended rupture of a reactor coolant pipe and to stop the missiles.

The structural design of the missile shields takes into account both the static and the impact loads and is based upon the state-of-the-art of missile penetration data.

Leak Before Break (LBB) analyses of the reactor coolant system (RCS) main coolant loops, the pressurizer surge line, and unisolable RCS portions of the safety injection and shutdown cooling piping allows the exclusion of the dynamic effects associated with pipe ruptures in the above piping segments from the design basis. Dynamic effects of pipe rupture include the effects of pipe whipping, subcompartment pressurization and discharging fluids.

Certain postulated incidents such as the massive, rapid failure of the reactor vessel, steam generators, pressurizers, and the main coolant pump flywheels and casings are considered incredible because of the material characteristics, inspections, quality control during fabrication, and the conservatism in design as applied to the particular components. These same factors also apply to the stems and bonnets of both motor-operated shutdown cooling suction valves located inside containment. Both valves have been subjected to detailed structural and functional analysis, and the stem and bonnets have been found to not be credible missiles.

In establishing the credibility of any missile source, the use of redundancy of load paths, such that no single failure could lead to a missile ejection, has been credited as the basis for adequate protection from missile generation. In the case of missiles originating from valves, the valve stem is considered a potential missile only when it is not back seated and where no air or motor operator exists that would interfere with the ejection of the valve stem. Valve bonnets are not considered as a source of missiles when the flanges and bolts are designed in accordance with ASME Section III and the torque is controlled during the tightening process by approved plant procedures. Valve bonnet missiles are also not considered credible when the bonnet is welded to the valve body or in cases where the bonnet is integral with the body of the valve. While the failure of single bolts and nuts is considered credible, they are considered a negligible concern due to the small amount of stored elastic energy that they process.

5.2.5.1.2 Design Criteria Outside the Containment

Missile protection outside the containment is provided to comply with the following requirements:

a. The containment steel liner plate and penetrations are protected from the loss of function due to damage by tornado borne missiles.

b. All engineered safety features piping which penetrates the containment, and which is required to maintain the containment integrity, is protected from a loss of function due to tornado borne missiles.

c. All components required to maintain the containment integrity, or whose failure would result in the uncontrolled release of radioactivity, are protected from a loss of function due to damage by tornado borne missiles.

Protection is provided for the following three types of tornado borne missiles:

a. A fir plank, 4 inches by 12 inches by 12 feet., weighing 105 pounds and traveling end-on at a speed of 250 mph.

b. A passenger auto (4,000 lb.) impact velocity of 50 mph not more than 25 feet above grade with a contact area of 20 square feet.

c. A 3 inch by 10 foot long (ASA Schedule 40) pipe (72 pounds) traveling end-on at a speed of 100 mph at any elevation on the structure.

Appendix 5.D addresses an expanded spectrum of tornado missiles which are additionally considered in the design of Tornado Missile Protection that was requested by the NRC. The design criteria presented in Appendix 5.D is an expansion of the tornado missile protection criteria of this Section and does not delete any previous requirements. The fir plank of this Section and the wooden beam of Appendix 5.D are the same missiles.

Analysis of the effect of the impact of the missiles on structures is based on the methods presented in the NavDocks P-51, "Design of Protective Structures-a New Concept of Structural Behavior," published by U.S. Bureau of Yards and Docks, August 1950, Washington, D.C.

Provisions to tie down all slabs, blocks, or partitions outside of containment which are potential seismic or tornado missiles are described as follows:

1. <u>Slabs and Blocks</u> - Slabs and blocks which are potential seismic or tornado missiles are those items which fall into the category of hatch covers or removable partitions and lie within the Class I structures in areas containing Class I equipment or components.

   All removable wall panels are tied structurally to the building by retaining members and reinforcing within the wall panel. In all cases, removable wall panels are designed to remain in place and intact, sustaining seismic or pressure loadings appropriate to the elevation within the buildings. Hatch covers which do not serve as vents during build-up and decay of pressures which would possibly occur during a tornado, are secured with fastening devices which will resist all design forces due to such loading. Hatch covers which serve as vents are designed to open to relieve internal pressures, but are provided with mechanical retaining devices which prevent the element from becoming a missile during seismic or tornado occurrences.

2. Partitions - The partitions and walls that are located within areas housing Class I equipment or components are reinforced vertically and horizontally and are anchored around the perimeter of the elements to the building structure. All partitions within these areas are constructed of either reinforced concrete or reinforced concrete masonry units. The design provides structural adequacy to sustain appropriate seismic or differential pressures resulting from a tornado occurrence.

   Earthquake loads are defined in Sections 5.2.2 and 5.8 of the FSAR.

   Tornado pressure design criteria is defined in Section 5.2.2 of the FSAR.

5.2.5.1.3 Turbine Missile Consideration

The provisions to control turbine overspeed for the Millstone Unit Number 2 turbine generator are documented in Amendment 11 to the Millstone Unit Number 2 Preliminary Safety Analysis Report. Amendment 11 was filed with the Commission on June 12, 1970. Based on the overspeed protection and controls provided, the applicants conclude that the probability of destructive overspeed as a possible cause of turbine failure is extremely low.

Turbine missiles were discussed in Amendment 7 to the Millstone Unit Number 2 Preliminary Safety Analysis Report filed with the Commission on April 24, 1970.

The MP2 LP rotors, which were of the original shrunk-on wheel design, were replaced during 2R15 with monoblock rotors. The monoblock design eliminates the keyway between the rotor shaft and the shrunk-on wheels. In accordance with NRC approved GE methodology (ref. NUREG-1048), the dominant mode of wheel failure and resulting missile generation was identified to be brittle fracture, due to keyway stress corrosion cracking occurring at or near normal operating speed. The requirement for the unfavorably oriented turbine (the case of MP2), in NUREG-1048 is that the probability be less than 10^{-5} per year. Since the monoblock rotor design eliminates the brittle fracture mode, the probability for a burst and a missile at normal operating speed is negligible. The remaining probability is a function of a ductile failure that is dependent upon the probability of a turbine overspeed event. The probability of a complete control system failure was determined to be 1.7×10^{-6} with a GE Mark I control system (reference DCR M2-03001, Rev. 0, Attachment 2). As part of the GE Mark VIe turbine controls digital upgrade (reference Design Change MP2-10-01016, Attachment 15), GE determined that the probability of an overspeed event is less with a Mark VIe control system than with a Mark I control system.

Refer to Figure 1.2–15 of the FSAR, Section E-E through the turbine building. When viewed from the south, the turbine generator unit rotates in a counter-clockwise direction. As can be seen in this figure, the safety-related structures such as the containment, diesel generator rooms, auxiliary and control buildings, are protected from low angle missiles by the massive turbine generator pedestal. The probability of a high angle missile returning under gravity forces to strike any portion of the plant is extremely low. This coupled with a review of the plant layout containing the components and systems required to bring the plant to a safe shutdown condition without off site power available indicates that turbine generated missile damage will not preclude the safe shutdown of the plant.

5.2.5.2 Post-Tensioning Sequence

The detailed stressing sequence is based on the design requirements to limit the membrane tension in the concrete to $1.0(f'_{ci})^{1/2}$ and to minimize unbalanced loads which would produce differential stresses in the structure. Finite element mesh of the containment shell used to establish the post-tensioning sequence is shown on Figure 5.2–18.

Bechtel provides the post-tensioning system vendor with the prestressing force requirements, the anticipated concrete elastic shrinkage and creep, and the numerical order in which the tendons are to be stressed. The vendor then incorporates all this information, along with losses due to tendon relaxation, friction, and anchorage losses, if any, to establish the initial jacking force for each sequential operation.

Force measurements are obtained by measuring the elongation of the prestressing tendons and comparing it with the calculated forces indicated by the jack-dynanometer or pressure gauge. The latter represents the pressure in the jack with a tolerance of ± 2 percent. The calibration of the force-jack pressure gauge or dynanometer combination is traceable to the National Bureau of Standards and is so certified prior to the application of prestressing forces. Pressure gauges and jacks so calibrated are always used together.

During stressing, records are kept on the pressures applied and the corresponding elongations obtained. Jack-dynanometer or gauge combinations are checked against elongation of the tendons and any discrepancy exceeding ± 5 percent of the computed values utilizing the average load elongation curve is corrected and documented.

5.2.5.3 Differential Displacement Between Structures

A differential settlement of one-eighth inch between the containment and the auxiliary building foundations is assumed for design. Effects of the dynamic displacement of adjacent structures due to seismic disturbances are included in the analysis described in Section 5.8. It also includes the rocking of structures on dynamic elasticity of the foundations. The maximum and minimum values of displacement are taken as the separation of structures due to movement in opposite directions, both vertically and horizontally. The maximum differential movement at the level of the auxiliary building roof is expected to be in the order of ± 1.5 inches. The maximum displacements at the level of the auxiliary building roof and the containment structure at the same elevation were calculated to be three-eighth and one-eighth inches, respectively, when subject to a design-basis earthquake. Provisions for 1.5 inches at all junctions between the containment structure and the auxiliary building were considered to be a conservative value. Provisions are made at all junctions between the containment and the auxiliary building to permit the differential movement to take place with no significant transfer of loads to the containment.

Piping flexibility analyses include the effect of the differential movement between structures, as well as the effect of seismic acceleration on the piping and its contained masses.

5.2.5.4 Polar Crane for the Containment

The polar crane is designed to meet the loading requirements of the applicable portions of the Electric Overhead Crane Institute (EOCI) "Specification Number 61 for Electric Overhead Traveling Cranes," except that the earthquake loading is the seismic response of the containment at the crane supports. In addition, the crane bridge and trolley are provided with mechanical guides to prevent possible derailment at the design basis earthquake. Furthermore, the polar crane is designed so that even in the unlikely event of the failure of a rail, the crane will remain on the supports.

Typical details for the polar crane supports are shown in Figures 5.2–9 and 5.2–15.

5.2.5.5 Containment Maintenance Truss

A maintenance truss is provided in the containment for use in the maintenance of containment spray piping and ease of inspection of the interior of the dome liner plate.

It is supported at the top by a large pin embedded in the containment dome. The bottom of the truss rests on the polar crane runway rail. The truss is moved around the containment by the polar crane.

The truss is a Class I item designed for all postulated load conditions that are used for the containment design.

5.2.5.6 Unit 2 Stack

The Unit 2 stack extends approximately 12 feet above the enclosure building and serves as a ventilation discharge duct. It is designed to the following factored load equations. The seismic response of the stack is taken as that of the enclosure building in which the stack is attached:

$$C = 1/\phi\ [(1.00 + 0.05)\ (D) + 1.25\ E]$$

$$C = 1/\phi\ [(1.00 + 0.05)\ (D) + 1.25\ W]$$

$$C = 1/\phi\ [(1.00 + 0.05)\ (D) + 1.0\ E']$$

where:

C = required capacity of the structure to resist factored loads

ϕ = capacity reduction factor

D = dead loads of structure

E = operating basis earthquake

E′ = design basis earthquake

W = wind or tornado loads

5.2.5.7 Pipe Whip Protection Criteria

Details may be found in Section 6.1.4.1.1.1.

5.2.5.7.1 Methods of Protection Against Pipe Whip

Protection against critical pipe failures as defined in Section 6.1.4.1 of critical safety-related targets is assured by one of the following means:

a. Plant layout is arranged such that the targets are physically separated from the effects of potential pipe failures. (See Section 6.1.4.1.1.2.)

b. Either functional structure (such as walls, floors, etc.) or specially designed barriers are provided. Such barriers are also used to protect against the effects of jet impingement forces.

c. In areas where unrestrained motion of a pipe failure as defined in Section 6.1.4.1.1.2 could hit a safety-related target, pipe restraints are employed. Spacing of the restraints is as follows:

(i) Along a continuous run of restraints the spacing of restraints shall be:

$$L = (4M_p/F_j)$$

where:

L = center-to-center span of restraint

M_p = plastic moment of the pipe, considering the effect of longitudinal stresses resulting from the internal pipe pressure. Allowable stress is taken 1.2 times the lower limit yield stress as defined in ASME Section III, “Nuclear Power Plant Components,” Paragraph NB-3225.

F_j = jet impingement force as defined in Subsection d.

(ii) The maximum distance of a restraint from the working point of an elbow for a circumferential break is:

$$L = (M_p/F_j)$$

d. If an analysis demonstrates a component can withstand the loading generated by either the jet impingement force, or the effect of a pipe whip impact, protection is assured.

The jet impingement load acting on a target shall be taken as:

$$F_j = (DLF)\, PK_\phi A_{TE}$$

where:

F_j = jet impingement force

DLF ≤ 2.0, Dynamic Load Factor to account for the amplification resulting from the sudden impulse nature of the jet force. The use of DLF values lower than 2.0 require justification.

P = the steady state blowdown pressure established for the specific break location, with consideration of internal fluid parameters based on the maximum Normal Operating Condition and piping internal friction, as appropriate.

$K\phi$ = 1.0, for cold water and non-flashing/subcooled fluid blowdown.

$K\phi$ = jet dispersion correction factor for steam and steam/water mixtures.

$= [1 + (2x/D_e) \tan \phi]^{-2}$, for $x < 5\ D_e$

$= 0.131$, for $x \geq 5\ D_e$

x = distance from pipe break to target

D_e = effective diameter of break taken as internal area of pipe

ϕ = dispersion half angle-taken as 10°

A_{TE} = effective impingement area of target; i.e., area perpendicular to axis of jet spray. The value depends upon the relative size and orientation of the jet and the target. For pipes and targets with curved surfaces, an appropriate shape factor may be used.

5.2.5.7.2 Design Procedures for Restraints and Barriers

a. Pipe Restraints:

Reaction forces on restraints resulting from pipe breaks are taken to be:

$$F_r = 2.0\ P \times A$$

where

P = maximum normal operating pressure

A = internal cross-sectional area of pipe

The factor of two accounts for the dynamic amplification resulting from the sudden impulse nature of the jet force and its effect on supporting restraints. A lower value may be used if justified.

All restraints are designed using the working stress method. Allowable bending and tension stresses are taken as 1.2 times the lower limit yield stress, after the ASME Section III, “Nuclear Power Plant Components,” Paragraph NB-3225. Other stresses are based upon AISC, “Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings,” 1963, Section 2.0. However, allowable stresses for welds and bolt loads are reduced to 90 percent the specified allowable values.

The structures supporting the pipe restraints are designed for the reaction load (F_r) as described in Section 5.2.2 for the Containment, Section 5.2.4 for the Containment Internals, and Section 5.4.3 for the Auxiliary Building.

b. Pipe Impact

Impact from unrestrained pipe motion is analyzed as follows:

The pipe is assumed to swing about its nearest restraint in a configuration that produces the worst analytical condition. Impact energy is taken to be:

$$E_I = KE - E_H$$

where:

KE = (jet force) x (arc of swing)

E_H = (plastic moment of pipe) x (angle of swing)

This information is then utilized to compute the structural effects on barriers. To account for local deformation the methods presented in the NAVDOCKS P-51, “Design of Protective Structures - a New Concept of Structural Behavior,” published by the U.S. Bureau of Yards and Docks, August 1950, Washington, D.C., and the Stanford Research Institute Formula as presented in Reference 5.2-5.

Design criteria for barriers are based upon energy methods, utilizing ductility ratios of 10 and 5, respectively, for steel and concrete in tension or flexure. The method of analysis is similar to that presented in Reference 5.2-6. Allowable stresses are taken as defined in AISC, “Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings,” Section 2.0 and

ACI 318-63, Part IV-B. In those instances where functional structure is utilized as a barrier, the loading condition is as determined in either Section 5.2.2 for the Containment, Section 5.2.4 for the Containment Internals, and Section 5.4.3 for the Auxiliary Building.

5.2.5.8 Jib Crane for Containment

The Jib Crane for Containment is designed and fabricated in accordance with the requirements of ASME NOG-1 “Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder)”, 2004 Section 4100 and 4300 for structural and seismic analysis, Section 4200 for materials, welded and bolted connections, and Section 7000 for inspection and nondestructive examination testing. Areas not covered by ASME NOG-1-2004 conform to the requirements of CMM 70-2010 to the extent applicable to a jib crane.

The mounting structure attaching the crane to the pressurizer cubicle Is designed to meet specific seismic requirements (Seismic II/I) such that in the event of a design basis earthquake, the crane will retain its structural integrity. A boom support structure, also designed to meet seismic criteria (Seismic II/I), provides the support to secure the crane boom in the stored position to the top of the pressurizer cubicle In the event of a design basis earthquake. The design of the Jib Crane for Containment on the pressurizer cubicle is based on the following criteria.

- The crane is only operated during outage conditions. This dictates that the 3 psi accident condition for which the pressurizer cubicle is designed will not occur concurrently with the crane operation of the Jib Crane.

- During plant operation, the crane is in a stored position, tied down, and that seismic considerations are accounted for.

- The crane is operated only during outage conditions and utilizes predefined safe load paths approved in station heavy load procedures which were developed to meet NUREG 0612 requirements. Components/parts need not be operational following a seismic event, but are designed to retain structural integrity during an OBE or SSE.

5.2.6 CONTAINMENT PENETRATIONS

5.2.6.1 Types of Penetrations

All penetrations are pressure resistant, leak-tight, welded assemblies, which are fabricated, installed, inspected, and tested in accordance with the ASME Nuclear Vessel Code, Section III, 1971 and the ANSI Nuclear Piping Code B 31.7.

5.2.6.1.1 Electrical Penetrations

The electrical penetration assemblies through the wall of the containment structure form part of the containment pressure boundary and carry the (NPT) stamp in accordance with ASME Section III (1971). The low voltage power and control modules are mounted in a stainless steel header plate and are designed to meet or exceed all requirements of IEEE Standard 317, 1976.

The medium voltage electrical penetration assemblies are double sealed modules mounted in a stainless steel binder plate and are designed to meet or exceed all requirements pursuant to IEEE 317, 1971, “Electrical Penetration Assemblies in Containment Structures for Nuclear Fueled Power Generating Stations.” In addition to complete electrical prototype tests as described in Section 8.7.2.2, prototype tests are performed which include leakage integrity of 1 x 10^{-6} standard cc/second of dry helium under post-accident environmental conditions, seismic integrity, thermocycling, and simulated field installation. Replacement medium voltage electrical penetration modules are designed to meet or exceed all requirements pursuant to IEEE 317, 1976.

Electrical aspects of these penetrations are described in Section 8.7.2.2.

5.2.6.1.2 Piping Penetrations

Single barrier piping penetrations are provided for all piping passing through the containment walls. Carbon steel pipe sleeves of adequate diameter to allow passage of pipe and insulation where required are supplied.

The closure, forming the single barrier, consists of modified pipe caps, double-flued heads, or closure plates welded to the wall sleeves. The applicable closure types are shown on Figure 5.2–8.

The design data for the containment wall sleeves are included in Section 5.2.3.

5.2.6.1.3 Equipment Hatch and Personnel Lock

An equipment hatch, 19 feet in diameter, is provided to permit the transfer of equipment up to and including the size of the reactor vessel head, into and out of the containment. It is fitted with a double-gasketed flange around the dished door to minimize leakage in the unlikely event of a LOCA. Typical details of the equipment hatch are shown in Figure 5.2–4 and 5.2–10.

In addition to the (20) internal bolting assemblies used to secure the equipment hatch during normal operating modes, the equipment hatch has also been outfitted with four external swing bolt attachments which are welded to the equipment hatch ring flange and hatch barrel liner plate. The external attachments provide a method of securing the hatch from the outside during non-power operation.

A personnel lock is also provided for access into and out of the containment. The personnel lock is equipped with double doors and a quick acting type, equalizing valve which connects the lock with the interior and exterior of the containment to equalize the pressure in the two systems when personnel are either entering or leaving the containment.

The two doors in the personnel lock are interlocked to prevent both being opened simultaneously, and to ensure that one door is completely closed before the opposite door can be opened. Remote indicating lights and annunciators in the control room indicate the operational status of the door. Provision is made to bypass the interlock system and leave the doors open during plant cold shutdown. The lock interior is provided with lighting and a communication system, each operating from an external supply.

The doors on the Personnel Lock and Equipment Hatch are provided with double gaskets along the closure surfaces. Using the pressure taps furnished, the air space between these two gaskets may be pressurized and checked to assure leak tightness in accordance with the Technical Specifications.

In addition, the Personnel Lock has been designed to withstand the pressurization of the lock chamber to the pressure of 54 psig. During the test, the door on the containment side of the chamber is held closed by a special yoke installed only for the test. This pressurization of the chamber assures the leak-tightness of the door penetrations and the outside lock door under the peak pressure conditions.

Typical details of the personnel lock are shown on Figures 5.2–5 and 5.2–10.

5.2.6.1.4 Fuel Transfer Tube

A fuel transfer tube is provided for fuel movement between the refueling canal in the containment and the transfer canal in the auxiliary building.

The penetration consists of a 36 inch diameter stainless steel tube installed inside a 42 inch sleeve. The transfer tube is fitted with a double-gasketed blind flange in the refueling pool and a standard gate valve in the transfer canal. This arrangement prevents leakage through the transfer tube in the event of an accident. The 42 inch sleeve is welded to the containment liner. A bellows expansion joint is provided on the 42 inch sleeve to compensate for differential movement. Typical details of the fuel transfer tube are shown in Figure 5.2–10.

5.2.6.2 Design of Penetrations

5.2.6.2.1 Design Criteria

All piping passing through the containment walls are permanently welded to the wall sleeves, forming an extension of the containment:

To preserve the integrity of the containment, provisions are incorporated to prevent internal and external forces exerted by connecting piping on the wall sleeves from fracturing or breaching the containment pressure boundary. Additionally, protection against missiles is provided for the penetration piping and valving inside and outside the containment.

Further protection of each line, necessary to preclude the loss of pipe structural integrity between penetration and the first valve, is accomplished by shortening the exposed length of pipe and installing the first valve as close as possible to the internal or external wall of the structure, depending upon valve operating and maintenance clearances. Design bases which apply to the provision of automatic and manual isolation valves in the penetration lines are contained in Section 5.2.6.2.4.

5.2.6.2.2 Design of High-Temperature Penetrations

High-temperature piping penetrations consist of two for feedwater, two for main steam, and two for steam generator blowdown. These have a maximum operating temperature range between 435°F and 550°F. Thermal insulation is provided in the air gap between the pipe and penetration liner sleeve. The combination of insulation and penetration cooling is designed to restrict maximum temperature in the concrete to 150°F.

For the condition created by loss of penetration cooling, the maximum steady state temperature in the concrete is 300°F at the penetration surface and decreases to 120°F at a maximum radial depth of 48 inches in the containment wall (Section 9.9.4.4.1). This thermal gradient produces localized compressive stresses in the concrete immediately surrounding the penetration and low tensile stresses distributed some distances from the sleeve. These compressive stresses plus the restraint provided by the prestress loads minimize the effect of elevated temperatures on the concrete (Reference 5.2-4).

In addition, the normal operating temperature of 150°F continues to cure and dry concrete near the high-temperature penetrations. Since dry concrete is only slightly affected by high temperatures, normal operation is beneficial and reduces strength losses from a temperature rise.

On this basis, the concrete in the localized area around the penetrations can withstand 300°F without significant strength loss.

5.2.6.2.3 Penetration Materials

The materials for containment penetrations, which include mechanical, electrical, and access penetrations, conform to the requirements of the ASME Nuclear Vessel Code and ANSI B31.7-68 including Code Case 1427.

As required by these codes, carbon steel penetration materials, which form the containment pressure boundary meet the necessary Charpy V-notch impact test values at a temperature 30°F below the lowest service temperature. Impact testing for mechanical piping systems is performed at + 20°F, for uniformity.

a. Mechanical Penetration Material Specification

Penetration Sleeve (Pipe)	SA-333, Grade 6
(Rolled Plate)	SA-516, Grade 60
Penetration Reinforcing Rings	ASTM A-516, Grade 60
Penetration Sleeve Reinforcing	ASTM A-516, Grade 60
Bar Anchoring Rings and Plates	ASTM A-516, Grade 60
Rolled Shapes (Nonpressure Boundary)	ASTM A-36
System Piping	ASTM A-333, Grade 6 SA 335 GR P22

	ASTM A-312 TP 304 SA 234 GR WP22
	ASTM A-376 TP 316
	ASTM A-376 TP 304
	ASTM A-155 Gr KCF 70
	ASTM A-106, Grade B
Isolation Valves	ASTM A-182, Grade F316
	ASTM A-105, Grade II
	ASTM A-351, Grade CF8, CF8M
	ASTM A-216, Grade WCB
	ASTM A-350, Grade LFI
	ASTM A-516, Grade 70
	ASTM A-515, Grade 70

b. Electrical Penetrations

The penetration sleeves to accommodate the electrical penetration assembly canisters are SA-333 Grade 6 carbon steel, pipe schedule 80.

c. Access Penetration Materials

The containment equipment hatch insert, flanges, and head are fabricated of ASTM A-516 Grade 60 carbon steel plate.

The personnel lock is fabricated on ASTM A-516 Grade 60 carbon steel plate.

5.2.6.2.4 Provisions for Isolation Valves

Those piping systems which penetrate the containment are provided with isolation valves which conform with the requirements of 10 CFR Part 50, General Design, Criteria 54, 55, 56, and 57.

These provisions are described in detail in Section 5.2.7.

5.2.6.3 Installation of Penetrations

Penetration sleeves are welded to the liner plate and embedded in the concrete containment wall as shown in Figure 5.2–8. All welding and approved welding procedures used are in conformance with the requirements of ASME Section IX and ASME Section III, 1971, Nuclear Vessels.

Piping systems passing through the sleeves are installed as shown in Figure 5.2–8. The single barrier closure provided is welded to the sleeve, using the same requirements noted previously.

5.2.6.4 Testability of Penetrations

All containment penetrations are subject to an initial leak rate test and periodic leak rate testing in accordance with the requirements of 10 CFR Part 50, Appendix J.

Provisions have been made to pressurize the containment pressure boundary for leak rate testing as described in Section 5.2.8.1.

Valve arrangements incorporate provisions for leak testing of the piping systems penetrations, the containment and the isolation valves for these systems, in accordance with Criterion 54 of 10 CFR Part 50, Appendix A, as necessary to perform the leak rate testing required by 10 CFR Part 50, Appendix J.

These provisions are defined in Sections 5.2.7.1 and 5.2.7.2.

5.2.7 CONTAINMENT ISOLATION SYSTEM

5.2.7.1 Design Bases

5.2.7.1.1 Functional Requirements

The containment isolation system functions to provide a double barrier to prevent leakage through all containment fluid penetrations. As a result, no single, credible failure, or malfunction of an active component can result in loss-of-isolation capabilities or intolerable leakage.

5.2.7.1.2 Design Criteria

The following criteria have been used in the design of the containment isolation system according to 10 CFR Part 50:

a. Piping systems penetrating primary reactor containment shall be provided with leak detection and isolation.

b. 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 as necessary per the testing requirements of 10 CFR Part 50, Appendix J.

c. Each line that is part of the reactor coolant pressure boundary and penetrates primary reactor containment shall be provided with containment isolation valves 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. The valves are:

    1. One locked closed isolation valve inside and one locked closed isolation valve outside the containment; or

2. One automatic isolation valve inside and one locked closed isolation valve outside the containment; or

3. One locked closed isolation valve inside and one automatic isolation valve outside the containment. A simple check valve may not be used as the automatic isolation valve outside the containment; or

4. One automatic isolation valve inside and one automatic isolation valve outside the containment. A simple check valve may not be used as the automatic isolation valve outside the containment.

d. Each line that connects directly to the containment atmosphere and penetrates primary reactor containment shall be provided with containment isolation valves 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. The valves which are used are:

1. One locked closed valve inside and one locked closed isolation valve outside the containment; or

2. One automatic isolation valve inside and one locked closed isolation valve outside the containment; or

3. One locked closed isolation valve inside and one automatic isolation valve outside the containment. A simple check valve may not be used as the automatic isolation valve outside the containment; or

4. One automatic isolation valve inside and one automatic isolation valve outside the containment. A simple check valve may not be used as the automatic isolation valve outside the containment.

e. Each line that penetrates the 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.

f. Isolation valves outside of the containment shall be located as close to the containment as practical. The automatic isolation valves shall be designed to take the position that provides greater safety upon loss of actuating power.

g. 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 in-service inspection, protection against most 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.

5.2.7.2 System Description

5.2.7.2.1 System

Containment isolation data is listed in Table 5.2-11. The containment isolation system is shown schematically in Figures 5.2–27 through 5.2–34.

The containment isolation system closes all fluid penetrations not required for operation of the engineered safety feature systems, to prevent the leakage of radioactive materials to the environment. Fluid penetrations serving engineered safety feature systems also meet the design basis double barrier criteria, but are not closed by the containment isolation signal.

The location and types of containment isolation valves meet the requirements of Appendix A of 10 CFR Part 50, General Design Criteria 54, 55, 56, and 57 as noted below in parentheses.

The fluid penetrations which may require isolation after an accident are categorized (and listed in Table 5.2-11 under the column heading “Pene. Category type”) as follows:

Type P - Lines that connect to the reactor coolant pressure boundary (Criterion 55 of 10 CFR Part 50, Appendix A).

These lines are provided with two isolation valves. One valve is inside the containment and the other is outside, located as close as practical to the containment structure. For safety injection/shutdown cooling where flow direction is inward, the inside valve is a check valve and the outside isolation valve is capable of remote operation. Otherwise, the inside valves are either remotely operated or locked closed valves. The outside valves are always either remotely operated or locked closed.

Type O - Lines that are open to the containment internal atmosphere (Criterion 56 of 10 CFR Part 50, Appendix A).

These lines are provided with similar containment isolation valve arrangements as described for Type P penetrations.

Type N - Lines that are neither connected to the reactor coolant pressure boundary nor open to the containment atmosphere but do form a closed system within the containment structure (Criterion 57 of 10 CFR Part 50, Appendix A).

These lines are provided with single isolation valves located outside the containment as close to the containment structure as practical. These valves are either remotely operated or are locked closed manual valves.

Prior to and during the construction and licensing of Millstone Unit 2, a system inside containment was defined as a closed system if the system neither communicated with the primary coolant nor the containment atmosphere. All Type N penetrations listed in Table 5.2-11 are categorized based on this definition and are GDC compliant. However, some of the systems associated with Type N penetrations (primary make-up water system, nitrogen supply system, waste gas header, and RBCCW system) are not in strict compliance with the criteria for a closed system as defined by the current standards (SRP 6.2.4 and ANS 56.2/ANSI N271-76). The current standards for a closed system inside containment call for them to be Class 2. The RBCCW system is fabricated to Safety Class 3 requirements in accordance with the acceptance criteria for the systems in effect at the time it was designed. NRC has accepted that for a low energy system such as RBCCW, the differences in safety classes 2 and 3 in terms of fabrication and surveillance requirements is sufficiently small that there is good likelihood that the system will remain intact during an accident (NRC Letter from John F. Stolz to Edward J. Mroczka, dated January 15, 1991, "Issuance of Exemption to 10 CFR Part 50, Appendix J, Sections III.A and III.C for the Millstone Nuclear Power Station, Unit Number 2 (TAC Number 75970)").

Due to the various acceptable options in arranging containment isolation valves, the specific valve arrangements have been given for each penetration in Figures 5.2–27 through 5.2–34. Complete descriptions of these isolation valve arrangements are summarized in Table 5.2-11 indicating each penetration by number, the valve arrangement identification, the penetration category, the testing requirements, the size and type of the valves, the mode actuation, and the valve positions in respect to normal and/or isolation conditions including the positions during air or power failure.

The piping penetrations forming an extension of the containment are designed in accordance with ANSI B-31.7, Nuclear Piping Code, Class I or II as a minimum and are installed in accordance with ASME Section III, Nuclear Power Plant Components Code. These penetrations are described in Section 5.2.6.1.2 and 5.2.6.3. The piping is subjected to a stress report as prescribed in ASME, Section III, which includes its being subjected to a seismic analysis in accordance with the State of Connecticut Building Code, Class I, as described in Section 5.2.4.3.3. The penetration is subjected to the missile and whip protection criteria described in Section 5.2.5.

Containment isolation valves are designed in accordance with the Draft ASME Code for Pumps and Valves for Nuclear Power or ASME Section III (1971 Edition). Isolation valves are subjected to a stress report or analysis which includes seismic analysis as prescribed in Section 5.2.4.3.3. Installation of these valves is subjected to the same requirements imposed on the containment penetration piping as described above.

There is sufficient redundancy in the instrumentation circuits of the engineered safety features protection system to minimize the possibility of inadvertent tripping of the isolation system. Further discussion of this redundancy and the instrumentation signals which trip the isolation system is presented in Section 7.3.

The containment pressure instrumentation is physically located close to the containment and installed using short couplings between the containment penetrations and instrumentation provided. All instrumentation provided is designed as a pressure containing device, whereby rupture of the sensing bellows will not release radioactivity to the environment but will be contained within the housing of the instrument itself. The instrument lines are sized or orificed on the inside of the containment such that the response time of the transmitters remains within an acceptable level while in the unlikely event of instrument line or transmitter housing failure, the leakage is reduced to the minimum extent practical. One shutoff valve is provided in each line for test purposes.

The instrument lines, up to and including the pressure retaining parts of instruments, are Seismic Class I, subject to quality assurance surveillance, and conservatively designed to a quality equivalent to the containment penetrations or better. All containment pressure instrumentation equipment is located in an enclosed area and protected against physical damage due to pipe whip or missiles. Provisions are included in plant design for periodic visual in-service inspection of lines from the outside of containment up to an including the pressure instruments. The single manual shutoff valve in each line will enable periodic pressurization of the impulse line from the shutoff valve up to and including the pressure transmitter for leak testing and instrument calibration.

All containment atmosphere sampling lines are designed in accordance with Regulatory Guide 1.11.

All containment penetrations except the containment sump recirculation lines, Numbers 12 and 13, and the containment pressure transmitter penetration lines, Numbers 47, 69, 70, and 71, comply with NRC General Design Criteria 55, 56, and 57 or to Regulatory Guide 1.11. Valving inside the containment is eliminated on these penetrations because of the critical nature of these penetrations.

The containment sump recirculation lines are embedded in concrete inside the containment and protected by a guard pipe outside the containment to maintain the containment boundary and system integrity. The system is required to function during the post-accident condition; therefore, if valving inside the containment existed, it would be required to be open.

The containment pressure transmitter installation is described in Table 5.2-11 and is illustrated in Figure 5.2–34 (Arrangement 30).

5.2.7.2.2 Components

The major system components and materials of fabrication for the containment isolation system are indicated in Tables 5.2-12 and 5.2-13 for piping and valves, respectively.

5.2.7.3 System Operation

5.2.7.3.1 Emergency Operation

In the unlikely event of a LOCA, containment isolation system automatically isolates the nonessential process lines coincident with the containment isolation actuation signal (CIAS) as described in Section 7.3. Containment penetrations that are opened directly to the containment, such as the normal sump drain, are also automatically isolated by the CIAS as described in

Section 7.3. Other process lines which are not opened directly to the containment, such as the steam generator bottom blowdown and demineralized water, are also automatically isolated by the CIAS.

Process lines which are essential for post-accident operation, such as the safety injection, are automatically opened by the Safety Injection Actuation Signal (SIAS). The only lines that automatically open on Containment Spray Actuation Signal are the containment spray headers. The only lines that automatically open on Sump Recirculation Actuation Signal are the containment sump recirculation lines.

5.2.7.4 Availability and Reliability

5.2.7.4.1 Special Features

All containment isolation valves are designed to ensure leak-tightness and reliability of operation. Gate, globe, ball and check valves used for containment isolation meet the leak tightness requirements of MSS-SP-61 except that the maximum seat leakage rate is less than 2.0 cc/hour per inch of seat diameter. Butterfly valves used for isolation are purchased and tested to ensure essentially zero seat leakage preinstallation under the test pressure conditions. Subsequent to installation, leak tightness is confirmed by testing as described in Section 5.2.7.1.2.b. The required valve closing times are achieved by appropriate selection of the valve operator type and size.

Containment isolation valve operators that receive the automatic CIAS, and are of the piston (air cylinder) or diaphragm operator type are selected based on the design of the operators being capable of meeting a closure time of 5 seconds. The MSIVs (2-MS-64A and 64B) closure times are described in Section 10.3.2.

The four containment air purge supply and exhaust valves (2-AC-4, -5, -6 & -7) do not receive a CIAS signal but receive a signal to close in response to a high containment radiation when the plant is in modes 5 and 6. These valves are locked closed, pneumatically isolated and electrically disconnected when the plant is in modes 1 through 4. This is accomplished by closing the instrument air isolation valves and pulling the control power fuses for each of the valves. The associated instrument air isolation valves and fuse blocks are then locked. By locking them closed in this manner, these valves are considered sealed closed isolation valves.

Motor-operated containment isolation valves have no closing stroke time requirements assumed in the accident analysis. The containment spray header and LPSI Injection Header motor-operated isolation valves have opening stroke time requirements that support the system response times assumed in the accident analysis. Refer to the Technical Requirements Manual for the listed system response times. Stroke time requirements are established for motor-operated containment isolation valves in accordance with the Inservice Test Program to ensure the valve operation is monitored for degradation.

Containment penetrations which form closed systems (as defined in GDC 57 inside containment) are not exposed to the containment environment and, therefore, do not constitute a potential leakage path.

The containment penetrations which are open to the atmosphere are listed in Section 5.3.4. For those connected piping systems which will withstand and contain the post-accident atmosphere outside containment, the closure time of the associated isolation valves is not dictated by potential containment leakage to the site boundary. Any leakage from the Containment Air Purge, Hydrogen Purge, Hydrogen Sample, and Containment Air Monitoring lines is contained within the Enclosure Building Filtration System as stated in Section 5.3.4.

The closure time for the containment purge valves is based on 5 seconds. This was demonstrated by testing by the valve manufacturer.

The position of greatest safety for air-operated valves is the position the valve will assume upon loss of air instrument supply.

Motor-operated valves on the shutdown cooling line fail “as is” in the event of loss of electrical power supply. Since these valves are closed during operation (reactor coolant system pressure greater than 300 psig), the “as is” position is the safest position.

In the event of electrical power loss to the motor-operated containment isolation valves, the valves are supplied by emergency power to achieve the greatest safety position.

The assigned locations of containment penetrations are designed to ensure adequate separation of redundant piping and valving. The penetrations leaving the containment below grade are located in the penetration rooms of the auxiliary building while the penetrations above grade are located in the enclosure building. The applicable portions of these structures are designed to protect internal equipment from potential tornadoes and missiles.

Most remotely operated containment isolation valves have provisions for remote manual operation from the main control room. Valves 2-MS-65A, 2-MS-65B, 2-MS-202, and 2-SI-651 have disconnect switches in their power circuits to ensure the valves are in their proper position in the event of an Appendix R fire, and to prevent spurious movement during specific operational modes as shown and detailed in Table 5.2-11. Position indicators (open or closed) are also provided in the control room to assure valve position during an emergency.

The isolation valves which are not required to maintain their full operational capabilities during and after a LOCA or Seismic event are designed to fail in the safe position. These valves are provided with air operators and a spring return to the safe position. The reliability of these valves is proven by seismic analysis and/or testing of the valve, operator, solenoid, and limit switches under the seismic loadings described in Section 5.8. To assure that the valves will operate under the system flow conditions, the air operator and spring are sized to operate at maximum differential pressure across the valve.

The isolation valves in vital service which must be capable of full operation during and after a LOCA are provided with motor operators or air accumulator tanks. Electrical power is supplied to the motor-operated valves from the emergency diesel generators during an onsite and/or offsite power-loss to assure that these valves are always capable of full operation. The reliability of the motor-operated valves to function during the seismic disturbance or LOCA is proven by seismic analysis and testing. To demonstrate that the motor operator has sufficient torque to operate the valves, the valves are open and/or closed under full disc differential pressure.

Manually-operated containment isolation valves are positioned using approved operating procedures, a portion of which is a Valve Check List which states the required position(s) of the valve for various plant conditions such as startup or shutdown. As appropriate, the list indicates if the valve is locked open or locked closed. The position of manually-operated containment isolation valves is verified using appropriate valve list(s) before the plant leaves the cold shutdown condition.

The isolation valves required for essential post-accident processes having air operators are provided with emergency reserve air supply tanks which are capable of actuating the valves several times. This is necessary because of the postulated failure of the station and instrument air system during an emergency. However, the twelve RBCCW CAR cooler inlet and outlet containment isolation valves are not equipped with reserve air supply tanks. The RBCCW System was designed (back fitted) with the 12 containment isolation valves as a defense in depth design feature for containment integrity to meet the requirements of GDC 57. These valves fail open (which is their accident position) on loss of air. They are located outside containment and are equipped with a hand wheel as a secondary mode of operation.

Upon receiving a manual CIAS or SIAS (either manual or automatic) actuation signal, isolation valves required to isolate the containment from the surrounding environment and other systems within the station, close automatically. Valve operators are sized to close these isolation valves before any significant amount of radioactivity can be released from the containment. In most systems, standard valve operators are sufficient.

The containment isolation valve operators have a certified proven record of reliability under operating conditions similar or more severe than those to which exposed during unit operation. These have been tested by the manufacturer to ensure the integrity in the event of inadvertent closure under operating conditions.

The worst environment to which containment isolation valves may be subjected is that described in Section 14.8.2 Containment Analysis. In addition, static, dynamic, and seismic loads, as described in Section 5.8 and exposure to physical damage are taken into account. Valves have been designed to perform their intended function under these conditions and this forms their design basis.

Damage due to severe natural environment such as freezing is not considered credible since all areas which house containment isolation valves are maintained at temperatures above freezing.

Relief valves are required to prevent over-pressuring lines for low-pressure service but subject to possible valve leakage from lines transporting fluids of potentially high temperature and pressure. Relief valves are also required to prevent over-pressuring lines due to uncontrolled thermal expansion of the process fluid.

Penetration Number 10, Figure 5.2–29, transports reactor coolant during shutdown cooling, normally at 300 psig, but it is also subject to the maximum pressure of reactor coolant system during normal plant operation. To prevent over-pressuring the piping external to the containment due to isolation valve leakage, a relief valve set at 300 psig is required. This relief valve is inside the containment and discharges to the liquid radwaste system, as shown in Figure 6.1–1.

Penetration Number 11, Figure 5.2–31, is for the Safety Injection tank testing and RCS check valve leakage bleed. The relief valve is set at 450 psig to protect the piping from over-pressure. This relief valve is also inside the containment and discharges to the Quench Tank, as shown in Figure 6.1–1.

The safety injection test line, penetration Number 11, conforms to General Design Criteria Number 57 of 10 CFR Part 50, Appendix A. This line is isolated from the reactor coolant pressure boundary by two valves, both closed, as allowed by 10 CFR 50.55a (January 1972 edition), footnote 1(b) for exclusion from the reactor coolant system. Therefore, the safety injection test line is neither open directly to the containment atmosphere nor part of the reactor coolant pressure boundary, as provided by Criterion Number 57.

Penetration Numbers 19 & 20, Figure 5.2–33, are for the main steam lines with relief valves required to protect the steam generator from over-pressure. There are 8 relief valves per main steam line for a total of 16 relief valves with nominal set points at 985 psig and the maximum pressure settings at 1,035 psig. A detailed description of these valves is given in Section 4.3, Table 4.3-3 of the FSAR. The relief valves are located outside the containment to facilitate inspection, testing, and maintenance and to protect the containment from over-pressure due to relief discharge.

Penetration Numbers 12 & 13, Figure 5.2–30, provide containment sump recirculation for long-term operation of ECCS and containment spray post accident. Piping equipped with rupture disks are connected to the body drains of containment isolation valves 2-CS-16.1A & B. The rupture disks discharge is contained within the closed piping system. The rupture disks prevent the possibility of the motor-operated valves becoming pressure locked in the closed position due to thermal expansion of trapped fluid in the valve bodies prior to initiation of sump recirculation.

5.2.7.4.2 Tests and Inspections

All isolation valves are shop tested and examined by the manufacturer in accordance with the governing code requirements to assure the integrity of the pressure retaining boundary. In addition, all valves are performance tested for seat leakage on an individual valve basis to assure reliability.

Each valve is tested after installation to ensure its leak tightness and performance. The valve operators specified have a proven record of a number of years of reliability in respect to the method of operation and material used. Throughout the plant life, these valves are tested periodically as required per 10 CFR Part 50, Appendix J and those which cannot be tested during operation (those which must remain open or closed) are tested during the scheduled shutdowns and plant outages.

Containment fluid penetration isolation valves are incorporated with provisions for periodic leak rate testing provided they are required to be tested per 10 CFR Part 50, Appendix J.

RBCCW CAR cooler inlet outlet containment isolation valves (2-RB-28.1A to D, 2-RB-28.2A to D and 2-RB-28.3A to D) are exempt from Appendix J Type C testing (NRC Letter from John F. Stolz to Edward J. Mroczka, dated January 15, 1991, "Issuance of Exemption to 10 CFR Part 50, Appendix J, Sections III.A and III.C for the Millstone Nuclear Power Station, Unit Number 2 (TAC Number 75970)"). They do not have leakage criteria. The leakage criteria for CIVs on closed systems that are exempt from Appendix J testing and whose primary safety function is to remain open or open is based on the functional leakage requirements, which is limited to "as low as reasonably attainable."

The containment isolation valves located outside the containment are accessible for maintenance and inspection during normal plant operation. The isolation valves located within containment are accessible during normal plant shutdown for maintenance and inspection.

On-line testing procedures and preoperation or acceptance testing of isolation valves are discussed with the respective process systems in Chapters 6, 9, and 10.

5.2.8 CONTAINMENT TESTING AND SURVEILLANCE

5.2.8.1 Integrated Leak-Rate Surveillance Test Program

A containment test program has been established to assure reactor containment building will adequately protect the public from core damage accidents and achieve compliance with 10 CFR 50, Appendix J, of the Code of Federal Regulations.

Containment leak tests are performed periodically per the requirements of Appendix J.

The objective of these tests is to demonstrate that leakage through the primary reactor containment and systems, and components penetrating the primary containment, do not exceed the allowable leakage rate specified in the Plant Technical Specifications (less than 0.75 L_a).

The containment penetrations are aligned in a post-LOCA configuration (i.e., plant systems penetrating the containment boundary isolated, via normal closure modes, drained of water and vented inside and outside of isolation valves) to the extent practical. A pressurization system is set up and connected to the containment through a temporary piping path. The pressurization system consists of a group of oil-free air compressors, dryer units, after-coolers, interconnecting spool pieces, and valves.

Refer to Table 5.2-14 for details of the leak-rate measurement system instrumentation.

A data acquisition system (with backup capability) is used to record ILRT containment-related test parameters, e.g., containment air pressure, temperatures, and dew point temperatures. The data acquisition system typically consists of a portable computer, a data logger, and a printer. The test data is processed via a quality related ILRT computer program.

When test prerequisites and initial conditions are satisfied, the containment is pressurized to slightly greater than accident pressure with external leak checks performed to identify any containment leakage. When test pressure is reached, containment pressurization is stopped and isolated. The containment air mass system is then allowed to thermodynamically stabilize. Once stabilization is attained, the data acquisition system records the test data and computes the ILRT leakage rate, L_{am}.

The Type A test and the supplemental verification test are performed according to the requirements of the MP2 Technical Specification and 10 CFR 50, Appendix J.

Two methods are available to calculate the containment leakage rate; mass point and total time.

The mass point method uses formulas from ANS 56.8 (Reference 5.2-8) and the total time method from BN-TOP-1 (Reference 5.2-9)

Quality-related software employing these techniques calculates the least-squares fit and upper confidence limit containment leakage rate, L_{am}. It automatically checks it against the acceptance criteria (0.75 L_a).

The software program containment model inputs are based upon quality-related engineering calculations: containment-free air volume V, Resistance Temperature Detector (RTD), and Dewcell sensor volume weight fractions (refer to Table 5.2-15 and Reference 5.2-12 for details), and the superimposed leakage rate, L_o.

A single failure RTD and Dewcell analysis calculation is completed and reweighted plan is established, per the guidance of EPRI (Reference 5.2-10).

Prior to depressurization of the containment, a verification test is completed. The verification test induces a known leakage rate, Lo, and a composite leakage calculation of L_c is made to verify that the test instrument data acquisition system was operating satisfactorily ($L_o + L_{am} - 0.25\ L_a \leq L_c \leq L_o + L_{am} + 0.25\ L_a$) and yielding accurate results.

Once this is verified, the containment is then slowly depressurized to normal atmospheric conditions and restoration is started.

5.2.8.1.1 Total Time Method for Calculating Containment Leakage Rate

The Total Time Method of the Absolute Method consists of calculating air lost from the containment using pressure, temperature, and dew point observations made during the ILRT using the Ideal Gas Law. The measured leakage rate at any time (t) is determined by calculating the percent leakage rate based on the most recent data and the data taken at the start of the test. The calculated leakage rate is then determined by plotting the measured leakage rate as a function of time and then performing a least-squares fit of the measured leakage rate values. The calculated leakage rate is expressed as a percentage of containment mass lost in a 24 hour period.

This is the primary calculation (data analysis) for use during a short duration test (i.e., test duration less than 24 hours). Bechtel Topical Report BN-TOP-1, Rev. 1, 1972, “Testing Criteria for Integrated Leakage Rate Testing of Primary Containment Structures for Nuclear Power Plants” (Reference 5.2-9) and ANSI N45.4-1972, Leakage Rate Testing of Containment Structures for Nuclear Reactors” (Reference 5.2-15), provide details on this method of determining the containment leakage rate.

5.2.8.1.2 Mass Point Method for Calculating Containment Leakage Rate

The Mass Point analysis technique consists of calculating the air mass within the containment structure over the test period using pressure, temperature, and dew point observations made during the ILRT using the Ideal Gas Law. The leakage rate is then determined by plotting the air mass as a function of time using a least-squares fit to determine the slope. The leakage rate is expressed as a percentage of air mass in a 24 hour period. This is the primary calculation (data analysis) for test durations of 24 hours or greater. This analysis technique is described in ANSI/ANS 56.8, “Containment System Leakage Testing Requirements” (Reference 5.2-8).

5.2.8.2 Structural Integrity Test

Prior to operation the containment was subjected to a pressure test equivalent to 115 percent of the postulated maximum accident pressure in accordance with Regulatory Guide 1.18. This test provides a direct verification of the structural integrity of the containment as a whole, i.e., it is equal to or better than that required to sustain the forces imposed by the accident loading condition.

The structure was pressurized to 115 percent of the postulated accident pressure (62.4 psig) during the integrated leak rate test. Radial measurements were made along six equally spaced meridians at locations near the base, at mid-height of the cylinder, and at the spring line as well as the top of the dome. Radial deflections were also taken at 12 positions around the equipment hatch.

Cracks greater than 0.005 inch were mapped near the base wall intersection, at mid-height, just below the ring girder, at the intersection of a buttress and the wall, and at the dome. At each location an area of at least 40 square feet was mapped.

Measurements and observations were taken at 0, 14, 27, 40, 54, and 62.4 psi increments while pressurizing and depressurizing. The results of these provided de facto indication that the structure is more than adequate to withstand the design internal pressures.

5.2.8.3 Post-Operational Leakage Monitoring

The procedure deviates from the Regulatory Guide 1.18 in that the tangential deflections at 12 positions around the equipment hatch were not measured since tangential deflections were not predicted. Tangential deflections measured during test on similar structure were of minimal significance.

5.2.8.4 Tendon Surveillance

5.2.8.4.1 Program Description

The primary objective of the surveillance program for the containment structure concrete and tendons during the lifetime of the plant is to ensure the strength and reliability of the post-tensioning steel and other major components such as stressing washers, shims, and bearing plates. The surveillance program is intended to provide sufficient in-service historical evidence to maintain a high level of confidence so that the integrity of the containment structure may be preserved.

This program consists of the following operations:

a. Recording lift-off pressure readings and any significant visual difference of stressing washers, shims, bearing plates, and concrete cracks.

b. Checking for possible corrosion of wires and anchorage components.

To accomplish this surveillance program, a total of twenty-one (21) tendons were provided in accordance with the Regulatory Guide 1.35, Revision 1 as follows:

a. Horizontal (hoop): Ten (10) tendons randomly selected but approximately equally distributed.

b. Vertical: Five (5) tendons are located in the wall, approximately equally spaced around the containment.

c. Dome: Six (6) tendons, two located in each 60 degrees group.

d. For the fourth surveillance (tenth year), if no significant problems were indicated by the previous surveillances, the total number of surveillance tendons could be reduced to three from each group for a total of nine surveillance tendons.

In July 1990, Regulatory Guide 1.35, Revision 3, was published. Revision 3 requires a random selection from all tendon groups. Under Revision 3, the third surveillance would have required a total of fourteen surveillance tendons to be inspected, while only nine would be required for the fourth surveillance onward assuming no problems were found in the earlier surveillances. Accordingly, from the fifth surveillance on the tendon selection, inspections, testing and sampling is being performed in accordance with Regulatory Guide 1.35, Revision 3. The sampling may be expanded for any surveillance when it is deemed desirable or necessary.

The requirements of 10 CFR 50.55a were amended by the Nuclear Regulatory Commission (NRC) to incorporate Subsections IWE and IWL of Section XI of the ASME B&PV Code, on September 9, 1996. Following this change, DENC revised the containment inservice inspection program for MPS2 to implement the requirements of ASME Section XI, Subsections IWE and IWL. RG 1.35 was determined to be redundant with ASME Section XI, Subsection IWL and was no longer needed. RG 1.35 was subsequently withdrawn by the NRC in August 2015.

5.2.8.4.2 Compliance with Regulatory Guide

The tendon surveillance program was developed and complied with the Regulatory Guide 1.35. However, in 2000 ASME Section XI, Subsections IWE and IWL were also implemented. The NRC determined RG 1.35 was redundant with ASME Section XI, Subsection IWL and subsequently withdrawn in August 2015. All inspections and testing are now performed in accordance with the requirements of ASME Section XI, IWL as amended by 10 CFR 50.55a.

5.2.9 REFERENCES

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5.2-7 10 CFR 50, Appendix J, Primary Reactor Containment Leakage Testing for Water-Cooled Power Reactors.

5.2-8 ANS 56.8, “Containment System Leakage Testing Requirements.”

5.2-9 BN-TOP-1, Revision 1, November 1, 1972. Bechtel Corporation Testing Criteria for Integrated Leak-Rate Tests of Primary Containment Structures for Nuclear Power Plants.

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5.2-11 T2621-P, MP2 Preoperational Test, April 15, 1975.

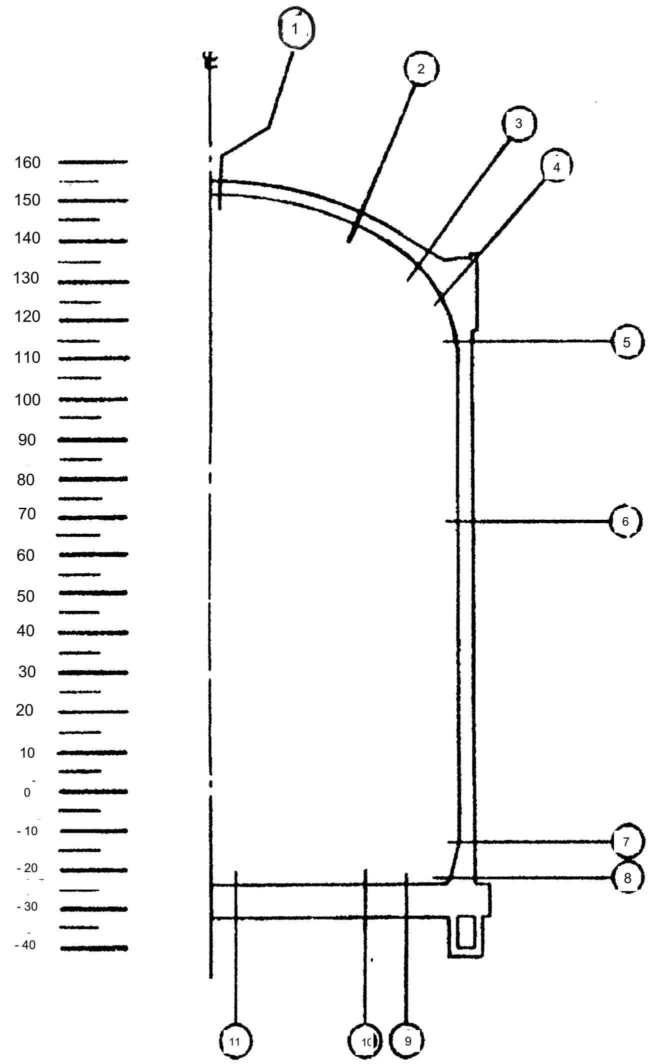
5.2-12 Engineering Calculation Number 95-ENG-1184-M2, “MP2 Containment ILRT Sensor Volume Fractions.”

5.2-13 Millstone Unit 3, Final Safety Analysis Report, Section 2.3-Meteorology.

5.2-14 Richard Jr., F. E., Hall Jr., J. R., and Woods, R. E., Vibrations of Soils and Foundations, Prentice Hall, Inc., NJ, 1970.

5.2-15 ANSI N45.4-1972, “Leakage Rate Testing of Containment Structures for Nuclear Reactors.”

TABLE 5.2-1 CONTAINMENT STRUCTURE ANALYSIS SUMMARY



KEY ELEVATION SHOWING LOCATION OF REFERENCE SECTIONS

Notes on Table Values

The values shown in the following tables are taken from the cracked section analysis for the containment building. Earthquake forces are added to the forces from other loads and the resulting total is solved to obtain the stresses given.

The allowable stresses are based on the criteria presented herein. The entire containment shell is constructed using 5000 psi concrete and Grade 60 bonded reinforcing steel. The liner plate has a guaranteed minimum yield strength of 24,000 psi.

Values in these tables correspond to Figures 5.2–19 to 5.2–26.

TABLE 5.2-2 CONTAINMENT STRUCTURE ANALYSIS SUMMARY - DEAD LOAD, INITIAL PRESTRESS AND LIVE LOAD (D+KF+L)

Portion	Section	Concrete Stress – Flexural & Membrane: Merdian (PSI)	Concrete Stress– Flexural & Membrane: Hoop (PSI)	Concrete Stress – Membrane: Merdian (PSI)	Concrete Stress – Membrane: Hoop (PSI)	Concrete Stress – Actual Shear Allowable Shear Capacity	Concrete Stress – Reinforcing Stress: Merdian (PSI)	Concrete Stress – Reinforcing Stress: Hoop (PSI)	Concrete Stress – Liner Plate Strain: Meridian %	Concrete Stress – Liner Plate Strain: Hoop %
Allowable		-3750	-3750	-1500	-1500		+30000	+30000	± 0.2%	± 0.2%
DOME	1	-977	-1497	-953	-1438	0.255	-7482	-11131	-0.036	-0.056
	2	-1142	-665	-1098	-594	0.442	-8514	-5246	-0.043	-0.020
	3	-1820	-436	-784	-204	0.557	-11052	-2705	-0.068	-0.016
RING GIRDER	4	-610	-445	-558	-351	0.198	-2302	-1821	-0.023	-0.017
WALL	5	-829	-942	-723	-788	0.709	-6322	-7079	-0.031	-0.035
	6	-1011	-1629	-898	-1374	0.035	-6581	-9612	-0.038	-0.061
	7	-973	-773	-961	-663	0.394	-7765	-5981	-0.036	-0.021
HAUNCH	8	-749	-292	-569	-278	0.364	-5680	-2313	-0.028	-0.011
BASE SLAB	9	-342	-106	-109	-63	0.379	+6781	+2106	-0.013	-0.004
	10	-108	-61	-64	-42	0.138	-807	-466	-0.004	-0.002
	11	-36	-35	-29	-28	0.005	-276	-273	-0.001	-0.001

TABLE 5.2-3 CONTAINMENT STRUCTURE ANALYSIS SUMMARY - DEAD LOAD, INITIAL PRESTRESS AND LIVE LOAD (D+F+L+1.15P)

Portion	Section	Concrete Stress – Flexural & Membrane: Merdian (PSI)	Concrete Stress– Flexural & Membrane: Hoop (PSI)	Concrete Stress – Membrane: Merdian (PSI)	Concrete Stress – Membrane: Hoop (PSI)	Concrete Stress – Actual Shear Allowable Shear Capacity	Concrete Stress – Reinforcing Stress: Merdian (PSI)	Concrete Stress – Reinforcing Stress: Hoop (PSI)	Concrete Stress – Liner Plate Strain: Meridian %	Concrete Stress – Liner Plate Strain: Hoop %
Allowable		-3750	-3750	-1500	-1500		+30000	+30000	± 0.2%	± 0.2%
DOME	1	-70	-145	-24	-38	0.298	+1134	+3855	-0.003	-0.005
	2	-139	0	-88	+56	0.074	-355	+9394	-0.005	-0.003
	3	-173	-202	-89	-101	0.914	-1101	-1277	-0.006	-0.008
RING GIRDER	4	-320	-291	-278	-294	0.088	+13653	-1957	+0.006	-0.011
	5	-559	-412	-191	-410	0.114	+8692	-3304	+0.004	-0.015
WALL	6	-318	-87	-265	-63	0.017	-1829	-374	-0.012	-0.003
	7	-1762	-981	-319	-646	0.400	+11847	-7212	+0.014	-.0012
HAUNCH	8	-1779	-580	-231	-386	0.357	+19385	-4432	+0.020	-0.007
	9	-636	-626	+53	-3	0.585	+17764	+12418	-0.024	-0.023
BASE SLAB	10	-1488	-613	+72	-18	0.154	+29512	+12153	-.0056	-0.023
	11	-64	-63	-44	-43	0.015	-486	-480	-0.002	-0.002

TABLE 5.2-4 CONTAINMENT STRUCTURE ANALYSIS SUMMARY - DEAD LOAD, INITIAL PRESTRESS AND LIVE LOAD, OPERATING TEMPERATURE AND OBE ($D+F+L+T_0+E$) [A]

Portion	Section	Concrete Stress – Flexural & Membrane: Merdian (PSI)	Concrete Stress– Flexural & Membrane: Hoop (PSI)	Concrete Stress – Membrane: Merdian (PSI)	Concrete Stress – Membrane: Hoop (PSI)	Concrete Stress – Actual Shear Allowable Shear Capacity	Concrete Stress – Reinforcing Stress: Merdian (PSI)	Concrete Stress – Reinforcing Stress: Hoop (PSI)	Concrete Stress – Liner Plate Strain: Meridian %	Concrete Stress – Liner Plate Strain: Hoop %
Allowable		-3750	-3750	-1500	-1500		+40000	+40000	±0.2%	±0.2%
DOME	1	-1400	-2124	-840	-1276	0.161	-1925	-3360	-0.063	-0.086
	2	-1745	-1270	-906	-519	0.414	-281	+3032	-0.074	-0.057
	3	-2737	-1456	-618	-302	0.485	+29937	+20938	-0.111	-0.063
RING GIRDER	4	-690	-1662	-491	-542	0.018	-1371	+16143	-0.026	-0.068
	5	-1610	-1930	-615	-708	0.523	+6967	+11510	-0.069	-0.081
WALL	6	-1816	-2557	-770	-1194	0.028	+7085	+13832	-0.077	-0.104
	7	(-600)	-1134	-1015	-546	(0.886)	(+4492)	+554	-0.136	-0.052
		-3428				0.990	+20208			
HAUNCH	8	(-1051)	-845	-601	-356	0.595	(+14090)	+4192	-0.104	-0.037
		-2601					+39554			
	9	(-1142)	(-274)	-294	+7	0.615	(+10720)	(+7443)	(-0.051)	(-0.018)
		-1098	-261				+10308	+7089	-0.049	-0.017
BASE SLAB	10	(-1064)	(-628)	-134	-5	0.350	(+12777)	(+13656)	(-0.050)	(-0.032)
		-858	-616				+10304	+13388	-0.040	-0.031
	11	(-968)	(-855)	-308	-291	0.222	(+5539)	(+3911)	(-0.045)	(-0.040)
		-978	-864				+5595	+3950	-0.045	-0.040

a. Note: Values in parentheses reflect current controlling stresses/strains resulting from the revised seismic analysis performed in 1998-99.

TABLE 5.2-5 CONTAINMENT STRUCTURE ANALYSIS SUMMARY - DEAD LOAD, PRESTRESS, LIVE LOAD, 100% ACCIDENT PRESSURE AND ACCIDENT TEMPERATURE ($D+F+L+1.0P+T_1$)

Portion	Section	Concrete Stress					Reinforcing Stress		Liner Plate Strain	
		Flexural & Membrane		Membrane		Actual Shear Allowable Shear Capacity				
		MER. (PSI)	HOOP (PSI)	MER. (PSI)	HOOP (PSI)		MER. (PSI)	HOOP (PSI)	MER. %	HOOP %
Allowable		-3750	-3750	-1500	-1500		+40000	+40000	± 0.5%	±0.5%
DOME	1	-238	-1312	-125	-204	0.400	+28479	+33263	-0.0104	-0.141
	2	-419	0	-139	-32	0.321	+28276	+28784	-0.110	-0.086
	3	-1089	-2238	-103	-284	0.543	+23322	+20642	-0.136	-0.177
RING GIRDER	4	-1283	-2956	-305	-515	0.038	-3608	+8183	-0.155	-0.201
	5	-2122	-2716	-223	-526	0.193	+27302	+5991	-0.079	-0.196
WALL	6	-2115	-2053	-304	-206	0.036	+14203	+14712	-0.173	-0.171
	7	-1854	-2420	-385	-630	0.199	+5215	+1545	-0.163	-0.185
HAUNCH	8	-305	-2271	-264	-574	0.313	-2402	+840	-0.008	-0.135
	9	-764	-853	+51	-75	0.432	+22444	+13189	-0.027	-.039
BASE SLAB	10	-1164	-847	+89	-110	0.119	+34050	+11248	-0.036	-0.039
	11	-708	-714	-265	-266	0.051	+2269	+2272	-0.034	-0.035

TABLE 5.2-6 CONTAINMENT STRUCTURE ANALYSIS SUMMARY - DEAD LOAD, PRESTRESS, OPERATING TEMPERATURE, THERMAL EXPANSION FORCES OF PIPES, PIPE RUPTURE FORCES AND DBE ($D+F+T_0+H+R+E^1)^A$

Portion	Section	Concrete Stress					Reinforcing Stress		Liner Plate Strain	
		Flexural & Membrane		Membrane		Actual Shear Allowable Shear Capacity				
		MER. (PSI)	HOOP (PSI)	MER. (PSI)	HOOP (PSI)		MER. (PSI)	HOOP (PSI)	MER. %	HOOP %
ALLOWABLE		-4250	-4250	-4250	-4250		+54000	+54000	± 0.5%	± 0.5%
DOME	1	-1400	-2124	-840	-1276	0.127	-1925	-3360	-0.063	-0.086
	2	-1745	-1270	-906	-519	0.371	-281	+3032	-0.074	-0.057
	3	-2737	-1456	-618	-302	0.430	+29937	+20938	-0.111	-0.063
RING GIRDER	4	-690	-1662	-491	-542	0.014	-1371	+16143	-0.026	-0.068
	5	-1610	-1930	-615	-708	0.442	+6967	+11510	-0.069	-0.081
WALL	6	-1816	-2557	-770	-1194	0.023	+7085	+13832	-0.077	-0.104
	7	(-949)	-1134	-1091	-546	(0.720)	(+7340)	+554	-0.155	-0.052
		-3935				0.941	+26160			
HAUNCH	8	(-1601)	-845	-641	-356	0.726	(+22520)	+4192	-0.124	-0.037
		-3142					+50111			
	9	(-1330)	(-77)	-314	+65	0.562	(+13639)	(+11574)	(-0.058)	(-0.000)
		-1265	-72				+12887	+10817	-0.055	-0.000

TABLE 5.2-6 CONTAINMENT STRUCTURE ANALYSIS SUMMARY - DEAD LOAD, PRESTRESS, OPERATING TEMPERATURE, THERMAL EXPANSION FORCES OF PIPES, PIPE RUPTURE FORCES AND DBE ($D+F+T_0+H+R+E^1$)[A] (CONTINUED)

Portion	Section	Concrete Stress					Reinforcing Stress		Liner Plate Strain	
		Flexural & Membrane		Membrane		Actual Shear Allowable Shear Capacity				
		MER. (PSI)	HOOP (PSI)	MER. (PSI)	HOOP (PSI)		MER. (PSI)	HOOP (PSI)	MER. %	HOOP %
BASE SLAB	10	(-1230)	(-710)	-118	-3	0.334	(+16530)	(+15531)	(-0.056	(-0.035)
		-939	-657				+12618	+14381	-0.043	-0.032
	11	-1043	(-873)	-306	-283	0.224	+6958	(+4664)	-0.047	(0.040)
			-891					+4759		-0.041

a. Note: Values in parentheses reflect current controlling stresses/strains resulting from the revised seismic analysis performed in 1998-99.

TABLE 5.2-6A DELETED BY FSARCR 04-MP2-016

TABLE 5.2-7 CONTAINMENT STRUCTURE ANALYSIS SUMMARY - DEAD LOAD, PRESTRESS, 100% ACCIDENT PRESSURE, THERMAL EXPANSION FORCES OF PIPES, PIPE RUPTURE FORCES AND DBE ($D+F+1.0P+H+T_1+E^1$) [A]

Portion	Section	Concrete Stress					Reinforcing Stress		Liner Plate Strain	
		Flexural & Membrane		Membrane		Actual Shear Allowable Shear Capacity				
		MER. (PSI)	HOOP (PSI)	MER. (PSI)	HOOP (PSI)		MER. (PSI)	HOOP (PSI)	MER. %	HOOP %
ALLOWABLE		-4250	-4250	-4250	-4250		+54000	+54000	± 0.5%	± 0.5%
DOME	1	-238	-1312	-125	-204		+28479	+33263	-0.104	-0.141
	2	-419	0	-139	-32	0.271	+28276	+28784	-0.110	-0.086
	3	-1089	-2238	-103	-284	0.481	+23322	+20642	-0.136	-0.177
RING GIRDER	4	-1283	-2956	-305	-515	0.035	-3608	+8183	-0.155	-0.201
	5	-2122	-2716	-223	-526	0.171	+27302	+5991	-0.079	-0.196
WALL	6	-2115	-2053	-304	-206	0.032	+14203	+14712	-0.173	-0.171
	7	-691	-2420	-622	-630	0.921	+16649	+1545	-0.231	-0.185
HAUNCH	8	-1972	-2271	-405	-574	0.837	+17453	+840	-0.120	-0.135
	9	(-1638)	(-511)	-52	+234	0.816	(+30071)	(+33602)	(-0.070)	(+0.047)
		-1575	-487				+28914	+32002	-0.067	+0.045
BASE SLAB	10	(-3825)	(-803)	+172	+137	0.328	(+52609)	(+32029)	(-0.010)	(-0.004)
		-1591	-750				+51617	+29934	-0.009	-0.004
	11	-948	(-784)	-253	-232	0.237	+6921	(+4659)	-0.044	(-0.037)
			-800					+4754		-0.038

a. Note: Values in parentheses reflect current controlling stresses/strains resulting from the revised seismic analysis performed in 1998-99.

TABLE 5.2-8 CONTAINMENT STRUCTURE ANALYSIS SUMMARY - DEAD LOAD, PRESTRESS, 125% ACCIDENT PRESSURE, 125% THERMAL EXPANSION FORCES OF PIPES, ACCIDENT TEMPERATURE AND 125% DBE ($D+F+1.25P+1.25H+T_1+1.25E$)[A]

Portion	Section	Concrete Stress					Reinforcing Stress		Liner Plate Strain	
		Flexural & Membrane		Membrane		Actual Shear Allowable Shear Capacity				
		MER. (PSI)	HOOP (PSI)	MER. (PSI)	HOOP (PSI)		MER. (PSI)	HOOP (PSI)	MER. %	HOOP %
ALLOWABLE		-4250	-4250	-4250	-4250		+54000	+54000	± 0.5%	± 0.5%
DOME	1	-233	-374	+51	+60	0.359	+34254	+44242	-0.079	0.097
	2	-91	0	+50	+92	0.252	+44348	+38368	-0.033	-0.041
	3	-325	-2125	+26	-269	0.456	+22511	+19599	-0.107	-0.173
RING GIRDER	4	-211	-2903	-260	-514	0.035	-2478	+5846	-0.008	-0.199
	5	-699	-2559	-125	-468	0.070	+14752	+7094	-0.117	-0.190
WALL	6	-1247	-14	-191	+41	0.032	+20789	+39086	-0.141	-0.040
	7	-2908	-2361	-454	-606	0.947	+11400	+1640	-0.202	-0.183
HAUNCH	8	-2292	-2264	-93	-525	0.859	+40726	+312	0.000	-0.135
	9	(-1580)	(-592)	+9	+155	0.870	(+33430)	(+28651)	(-0.068)	(+0.014)
		-1463	-564				+30954	+27287	-0.063	+0.013
BASE SLAB	10	(-1767)	(-806)	+67	+42	0.277	(+46578)	(+23033)	(-0.064)	(-0.039)
		-1485	-753				+39141	+21526	-0.054	-0.036
	11	(-961)	(-829)	-279	-263	0.177	(+5903)	(+4143)	(-0.044)	(-.039)
		-933	-821				+5731	+4102	-0.043	0.039

a. Note: Values in parentheses reflect current controlling stresses/strains resulting from the revised seismic analysis performed in 1998-99.

TABLE 5.2-9 CONTAINMENT STRUCTURE ANALYSIS SUMMARY - DEAD LOAD, PRESTRESS, 150% ACCIDENT PRESSURE, AND ACCIDENT TEMPERATURE (D+F+1.5P+T_1)

		Concrete Stress					Reinforcing Stress		Liner Plate Strain	
		Flexural & Membrane		Membrane		Actual Shear Allowable Shear Capacity				
Portion	Section	MER. (PSI)	HOOP (PSI)	MER. (PSI)	HOOP (PSI)		MER. PSI)	HOOP (PSI)	MER. %	HOOP %
ALLOWABLE		-4250	-4250	-4250	-4250		+54000	+54000	± 0.5%	± 0.5%
DOME	1	-249	-366	+226	+324	0.359	+45769	+54195	-0.029	-0.020
	2	-114	0	+239	+216	0.239	+43842	+34850	-0.039	-0.058
	3	0	-1995	+155	-252	0.456	+30200	+18706	-0.018	-0.168
RING GIRDER	4	-356	-2856	-215	-512	0.028	-2855	+3587	-0.003	-0.197
	5	-47	-2452	-30	-409	0.019	+26249	+8102	-0.092	-0.186
WALL	6	-166	0	-78	+287	0.032	+28588	+33155	-0.100	-0.065
	7	-895	-2298	-157	-581	0.424	+9106	+1776	-0.127	-0.180
HAUNCH	8	-607	-2255	-145	-549	0.503	+9998	-224	0.000	-0.124
	9	-1121	-1094	+135	-115	0.629	+37848	+15961	-0.001	-0.048
BASE SLAB	10	-1597	-1004	-88	-208	0.284	+27145	+9735	-0.068	-0.045
	11	-795	-797	-311	-313	0.023	+1981	+2002	-0.038	-0.038

TABLE 5.2-10 SPECTRUM OF POTENTIAL MISSILES FROM INSIDE THE CONTAINMENT

Table 1: I. Reactor Vessel

ITEM	KINETIC ENERGY (ft-lb.)	Weight (lb.)	LEADING SECTION	POINT OF IMPACT
A. Closure Head Nut	2,022	116	Annular OD = 10-9/16 inch ID = 6.8 inch	Overhead missile shield
B. Closure Head Nut & Stud	4,932	710	Solid Circle 7 inch Diameter	Overhead missile shield
C. Instrumentation Assembly	12,700	335	Solid Disk 6.5 inch Diameter and 3 inch Thick	Overhead missile shield
D. Instrumentation from Flange Up	14,000	165	Solid Disk 6.5 inch Diameter and 3 inch Thick	Overhead missile shield
E. Instrument Flange Stud	14.3	6.5	Solid Circle 1.5 inch Diameter	Overhead missile shield

Table 2: II. Steam Generator

ITEM	KINETIC ENERGY (ft-lb.)	Weight (lb.)	LEADING SECTION	POINT OF IMPACT
A. Steam Generator Primary Manway Stud and Nut	48.53	9	Solid Circle 1.338 inch Diameter	Containment Floor or Secondary Shield Wall
B. Steam Generator Secondary Manway Stud and Nut	10.0	5	Solid Circle 1.25 inch Diameter	Steam Generator Blockhouse Wall
C. Steam Generator Secondary Handhole Stud and Nut	3.98	1.7	Solid Circle 0.82 inch Diameter	Primary or Secondary Shield Wall

TABLE 5.2-10 SPECTRUM OF POTENTIAL MISSILES FROM INSIDE THE CONTAINMENT (CONTINUED)

Table 3: III. Pressurizer

Item	Kinetic Energy (ft-lb.)	Weight (lb.)	Leading Section	Point of Impact
A. Pressurizer Manway Stud & Nut [NOTE: This missile envelops the Pressurizer vent port stud missile, which would strike the south wall.]	57.16	6.8	Solid Circle 1.0 inch Overhead missile shield Diameter	Upper west wall of Pressurizer Blockhouse
B. Pressurizer Temperature Element	290	3	Solid Disk 2.75 inch Overhead missile shield Diameter and 0.5 inch Overhead missile shield Thick	Pressurizer Blockhouse wall, roof slab or platform
C. Pressurizer Instrument Nozzle and Instrument (Based on V & VII Nozzle & RTD cases.)	1,125	11.1	Solid Disk 2.75 inch Overhead missile shield Diameter and 0.5 inch Overhead missile shield Thick	Pressurizer Blockhouse wall, roof slab or platform
D. Safety Valve Flange Bolt [NOTE: This missile envelops stud missiles from either flange or from the bonnet bolted connection.]	15	3.7	Solid Circle 1.25 inch Overhead missile shield Diameter	Pressurizer Blockhouse Wall or Floor

Table 4: IV.

Item	Kinetic Energy (ft-lb.)	Weight (lb.)	Leading Section	Point of Impact
Control Element Drive Mechanism (Magnetic Jack	47,800	2,100	Solid Circle 11 inch Overhead missile shield Diameter	Overhead missile shield

TABLE 5.2-10 SPECTRUM OF POTENTIAL MISSILES FROM INSIDE THE CONTAINMENT (CONTINUED)

Table 5: V.

Item	Kinetic Energy (ft-lb.)	Weight (lb.)	Leading Section	Point of Impact
Main Coolant Piping Temperature Nozzle with RTD	1,125	11.1	Solid Disk 2.75 inch Overhead missile shield Diameter and 0.5 inch Overhead missile shield Thick	Secondary shield wall

Table 6: VI.

Item	Kinetic Energy (ft-lb.)	Weight (lb.)	Leading Section	Point of Impact
Surge and Spray Piping Thermal Wells with RTD Assembly	277	1/34	Solid Disk 2.75 inch Overhead missile shield Diameter and 0.5 inch Thick	Secondary shield wall

Table 7: VII.

Item	Kinetic Energy (ft-lb.)	Weight (lb.)	Leading Section	Point of Impact
Main Coolant Pump Thermal Well with RTD	1,125	11.1	Solid Disk 2.75 inch Overhead missile shield Diameter and 0.5 inch Overhead missile shield Thick	Secondary shield wall

The basic formulas used for the calculation of missile penetration are those presented in NavDocks P-51, "Design of Protective Structures - a New Concept of Structural Behavior," published by the U.S. Bureau of Yards and Docks, August, 1950, Washington, D.C.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION

Penetration Number	Service	System	Penetration Type [1]	Penetration Category [2]	Flow Direction	Valve Arrangement	Type C Testing Requirements [3]	Valve Identification	Location Reference to Containment Structure	Valve Size	Valve Type	Valve Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
1	Demineralized Water	PMW	IA	N [12]	IN	1B	Yes	2-PMW-43	Outside	2 inch	Globe	1	2 inch	Diaphragm	CIAS	Closed	Closed	Yes	Closed
							No	2-PMW-165	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							Yes	2-PMW-3	Inside	2 inch	Check	1		–	–	–	–	No	–
2	Letdown Line to Purification Demineralizer	CVCS	IA	P	OUT	7	Yes [4]	2-CH-516	Inside	3 inch	Globe	1	2 inch	Diaphragm	CIAS	Open	Closed	Yes	Closed
							No	2-CH-006 [11]	Inside	2 inch	Gate	1		Manual	----	Open	As Is	No	Open
							Yes	2-CH-089	Outside	2 inch	Globe	1		Diaphragm	CIAS	Open	Closed	Yes	Closed
							No	2-CH-763, 2-CH-658, 2-CH-991	Inside	1 inch	Gate	3		Manual	----	Locked Closed	As Is	No	Closed
							No	2-CH-260, 2-CH-082, 2-CH-083	Inside	0.75 inch	Globe	3		Manual	----	Locked Closed	As Is	No	Closed
							No	2-CH-067	Outside	0.75 inch	Gate	1		Manual	----	Locked Closed	As Is	No	Closed
							Yes [4]	2-CH-515	Inside	3 inch	Globe	1		Diaphragm	SIAS	Open	Closed	Yes	Closed
3	Reactor Coolant Charging Line	CVCS	IA	P	IN	9	Yes [4]	2-CH-518, 2-CH-519	Inside	2 inch	Globe	2	2 inch	Diaphragm	Remote	Open	Open	Yes	Open
							Yes [4]	2-CH-517	Inside	2 inch	Globe	1		Diaphragm	Remote	Closed	Closed	Yes	Closed
							Yes [4]	2-CH-434	Inside	2 inch	Gate	1		Manual	----	Locked Closed	As Is	No	Closed
							Yes	2-CH-429	Outside	2 inch	Gate	1		MOV	Remote	Open	As Is	Yes	Open
							No	2-CH-004, 2-CH-003	Inside	0.75 inch	Gate	2		Manual	----	Locked Closed	As Is	No	Closed
							No	2-CH-001, 2-CH-002, 2-CH- 443, 2-CH-714	Inside	0.75 inch	Globe	4		Manual	----	Locked Closed	As Is	No	Closed
							No	2-CH-710	Outside	1 inch	Gate	1		Manual	----	Locked Closed	As Is	No	Closed
							Yes [4]	2-RC-71	Inside	0.75 inch	Globe	1		Manual	----	Locked Closed	As Is	No	Closed
							No	2-CH-661	Inside	1 inch	Gate	1		Manual	----	Locked Closed	As Is	No	Closed
							No	2-CH-435 [11]	Inside	2 inch	Spring Check	1 [5]		–	–	–	–	No	–

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

Penetration Number	Service	System	Penetration Type[1]	Penetration Category[2]	Flow Direction	Valve Arrangement	Type C Testing Requirements[3]	Valve Identification	Location Reference to Containment Structure	Valve Size	Valve Type	Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
							Yes	2-CH-986	Outside	3/4 inch x 1inch	Relief	1		–	–	–	–	No	–
4	Containment Spray Water	CSS	IA	O	IN	17A	Yes	2-CS-5A	Inside	8 inch	Check	1	8 inch	–	–	–	----	No	–
							Yes	2-CS-4.1A	Outside	8 inch	Gate	1		MOV	CSAS	Closed	As Is	Yes	Open
							No	2-CS-049C	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-CS-049A	Outside	1 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
5	Containment Spray Water	CSS	IA	O	IN	17B	Yes	2-CS-5B	Inside	8 inch	Check	1	8 inch	–	–	–	–	No	–
							Yes	2-CS-4.1B	Outside	8 inch	Gate	1		MOV	CSAS	Closed	As Is	Yes	Open
							No	2-CS-101	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
6,8	Safety Injection Low & High Pressure	SIS	IB	P	IN	10A (Penetration 6)	No	Penetration 6: 2-SI-645; Penetration 8: 2-SI-635	Outside	6 inch	Globe	1	6 inch	MOV	SIAS	Closed	As Is	Yes	Open
								2-SI-160 2-SI-161	Outside	3 inch	Gate	2	6 inch	Manual	-	Locked Closed	As Is	No	Locked Closed
								2-SI-163	Outside	0.75 inch	Globe	1	6 inch	Manual	-	Locked Closed	As Is	No	Locked Closed
						10B (Penetration 8)	No	2-SI-646, 2-SI-647; 2-SI-636, 2-SI-637	Outside	2 inch	Globe	2		MOV	SIAS	Throttled	As Is	Yes	Open
							No	2-SI-733, 2-SI-041E	Outside	(Penetration 8 only) 1inch	Globe	2		Manual	–	Locked Closed	As Is	No	Closed
							No	2-SI-144 [11]; 2-SI-134 [11]	Outside	6 inch	Check	1		–	–	–	–	No	–
							No	2-SI-143 [11], 2-SI-1009 [11]; 2-SI-133 [11], 2-SI-1010	Outside	2 inch	Check	2		–	–	–	–	No	–
							No	2-SI-095, 2-SI-1734	Outside	(Penetration 6 only) 1 inch	Globe	2		Manual	–	Locked Closed	As Is	No	Closed
							No	2-SI-041F, 2-SI-1735, 2-SI-1742D; 2-SI-041D, 2-SI-110, 2-SI-742C	Outside	0.75 inch	Globe	3		Manual	–	Locked Closed	As Is	No	Closed

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

Penetration Number	Service	System	Penetration Type[1]	Penetration Category[2]	Flow Direction	Valve Arrangement	Type C Testing Requirements[3]	Valve Identification	Location Reference to Containment Structure	Valve Size	Valve Type	Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
							No	2-SI-145[11], 2-SI-135[11], 2-SI-146[11] 2-SI-136[11]	Outside	0.75 inch	Globe	2		Manual	–	Open	As Is	No	Open
							No	2-SI-706D 2-SI-706C	Inside	6 inch	Check	1		–	–	–	–	No	–
							No	2-SI-848 S-SI-846	Outside	0.75 inch	Globe			Manual	–	Locked Closed	–	No	Closed
7	Safety Injection Low & High Pressure	SIS	IB	P	IN	10C	No	2-SI-615	Outside	6 inch	Globe	1	6 inch	MOV	SIAS	Closed	As Is	Yes	Open
							No	2-SI-616, 2-SI-617	Outside	2 inch	Globe	2		MOV	SIAS	Throttled	As Is	Yes	Open
							No	2-SI-114[11]	Outside	6 inch	Check	1		–	–	–	–	No	–
							No	2-SI-012[11], 2-SI-113[11]	Outside	2 inch	Check	2		–	–	–	–	No	–
							No	2-SI-041A, 2-SI-107, 2-SI-716, 2-SI-715, 2-SI-742A	Outside	0.75 inch	Globe	5		Manual	–	Locked Closed	As Is	No	Closed
							No	2-SI-717, 2-SI-718	Outside	1 inch	Gate	2		Manual	–	Locked Closed	As Is	No	Closed
							No	2-SI-115[11], 2-SI-116[11]	Outside	0.75 inch	Globe	2		Manual	–	Open	As Is	No	Open
							No	2-SI-706A	Inside	6 inch	Check	1		–	–	–	–	No	–
9	Safety Injection Low & High Pressure	SIS	IB	P	IN	10D	No	2-SI-625	Outside	6 inch	Globe	1	6 inch	MOV	SIAS	Closed	As Is	Yes	Open
							No	2-SI-626, 2-SI-627	Outside	2 inch	Globe	2		MOV	SIAS	Throttled	As Is	Yes	Open
							No	2-SI-124[11]	Outside	6 inch	Check	1		–	–	–	–	No	–
							No	2-SI-123[11], 2-SI-011[11]	Outside	2 inch	Check	2		–	–	–	–	No	–
							No	2-SI-722, 2-SI-723, 2-SI-720, 2-SI-721, 2-SI-742B	Outside	0.75 inch	Globe	5		Manual	----	Locked Closed	As Is	No	Closed
							No	2-SI-125[11], 2-SI-126[11]	Outside	0.75 inch	Globe	2		Manual	–	Open	As Is	No	Open
							No	2-SI-706B	Inside	6 inch	Check	1			–	–	–	No	–
10	Reactor Coolant Shutdown Cooling	SIS	IB	P	OUT	11	No	2-SI-709	Outside	12 inch	Gate	1	12 inch	Manual	–	Locked Closed	As Is	No	Closed

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

										Valve									
Penetration Number	Service	System	Penetration Type [1]	Penetration Category[2]	Flow Direction	Valve Arrangement	Type C Testing Requirements[3]	Valve Identification	Location Reference to Containment Structure	Size	Type	Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
							No	2-SI-651	Inside	12 inch	Gate	1		MOV	Remote[7]	Closed	As Is	Yes	Closed
							No	2-SI-101A	Outside	1 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-SI-102A	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-SI-043A	Inside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
11	Safety Injection Tank Test Line	SIS	IA	N	OUT	20	Yes	2-SI-463	Outside	2 inch	Gate	1	2 inch	Manual	–	Locked Closed	As Is	No	Closed
12 & 13	Containment Sump Recirculation Line	SIS	Special	Special	OUT	16	No	Penetration 12: 2-CS-16.1A; Penetration 13: 2-CS-16.1B	Outside	24 inch	Gate	1	24 inch	MOV	SRAS	Closed	As Is	Yes	Open
							No	2-CS-127 [11] 2-CS-125 [11]	Outside	0.75 inch	Gate	1		Manual	–	Locked Open	As Is	No	Open
							No	2-CS-130 2-CS-135	Outside	0.75 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-CS-131 [11] 2-CS-136 [11]	Outside	0.75 inch	Gate	1		Manual	–	Locked Open	As Is	No	Open
							No	2-CS-132 2-CS-137	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-CS-133 2-CS-138	Outside	0.75 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-CS-134 2-CS-139	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-CS-140 [11] 2-CS-141 [11]	Outside	0.75 inch	Check	1		–	–	Closed	As Is	No	Closed
14	Containment Sump to Aerated Drain Tank	RWS	IA	O	OUT	13A	Yes	2-SSP-16.2	Outside	3 inch	Globe	1	3 inch	Diaphragm	CIAS	Closed	Closed	Yes	Closed
							Yes	2-SSP-16.1	Inside	3 inch	Globe	1		Diaphragm	CIAS	Closed	Closed	Yes	Closed
							No	2-SSP-51	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-SSP-73	Outside	1 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

Penetration Number	Service	System	Penetration Type [1]	Penetration Category[2]	Flow Direction	Valve Arrangement	Type C Testing Requirements[3]	Valve Identification		Location Reference to Containment Structure	Valve Size	Valve Type	Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
15 & 16	Feedwater & Auxiliary Feedwater	FW	II	N	IN	15A	No	Penetration 15 2-FW-5A	Penetration 16 2-FW-5B	Outside	18 inch	Positive Acting Check Valve	1	18 inch	Backflow	–	Open	Closed	Yes	Closed
						for Penetration 15	No	2-FW-12A	2-FW-12B	Outside	6 inch	Positive Acting Check Valve	1		Backflow	–	Closed	Closed	Yes	Closed
						15B for Penetration 16	No	2-FW-16A [11]	2-FW-16B [11]	Outside	1 inch	Check	1		–	–	–	–	No	–
							No	2-FW-15A	2-FW-15B	Outside	1 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-FW-86	2-FW-182	Outside	1 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-FW-261A		Outside	(Pene. 15 only)	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
19 & 20	Main Steam	MSS	III	N	OUT	23	No	Penetration 19 2-MS-64A	Penetration 20 2-MS-64B	Outside	0.75 inch	Stop check	1	34 inch	Air Cylinder	MSI	Open	Closed	Yes	Closed [6]
							No	2-MS-371	2-MS-369	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-MS-201	2-MS-202	Outside	4 inch	Gate	1		MOV	Remote [7]	Open	As Is	Yes	Open
							No	2-MS-3A [11]	2-MS-3B [11]	Outside	12 inch	Gate	1		Manual	–	Open	As Is	No	Open
							No	2-MS-190A	2-MS-190B	Outside	8 inch	Globe	1		Diaphragm	Steam Generator Pressure	Closed	Closed	Yes	Closed
							No	2-MS-265B	2-MS-266B	Outside	1 inch	Globe	1		Diaphragm	MSI	Open	Closed	Yes	Closed
							No	2-MS-247, 2-MS-248, 2-MS-249, 2-MS-250, 2-MS-251, 2-MS-252, 2-MS-253, 2-MS-254	2-MS-239, 2-MS-240, 2-MS-241, 2-MS-242, 2-MS-243, 2-MS-244, 2-MS-245, 2-MS-246	Outside	6 inch	Relief	8		–	–	–	–	No	–
							No	2-MS-65A	2-MS-65B	Outside	3 inch	Globe	1 ***		MOV	MSI [7]	Closed	As Is	Yes	Closed
							No	2-MS-297	2-MS-296	Outside	1 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

Penetration Number	Service	System	Penetration Type [1]	Penetration Category [2]	Flow Direction	Valve Arrangement	Type C Testing Requirements [3]	Valve Identification	Location Reference to Containment Structure	Valve Size	Valve Type	Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
							No	2-MS-265A [11] 2-MS-266C [11]	Outside	1 inch	Gate	1		Manual	–	Open	As Is	No	Open
							No	2-MS-255	Outside	(Penetration 19 only) 0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-MS-258	Outside	(Penetration 20 only) 1 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-MS-41A [11] 2-MS-41B [11]	Outside	0.75 inch	Globe	1		Manual	–	Open	As Is	No	Open
							No	2-MS-461 [11] 2-MS-462 [11]	Outside	0.75 inch	Globe	1		Manual	–	Closed	As Is	No	Closed
							No	2-MS-459 2-MS-458	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Locked Closed
21	Reactor Coolant & Pressurizer Sampling	SS	IA	P	OUT	19	No	2-LRR-265 [11]	Inside	0.5 inch	Check	1	0.5 inch	–	–	–	–	No	–
							Yes	2-LRR-61.1	Inside	0.5 inch	Globe	1		Diaphragm	CIAS	Closed	Closed	Yes	Closed
							Yes	2-RC-45	Outside	0.75 inch	Globe	1		Diaphragm	CIAS	Closed	Closed	Yes	Closed
							Yes	2-RC-001, 2-RC-002	Inside	0.75 inch	Globe	2		Diaphragm	CIAS	Closed	Closed	Yes	Closed
							Yes	2-RC-003	Inside	0.75 inch	Globe	1		Diaphragm	CIAS	Closed	Closed	Yes	Closed
							No	2-RC-65 [11]	Inside	3/8 inch	Globe	1		Manual	–	Open	As Is	No	Open
							No	2-RC-434, 2-RC-435	Inside	3/8 inch	Globe	2		Manual	–	Locked Closed	As Is	No	Closed
22 & 23	Steam Generator Bottom Blowdown	SGBS	IA	N	OUT	14	No	Penetration 22: 2-MS-220A; Penetration 23: 2-MS-220B	Outside	2 inch	Globe	1	2 inch	Diaphragm	AFAIS, CIAS & High Containment Radiation High Radiation	Open	Closed	Yes	Closed

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

Penetration Number	Service	System	Penetration Type [1]	Penetration Category[2]	Flow Direction	Valve Arrangement	Type C Testing Requirements[3]	Valve Identification	Location Reference to Containment Structure	Valve Size	Valve Type	Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
24	Reactor Bldg. Closed Cooling Water Inlet to Reactor Coolant Pumps and Other Components [8]	RBCCW	IA	N	IN	24	Yes	2-RB-30.1A	Outside	8 inch	Gate	1	8 inch	MOV	Remote	Open	As Is	Yes	Open
							No	2-RB-289	Outside	1 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-RB-173A [11]	Outside	0.75 inch	Globe	1		Manual	–	Open	As Is	No	Open
25 & 26	Reactor Building Closed Cooling Water to Containment Air Recirculation Units	RBCCW	IA	N	IN	21A	No	Penetration 25: 2-RB-28.1D; Penetration 26: 2-RB-28.1B	Outside	10 inch	Butterfly	1	10 inch	Air Cylinder.	Remote	Open	Open	Yes	Open
							No	2-RB-282 2-RB-283	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-RB-345	Outside	(Penetration 26 only) 1 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
27 & 28	Reactor Bldg. Closed Cooling Water to Containment Air Recirculation Units	RBCCW	IA	N	IN	21B	No	Penetration 27: 2-RB-28.1A; Penetration 28: 2-RB-28.1C	Outside	10 inch	Butterfly	1	10 inch	Air Cylinder	Remote	Open	Open	Yes	Open
							No	2-RB-236 2-RB-237	Outside	1 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

Penetration Number	Service	System	Penetration Type [1]	Penetration Category [2]	Flow Direction	Valve Arrangement	Type C Testing Requirements [3]	Valve Identification	Location Reference to Containment Structure	Valve Size	Valve Type	Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
29	Reactor Building Closed Cooling Water Outlet from Reactor Coolant Pumps and Other Components [8]	RBCCW	IA	N	OUT	2	Yes	2-RB-37.2A	Outside	8 inch	Gate	1	8 inch	MOV	Remote	Open	As Is	Yes	Open
							No	2-RB-297A	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-RB-298	Outside	1 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
30, 31, 32 & 33	Reactor Bldg. Closed Cooling Water From Containment Air Recirculation Cooling	RBCCW	IA	N	OUT	22	No	Penetration 30 2-RB-28.3D Penetration 32 2-RB-28.3A	Outside	10 inch	Butterfly	1	10 inch	Air Cylinder	SIAS	Closed	Open	Yes	Open
								Penetration 31 2-RB-28.3B Penetration 33 2-RB-28.3C											
							No	Penetration30 2RB-28.2D Penetration 32 2RB-28.2A	Outside	6 inch	Butterfly	1	6 inch	Air Cylinder	Remote	Open	Open	Yes	Open
								Penetration 31 |2RB-28.2B Penetration 33 2RB-28.2C											
34	Nitrogen Supply	NS	IA	N [12]	IN	18	Yes	2-SI-312	Outside	0.75 inch	Globe	1	1 inch	Diaphragm	CIAS	Open	Closed	Yes	Closed
							No	2-SI-045	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
35	Drain from Primary Tank	RWS	IA	O	OUT	13B	Yes	2-LRR-43.2	Outside	3 inch	Globe	1	4 inch	Diaphragm	CIAS	Closed	Closed	Yes	Closed
							Yes [4]	2-LRR-43.1	Inside	3 inch	Globe	1		Diaphragm	CIAS	Closed	Closed	Yes	Closed
							No	2-LRR-291	Outside	1 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

Penetration Number	Service	System	Penetration Type [1]	Penetration Category[2]	Flow Direction	Valve Arrangement	Type C Testing Requirements[3]	Valve Identification	Location Reference to Containment Structure	Valve Size	Valve Type	Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
							No	2-LRR-293, 2-LRR-295	Outside	0.75 inch	Globe	2		Manual	–	Locked Closed	As Is	No	Closed
36	Instrument Air	IA	IA	O	IN	33	Yes	2-IA-566	Outside	0.5 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed [9]
							Yes	2-IA-569	Inside	0.5 inch	Check	1		–	–	–	–	No	–
							No	2-IA-572	Inside	0.5 inch	Gate	1		Manual	–	Locked closed	As Is	No	Closed
37	Instrument Air	IA	IA	O	IN	1A	Yes	2-IA-27.1	Outside	2 inch	Globe	1	2 inch	Diaphragm	Remote	Open	Closed	Yes	Open [9]
							No	2-IA-40	Outside	1 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							Yes	2-IA-43	Inside	2 inch	Check	1		–	–	–	–	No	–
38	Station Air	SA	IA	O	IN	3	Yes	2-SA-19	Outside	2 inch	Gate	1	2 inch	Manual	–	Locked Closed	As Is	No	Closed
							No	2-SA-28	Inside	1 inch	Gate	1	2 inch	Manual	–	Locked Closed	As Is	No	Closed
							Yes	2-SA-22	Inside	2 inch	Check	1		–	–	–	–	No	–
39	Purge Air Inlet	PA	IC	O	IN	4	Yes	2-AC-4	Outside	48 inch	Butterfly	1	48 inch	Air Cylinder	High Contain-ment Radiation	Locked Closed	Closed	Yes	Closed
							Yes[4]	2-AC-5	Inside	48 inch	Butterfly	1		Air Cylinder	High Contain-ment Radiation	Locked Closed	Closed	Yes	Closed
							No	2-AC-21	Outside	0.75 inch	Globe	1		Manual	----	Locked Closed	As Is	No	Closed
40	Purge Air Discharge	PA	IC	O	OUT	5	Yes	2-AC-7	Outside	48 inch	Butterfly	1	48 inch	Air Cylinder	High Contain-ment Radiation	Locked Closed	Closed	Yes	Closed
							Yes[4]	2-AC-6	Inside	48 inch	Butterfly	1		Air Cylinder.	High Contain-ment Radiation	Locked Closed	Closed	Yes	Closed
							No	2-AC-31	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
42	Fuel Transfer Tube	FTS	Special	O	IN/OUT	8	No, Type B	N/A	Inside	Special Closure			36 inch	–	–	Closed	–	–	Closed

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

Penetration Number	Service	System	Penetration Type [1]	Penetration Category[2]	Flow Direction	Valve Arrangement	Type C Testing Requirements[3]	Valve Identification	Location Reference to Containment Structure	Valve Size	Valve Type	Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
							No	2-RW-291	Inside	0.5 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-RW-31	Inside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-RW-292	Inside	0.5 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
43	Reactor Coolant Pump Seals Controlled Bleed Off	CVCS	IA	P	OUT	6	Yes	2-CH-506	Inside	0.75 inch	Globe	1	0.75 inch	Diaphragm	CIAS	Open	Closed	Yes	Closed
							Yes	2-CH-198	Outside	0.75 inch	Globe	1		Diaphragm	CIAS	Open	Closed	Yes	Closed
							Yes	2-CH-505	Outside	0.75 inch	Globe	1		Diaphragm	CIAS	Closed	Closed	Yes	Closed
							No	2-CH-758, 2-CH-768, 2-CH-701	Outside	0.75 inch	Globe	3		Manual	–	Locked Closed	As Is	No	Closed
							No	2-CH-767 [11], 2-CH-766 [11]	Outside	0.75 inch	Globe	2		Manual	–	Locked Open	As Is	No	Open
							No	2-CH-744	Outside	0.75 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
47, 69 70, 71	Pressure Monitoring		IA	Special	IN/OUT	30	No	Penetration 47 2-AC-97 [11]; Penetration 70 2-AC-98 [11]	Outside	0.5 inch	Globe	1	0.5 inch	Manual	–	Open	As Is	Yes	Open
								Penetration 69 2-AC-99 [11]; Penetration 71 2-AC-96 [11]											
51	Waste Gas Header	RWS	IA	N [12]	OUT	12	Yes	2-GR-11.2	Outside	3 inch	Globe	1	3 inch	Diaphragm	CIAS	Closed	Closed	Yes	Closed
							Yes [4]	2-GR-11.1	Inside	3 inch	Globe	1		Diaphragm	CIAS	Closed	Closed	Yes	Closed
							No	2-GR-63	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
49	Fire Protection	Fire	IA	O	IN	34	Yes	2-Fire-108	Outside	6 inch	Butterfly	1	6 inch	Manual	–	Locked Closed	As Is	No	Closed
							No	2-Fire-125	Outside	1 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
							Yes	2-Fire-109	Inside	6 inch	Check	1		–	–	–	As Is	No	Closed

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

Penetration Number	Service	System	Penetration Type [1]	Penetration Category[2]	Flow Direction	Valve Arrangement	Type C Testing Requirements[3]	Valve Identification	Location Reference to Containment Structure	Valve Size	Valve Type	Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
53	Reactor Building Closed Cooling Water Inlet to Reactor Coolant Pumps and Other Components [8]	RBCCW	IA	N	IN	24	Yes	2-RB-30.1B	Outside	6 inch	Gate	1	6 inch	MOV	Remote	Open	As Is	Yes	Open
							No	2-RB-291	Outside	1 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-RB-173B [11]	Outside	0.75 inch	Globe	1		Manual	–	Open	As Is	No	Open
54	Reactor Building Closed Cooling Water Outlet from Reactor Coolant Pumps and Other Components [8]	RBCCW	IA	N	OUT	2	Yes	2-RB-37.2B	Outside	6 inch	Gate	1	6 inch	MOV	Remote	Open	As Is	Yes	Open
							No	2-RB-300	Outside	1 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-RB-299A	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
85	Containment Leak Rate Pressurization		IA	O	IN/OUT	29	No, Type B	N/A	Inside	6 inch	Blind Flange	1		–	–	–	–	No	–
							No, Type B	SF-01	Outside	6 inch	Spectacle Flange	1	6 inch	–	–	–	–	No	–
							No	2-AC-107	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
61 & 86	Containment Air Sample	CAS	IC	O	OUT	26	Yes	Penetration 61: 2-AC-12; Penetration 86: 2-AC-47	Outside	1.5 inch	Butterfly	1	1 inch	Diaphragm	CIAS	Open	Closed	Yes	Closed[10]
							Yes	2-EB-88; 2-EB-89	Inside	1.5 inch	Butterfly	1		Diaphragm	CIAS	Open	Closed	Yes	Closed[10]
							No	2-AC-101; 2-AC-102	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

Penetration Number	Service	System	Penetration Type [1]	Penetration Category[2]	Flow Direction	Valve Arrangement	Type C Testing Requirements[3]	Valve Identification		Location Reference to Containment Structure	Valve Size	Valve Type	Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
62 & 87	Containment Air Sample	CAS	IC	O	IN	28	Yes	Penetration 62 2-AC-54	Penetration. 87 2-AC-55	Inside	0.5 inch	Check	1	1 inch	–	–	–	–	No	–
							Yes	2-AC-15	2-AC-20	Outside	1.5 inch	Butterfly	1		Diaphragm	CIAS	Open	Closed	Yes	Closed[10]
							No	2-AC-103	2-AC-104	Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
67	Refueling Water Purification	RPCS	IA	O	OUT	27A	Yes [4]	2-RW-232		Inside	4 inch	Gate	1	4 inch	Manual	–	Locked Closed	As Is	No	Closed
							Yes	2-RW-21		Outside	4 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-RW-158		Outside	0.75 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
68	Refueling Water Purification	RPCS	IA	O	IN	27B	Yes [4]	2-RW-154		Inside	4 inch	Gate	1	4 inch	Manual	–	Locked Closed	As Is	No	Closed
							Yes	2-RW-63		Outside	4 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-RW-159		Outside	0.75 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
82	Hydrogen Purge	HC	IA	O	OUT	25A	Yes [4]	2-EB-91		Inside	6 inch	Butterfly		6 inch	Air Cylinder	CIAS & High Contain-ment Radiation	Closed	As Is	Yes	Closed
							Yes	2-EB-92		Outside	6 inch	Butterfly	1		Diaphragm	CIAS & High Contain-ment Radiation	Closed	Closed	Yes	Closed
							Yes [4]	2-EB-86		Inside	0.75 inch	Gate	1		Manual	–	Locked Closed	As Is	No	Closed
							No	2-EB-120		Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
83	Hydrogen Purge	HC	IA	O	OUT	25B	Yes [4]	2-EB-100		Inside	6 inch	Butterfly	1	6 inch	Air Cylinder	CIAS & High Contain-ment Radiation	Closed	As Is	Yes	Closed

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-11 CONTAINMENT STRUCTURE ISOLATION VALVE INFORMATION (CONTINUED)

Penetration Number	Service	System	Penetration Type[1]	Penetration Category[2]	Flow Direction	Valve Arrangement	Type C Testing Requirements[3]	Valve Identification		Location Reference to Containment Structure	Valve		Number	Penetration Line Size	Method of Actuation	Signal	Normal Valve Position	Valve Position with Power Failure	Position Indication	Post Incident Position
											Size	Type								
							Yes	2-EB-99		Outside	6 inch	Butterfly	1		Diaphragm	CIAS & High Contain-ment Radiation	Closed	Closed	Yes	Closed
							No	2-EB-121		Outside	0.75 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
65 & 72	Steam Generator Blowdown Sample	SGBS	IA	N	OUT	14	No	Penetration 65 2-MS-191A	Penetration 72 2-MS-191B	Outside	0.5 inch	Globe	1	0.5 inch	Diaphragm	CIAS	Open	Closed	Yes	Closed
63 & 64	Containment Pressure Test Connection	ILRT	IC	O	OUT	31	Yes	Penetration 63 2-AC-114	Penetration. 64 2-AC-112	Outside	1 inch	Globe	1	1 inch	Manual	–	Locked Closed	As Is	No	Closed
							Yes	2-AC-117	2-AC-116	Inside	1 inch	Globe	1		Manual	–	Locked Closed	As Is	No	Closed
							No, Type B	1 inch -Blind Flange TC	1 inch - Blind Flange TC	Outside	1 inch	Blind Flange	2	1 inch	–	–	–	–	No	–

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

NOTES:
1. See Figure 5.2–8.
2. See Section 5.2.7.2.1.
3. Containment Isolation Valve Test (Type C) per 10 CFR Part 50, Appendix J.
4. Valve tested with pressure applied opposite to that applied during LOCA.
5. Valve 2-CH-435 no longer functions as a check valve; its internal disk and piston spring have been removed.
6. If steam generator pressure drops to $\leq$ 572 psia.
7. Valves 2-MS-65A, 2-MS-65B, 2-MS-202 and 2-SI-651 have disconnect switches in their power circuits to ensure the valves are in their proper position in the event of an Appendix R fire (hot short issue). Once these valves have been positioned and the disconnect switch is placed in the “off” position, the valves will not respond to automated signals. See FSAR Section 9.10.6.3, item (2), and Appendix R Compliance Report.
8. “Other components” include Reactor Vessel Support Cooling Coils, CEDM Coolers, and the Quench Tank & PDT Heat Exchanger.
9. The post incident position 2-IA-566 and 2-IA-27.1 is the same as their normal position unless changed in accordance with plant procedures.
10. Valves 2-EB-88, 2-EB-89, 2-AC-12, 2-AC-15, 2-AC-20, 2-AC-47 may be opened to permit post-accident hydrogen sampling of the Containment atmosphere. Thus, the sample systems, excluding the Radiation Monitor skids RM8123 & RM8262, are included in the 10 CFR 50, Appendix J testing at the accident pressure.
11. Valves within containment boundary that are instrument isolation valves or valves not required to go closed or be closed for isolation of the penetration.
12. The allowed outage time of 72 hours as specified in Technical Specification action statement 3.6.3.1.d is applicable for all ‘N’ Type penetrations except penetration numbers 1, 34, 51.

Note: When making changes to this table, also refer to the Technical Requirements Manual, Table 3.6-1, Containment Isolation Valve List.

TABLE 5.2-12 CONTAINMENT PENETRATION PIPING

Penetration Number	System	PIPE			Material	Class	Code	Fittings
		Size	Schedule	Line Designation				
1	DWS	2 inch	40S	HCB-4	A-312 TP 304	2	B31.7	3000 pound Socket Weld
2	CVCS	2 inch	160	CCB-5	A-376 TP 316	2	B31.7	6000 pound Socket Weld
3	CVCS	2 inch	160	CCB-6	A-376 TP 316	2	B31.7	6000 pound Socket Weld
4	CSS	8 inch	20	GCB-11	A-312 TP 304	2	B31.7	Butt welded
5	CSS	8 inch	20	GCB-11	A-312 TP 304	2	B31.7	Butt welded
6	SIS	6 inch	120	CCA-6	A-376 TP 316	2	B31.7	Butt welded
7	SIS	6 inch	120	CCA-6	A-376 TP 316	2	B31.7	Butt welded
8	SIS	6 inch	120	CCA-6	A-376 TP 316	2	B31.7	Butt welded
9	SIS	6 inch	120	CCA-6	A-376 TP 316	2	B31.7	Butt welded
10	SIS	12 inch	20	GCB-1	A-376 TP 316	2	B31.7	Butt welded
11	SIS	2 inch	40S	GCB-14	A-312 TP 304	2	B31.7	3000 pound Socket Weld
12	SIS	24 inch	10S	HCB-1	A-312 TP 304	2	B31.7	Butt welded
13	SIS	24 inch	10S	HCB-1	A-312 TP 304	2	B31.7	Butt welded
14	RWS	3 inch	10S	HSB-1	A-312 TP 304	2	B31.7	Butt welded
15	FW	18 inch	60	EBB-6	A-106 GR B	2	B31.7	Butt welded
16	FW	18 inch	60	EBB-6	A-106 GR B	2	B31.7	Butt welded
19	MSS	34 inch	0.977 inch wall	EBB-2	A-155 GR KCF70	2	B31.7	Butt welded
20	MSS	34 inch	0.977 inch wall	EBB-2	A-155 GR KCF70	2	B31.7	Butt welded

TABLE 5.2-12 CONTAINMENT PENETRATION PIPING (CONTINUED)

Penetration Number	System	PIPE			Material	Class	Code	Fittings
		Size	Schedule	Line Designation				
21	SS	0.75 inch	160	CCB-10	A-376 TP 316	2	B31.7	6000 pound Socket Weld
22	SGBS	2 inch	80	EBB-5	A-106 GR B	2	B31.7	3000 pound Socket Weld
23	SGBS	2 inch	80	EBB-5	A-106 GR B	2	B31.7	3000 pound Socket Weld
24	RBCCW	8 inch	40	HBB-5	A-333 GR 6	2	B31.7	Butt welded
25	RBCCW	10 inch	40	HBB-3	A-333 GR 6	2	B31.7	Butt welded
26	RBCCW	10 inch	40	HBB-3	A-333 GR 6	2	B31.7	Butt welded
27	RBCCW	10 inch	40	HBB-3	A-333 GR 6	2	B31.7	Butt welded
28	RBCCW	10 inch	40	HBB-3	A-333 GR 6	2	B31.7	Butt welded
29	RBCCW	8 inch	40	HBB-3	A-333 GR 6	2	B31.7	Butt welded
30	RBCCW	10 inch 6 inch	40	HBB-3	A-333 GR 6	2	B31.7	Butt welded
31	RBCCW	10 inch 6 inch	40	HBB-3	A-333 GR 6	2	B31.7	Butt welded
32	RBCCW	10 inch 6 inch	40	HBB-3	A-333 GR 6	2	B31.7	Butt welded
33	RBCCW	10 inch 6 inch	40	HBB-4	A-333 GR 6	2	B31.7	Butt welded
34	NS	1 inch	40S	GCB-12	A-312 TP 304	2	B31.7	3000 pound Socket Weld
35	RWS	4 inch	10S	HSB-2	A-312 TP 304	2	B31.7	Butt welded
36	IA	0.5 inch	40S	HCB-31	A-312 TP 304	2	B31.7	3000 pound Socket Weld

TABLE 5.2-12 CONTAINMENT PENETRATION PIPING (CONTINUED)

Penetration Number	System	PIPE			Material	Class	Code	Fittings
		Size	Schedule	Line Designation				
37	IA	2 inch	80	HBB-13	A-333 GR 6	2	B31.7	3000 pound Socket Weld
38	SA	2 inch	80	HBB-12	A-333 GR 6	2	B31.7	3000 pound Socket Weld
39	PA	48 inch	0.375 inch wall	HBB-7	A-333 GR 6	2	B31.7	Butt welded
40	PA	48 inch	0.375 inch	HBB-8	A-333 GR 6	2	B31.7	Butt welded
43	CVCS	0.75 inch	160	CCB-9	A-376 TP 316	2	B31.7	6000 pound Socket Weld
49	Fire Protection	6 inch	40	HBB-19	SA-106 GR B	2	ASME	Butt welded
51	RWS	3 inch	40	HRB-1	A-106 GR B	2	B31.7	Butt welded
53	RBCCW	6 inch	40	HBB-5	A-333 GR 6	2	B31.7	Butt welded
54	RBCCW	6 inch	40	HBB-6	A-333 GR 6	2	B31.7	Butt welded
61	CAS	1 inch	40S	HCB-9	A-312 TP 304	2	B31.7	3000 pound Socket Weld
62	CAS	1 inch	40S	HCB-9	A-312 TP 304	2	B31.7	3000 pound Socket Weld
65	SBGS	0.5 inch	80	EBB-8	A-106 GR B	2	B31.7	3000 pound Socket Weld
67	SFPCS	4 inch	10S	HCB-10	A-312 TP 304	2	B31.7	Butt welded
68	SFPCS	4 inch	10S	HCB-11	A-312 TP 304	2	B31.7	Butt welded
72	SGBS	0.5 inch	80	EBB-8	A-106 GR B	2	B31.7	3000 pound Socket Weld
82	H_2 Purge	6 inch	40	HBB-10	A-333 GR 6	2	B31.7	Butt welded
83	H_2 Purge	6 inch	40	HBB-10	A-333 GR 6	2	B31.7	Butt welded

TABLE 5.2-12 CONTAINMENT PENETRATION PIPING (CONTINUED)

Penetration Number	System	PIPE			Material	Class	Code	Fittings
		Size	Schedule	Line Designation				
85	Leak Rate Test	6 inch	40	HBB-1	A-333 GR 6	2	B31.7	Butt welded
86	CAS	1 inch	40S	HCB-9	A-312 TP 304	2	B31.7	3000 pound Socket Weld
87	CAS	1 inch	40S	HCB-9	A-312 TP 304	2	B31.7	3000 pound Socket Weld

Penetration 17 - Personnel Airlock.

Penetration 18 - Equipment Hatch.

Penetration 42 - Fuel Transfer Tube.

Penetrations 41, 46, 48, 50, 52, 55, 56, 57, 58, 59, 60, 66, 73, 74, 78, 79, 80, 81, 84, 88, 89 - SPARE.

Penetrations 47, 69, 70, 71 - Containment Pressure Transmission.

Penetration 75 - Electrical

Penetrations 63, 64 - Test Connection.

Penetrations 6, 7, 8, 9 were constructed to Class 1 requirements but are considered Class 2 piping upstream of valves 2-SI-706A, B, C, D.

TABLE 5.2-13 MAJOR[1] CONTAINMENT ISOLATION VALVES

VALVE ID NUMBER	PENETRATION NUMBER	SIZE	VALVE TYPE	BODY MATERIAL [2]	CODE	NUCLEAR CLASS	RATING
2-PMW-43	1	2 inch	Globe	A351, GR CF8M	*	II	150 pound
2-PMW-3	1	2 inch	Check	A182, GR F-316	*	II	600 pound
2-CH-006	2	2 inch	Gate	A182, GR F-316	*	II	1500 pound
2-CH-516	2	3 inch	Globe	A182, GR F-316	*	II	1500 pound
2-CH-089	2	2 inch	Globe	A182, GR F-316	*	II	1500 pound
2-CH-515	2	3 inch	Globe	A182, GR F-316	*	II	1500 pound
2-CH-434	3	2 inch	Gate	A182, GR F-316	*	II	1500 pound
2-CH-429	3	2 inch	Gate	A182, GR F-316	*	II	1500 pound
2-CH-518	3	2 inch	Globe	A351, GR CF-8	*	II	1500 pound
2-CH-519	3	2 inch	Globe	A351, GR CF-8	*	II	1500 pound
2-CH-517	3	2 inch	Globe	A351, GR CF-8	*	II	1500 pound
2-CH-435	3	2 inch	Check	A182, GR F-316	*	II	1500 pound
2-CS-5A	4	8 inch	Check	A351, GR CF-8	*	II	300 pound
2-CS-4.1A	4	8 inch	Gate	A351, GR CF-8	*	II	300 pound
2-CS-5B	5	8 inch	Check	A351, GR CF-8	*	II	300 pound
2-CS-4.1B	5	8 inch	Gate	A351, GR CF-8	*	II	300 pound
2-SI-645	6	6 inch	Globe	A182, GR F-316	*	II	1500 pound
2-SI-144	6	6 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-646	6	2 inch	Globe	A182, GR F-316	*	II	1500 pound
2-SI-647	6	2 inch	Globe	A182, GR F-316	*	II	1500 pound

TABLE 5.2-13 MAJOR[1] CONTAINMENT ISOLATION VALVES (CONTINUED)

VALVE ID NUMBER	PENETRATION NUMBER	SIZE	VALVE TYPE	BODY MATERIAL [2]	CODE	NUCLEAR CLASS	RATING
2-SI-009	6	2 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-143	6	2 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-706D	6	6 inch	Check	A351, GR CF-8	*	I	1500 pound
2-SI-160	6	3 inch	Gate	SA351, GR CF8M	*	II	1500 pound
2-SI-161	6	3 inch	Gate	SA351, GR CF8M	*	II	1500 pound
2-SI-615	7	6 inch	Globe	A182, GR F-316	*	II	1500 pound
2-SI-114	7	6 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-616	7	2 inch	Globe	A182, GR F-316	*	II	1500 pound
2-SI-617	7	2 inch	Globe	A182, GR F-316	*	II	1500 pound
2-SI-012	7	2 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-113	7	2 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-706A	7	6 inch	Check	A351, GR CF-8	*	I	1500 pound
2-SI-636	8	2 inch	Globe	A182, GR F-316	*	II	1500 pound
2-SI-637	8	2 inch	Globe	A182, GR F-316	*	II	1500 pound
2-SI-010	8	2 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-133	8	2 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-706C	8	6 inch	Check	A351, GR CF-8	*	I	1500 pound
2-SI-635	8	6 inch	Globe	A182, GR F-316	*	II	1500 pound
2-SI-134	8	6 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-625	9	6 inch	Globe	A182, GR F-316	*	II	1500 pound

TABLE 5.2-13 MAJOR[1] CONTAINMENT ISOLATION VALVES (CONTINUED)

VALVE ID NUMBER	PENETRATION NUMBER	SIZE	VALVE TYPE	BODY MATERIAL [2]	CODE	NUCLEAR CLASS	RATING
2-SI-124	9	6 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-626, 2-SI-627	9	2 inch	Globe	A182, GR F-316	*	II	1500 pound
2-SI-123	9	2 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-011	9	2 inch	Check	A182, GR F-316	*	II	1500 pound
2-SI-706B	9	6 inch	Check	A351, GR CF-8	*	I	1500 pound
2-SI-709	10	12 inch	Gate	A351, GR CF-8	*	I	1500 pound
2-SI-651	10	12 inch	Gate	A182, GR F-316	*	I	1500 pound
2-SI-463	11	2 inch	Gate	A182, GR F-316	*	II	CL800
2-CS-16.1A	12	24 inch	Gate	A351, GR CF-8	*	II	150 pound
2-CS-16.1B	13	24 inch	Gate	A351, GR CF-8	*	II	150 pound
2-SSP-16.2	14	3 inch	Globe	A351, GR CF8M	*	II	150 pound
2-SSP-16.1	14	3 inch	Globe	A351, GR CF8M	*	II	150 pound
2-FW-5A	15	18 inch	Check	A216, GR WCB	*	II	600 pound
2-FW-12A	15	6 inch	Check	A216, GR WCB	*	II	600 pound
2-FW-5B	16	18 inch	Check	A216, GR WCB	*	II	600 pound
2-FW-12B	16	6 inch	Check	A216, GR WCB	*	II	600 pound
2-MS-64A	19	34 inch	Check	A216, GR WCB	*	II	600 pound
2-MS-201	19	4 inch	Gate	A105, GR 2	*	II	600 pound
2-MS-3A	19	12 inch	Gate	A105, GR 2	*	II	600 pound

TABLE 5.2-13 MAJOR[1] CONTAINMENT ISOLATION VALVES (CONTINUED)

VALVE ID NUMBER	PENETRATION NUMBER	SIZE	VALVE TYPE	BODY MATERIAL [2]	CODE	NUCLEAR CLASS	RATING
2-MS-190A	19	8 inch	Globe	A216, GR WCB	*	II	600 pound
2-MS-265B	19	1 inch	Globe	A182, GR F-316	*	II	600 pound
2-MS-65A	19	3 inch	Globe	A105, GR 2	*	II	600 pound
2-MS-64B	20	34 inch	Check	A216, GR WCB	*	II	600 pound
2-MS-202	20	4 inch	Gate	A105, GR 2	*	II	600 pound
2-MS-3B	20	12 inch	Gate	A105, GR 2	*	II	600 pound
2-MS-190B	20	8 inch	Globe	A216, GR WCB	*	II	600 pound
2-MS-266B	20	1 inch	Globe	A182, GR F-316	*	II	600 pound
2-MS-65B	20	3 inch	Globe	A105, GR 2	*	II	600 pound
2-LRR-61.1	21	0.5 inch	Globe	A351, GR CF8M	*	II	2500 pound
2-RC-001	21	0.75 inch	Globe	A351, GR CF8M	*	II	2500 pound
2-RC-002	21	0.75 inch	Globe	A351, GR CF8M	*	II	2500 pound
2-RC-003	21	0.75 inch	Globe	A351, GR CF8M	*	II	2500 pound
2-RC-45	21	0.75 inch	Globe	A351, GR CF8M	*	II	2500 pound
2-MS-220A	22	2 inch	Globe	A216, GR WCB	*	II	600 pound
2-MS-220B	23	2 inch	Globe	A216, GR WCB	*	II	600 pound
2-RB-30.1A	24	8 inch	Gate	A350, GR LF1	*	II	150 pound

TABLE 5.2-13 MAJOR(1) CONTAINMENT ISOLATION VALVES (CONTINUED)

VALVE ID NUMBER	PENETRATION NUMBER	SIZE	VALVE TYPE	BODY MATERIAL (2)	CODE	NUCLEAR CLASS	RATING
2-RB-28.1D	25	10 inch	Butterfly	A516, GR 70	*	II	150 pound
2-RB-28.1B	26	10 inch	Butterfly	A516, GR 70	*	II	150 pound
2-RB-28.1A	27	10 inch	Butterfly	A516, GR 70	*	II	150 pound
2-RB-28.1C	28	10 inch	Butterfly	A516, GR 70	*	II	150 pound
2-RB-37.2A	29	8 inch	Gate	A350, GR LF1	*	II	150 pound
2-RB-28.3D	30	10 inch	Butterfly	A516, GR 70	*	II	150 pound
2-RB-28.2D	30	6 inch	Butterfly	A516, GR 70	*	II	150 pound
2-RB-28.3B	31	10 inch	Butterfly	A516, GR 70	*	II	150 pound
2-RB-28.2B	31	6 inch	Butterfly	A516, GR 70	*	II	150 pound
2-RB-28.3A	32	10 inch	Butterfly	A516, GR 70	*	II	150 pound
2-RB-28.2A	32	6 inch	Butterfly	A516, GR 70	*	II	150 pound
2-RB-28.3C	33	10 inch	Butterfly	A516, GR 70	*	II	150 pound
2-RB-28.2C	33	6 inch	Butterfly	A516, GR 70	*	II	150 pound
2-SI-312	34	0.75 inch	Globe	A182, GR F-316	*	II	150 pound
2-LRR-43.1	35	3 inch	Globe	A351, GR CF8M	*	II	150 pound
2-LRR-43.2	35	3 inch	Globe	A351, GR CF8M	*	II	150 pound
2-IA-569	36	0.5 inch	Check	A182, GR F-316	ASME 1983	II	600 pound
2-IA-566	36	0.5inch	Gate	A182, GR F-316	ASME 1983	II	600 pound

TABLE 5.2-13 MAJOR[1] CONTAINMENT ISOLATION VALVES (CONTINUED)

VALVE ID NUMBER	PENETRATION NUMBER	SIZE	VALVE TYPE	BODY MATERIAL [2]	CODE	NUCLEAR CLASS	RATING
2-IA-27.1	37	2 inch	Globe	A216, GR WCB	*	II	600 pound
2-IA-43	37	2 inch	Check	A216, GR WCB	*	II	600 pound
2-SA-19	38	2 inch	Gate	A105, GR 2	*	II	600 pound
2-SA-22	38	2 inch	Check	A105, GR 2	*	II	600 pound
2-AC-4	39	48 inch	Butterfly	A516, GR 70	*	II	150 pound
2-AC-5	39	48 inch	Butterfly	A516, GR 70	*	II	150 pound
2-AC-6	40	48 inch	Butterfly	A516, GR 70	*	II	150 pound
2-AC-7	40	48 inch	Butterfly	A516, GR 70	*	II	150 pound
2-CH-506	43	0.75 inch	Globe	A351, GR CF8M	*	II	2500 pound
2-CH-198	43	0.75 inch	Globe	A351, GR CF8M	*	II	2500 pound
2-CH-505	43	0.75 inch	Globe	A351, GR CF8M	*	II	2500 pound
2-FIRE-108	49	6 inch	Butterfly	A216, GR WCB	ASME 1977	II	150 pound
2-FIRE-109	49	6 inch	Check	A216, GR WCB	ASME 1977	II	150 pound
2-GR-11.2	51	3 inch	Globe	A216, GR WCB	*	II	150 pound
2-GR-11.1	51	3 inch	Globe	A216, GR WCB	*	II	150 pound
2-RB-30.1B	53	6 inch	Gate	A105, GR 2	*	II	150 pound

TABLE 5.2-13 MAJOR[1] CONTAINMENT ISOLATION VALVES (CONTINUED)

VALVE ID NUMBER	PENETRATION NUMBER	SIZE	VALVE TYPE	BODY MATERIAL [2]	CODE	NUCLEAR CLASS	RATING
2-RB-37.2B	54	6 inch	Gate	A105, GR 2	*	II	150 pound
2-AC-12	61	1.5 inch	Butterfly	A515, GR 70	*	II	300 pound
2-EB-88	61	1.5 inch	Butterfly	A515, GR 70	*	II	300 pound
2-AC-15	62	1.5 inch	Butterfly	A515, GR 70	*	II	300 pound
2-AC-54	62	0.5 inch	Check	A182, GR F-316	*	II	600 pound
2-MS-191A	65	0.5 inch	Globe	A216, GR WCB	*	II	600 pound
2-RW-232	67	4 inch	Gate	A182, GR F-316	*	II	150 pound
2-RW-21	67	4 inch	Gate	A182, GR F-316	*	II	150 pound
2-RW-154	68	4 inch	Gate	A182, GR F-316	*	II	150 pound
2-RW-63	68	4 inch	Gate	A182, GR F-316	*	II	150 pound
2-MS-191B	72	0.5 inch	Globe	A216, GR WCB	*	II	600 pound
2-EB-91	82	6 inch	Butterfly	A516, GR 70	*	II	150 pound
2-EB-92	82	6 inch	Butterfly	A516, GR 70	*	II	150 pound
2-EB-100	83	6 inch	Butterfly	A516, GR 70	*	II	150 pound
2-EB-99	83	6 inch	Butterfly	A516, GR 70	*	II	150 pound
2-AC-47	86	1.5 inch	Butterfly	A515, GR 70	*	II	150 pound
2-EB-89	86	1.5 inch	Butterfly	A515, GR 70	*	II	150 pound
2-AC-20	87	1.5 inch	Butterfly	A515, GR 70	*	II	150 pound
2-AC-55	87	0.5 inch	Check	A182, GR F-316	*	II	600 pound

Notes:

1 “Major” valves are principal valves used for containment integrity and process line function (does not include test, vent, drain valves).

2 ASTM material for valve body indicated. Designation is ‘A’ or ‘SA’ per controlling code.

* ASME Section III, 1971, or Draft ASME Pump and Valve Code Penetration numbers 41, 46, 48, 50, 52, 55, 56, 57, 58, 59, 60, 66, 73, 74, 78, 79, 80, 81, 84, 88, 89 -
SPARE Penetration number 85 - Leak Rate Testing Penetration numbers 63, 64 - Test Connection.

TABLE 5.2-14 TYPICAL LEAK RATE MEASUREMENT SYSTEM INSTRUMENTATION

QUANTITY	DESCRIPTION
18	Temperature Monitoring System: Resistance Temperature Detector; Accuracy: ± 0.5°F; Sensitivity: ± 0.1°F.
6	Dewpoint Temperature Monitoring System: Accuracy: ± 2°F; Sensitivity: ± 0.5°F.
2	Flowmeters: Mass Flow Meters; Accuracy: ± 2.0% full scale; Sensitivity: ± 1.0% full scale.
2	Pressure Monitoring: Precision Pressure Gages; Accuracy: ± 0.02% of reading; Sensitivity: ± 0.001 psi.

NOTE: Instrumentation listed above are typically used during an ILRT. The instrumentation parameters are obtained from ANS 56.8 and are used as guidance in selecting acceptable instruments. Appropriate alternatives to the above instrumentation can be used for the ILRT.

TABLE 5.2-15 TYPICAL CONTAINMENT RESISTANCE TEMPERATURE DETECTORS AND DEWCELL SENSOR VOLUME WEIGHT FRACTIONS

RTD	Elevation (feet)	AZ (Degrees)	Distance From Centerline (feet)	Volume Fraction
9769	150	90	12	0.127
8110	95	220	65	0.091
9767	95	40	65	0.091
8111	90	310	60	0.091
8112	90	130	60	0.091
8084	40	5	45	0.071
8108	44	135	60	0.071
8109	44	265	60	0.071
8097	30	95	20	0.026
8098	30	235	20	0.020
8094	20	350	45	0.029
9770	18	220	55	0.028
9771	18	90	50	0.028
8087	3	5	32	0.020
9765	3	240	65	0.020
9766	3	125	65	0.021
8091	-15	330	35	0.052
9768	-10	115	50	0.052
			TOTAL 1.000	

Dewcells	Elevation (feet)	AZ (Degrees)	Distance From Centerline (feet)	Volume Fraction
8101	105	40	45	0.245
8090	105	220	45	0.245
5458	45	300	45	0.150
8102	45	120	45	0.150
8093	-4	220	45	0.105
5457	-4	40	45	0.105
			TOTAL 1.000	

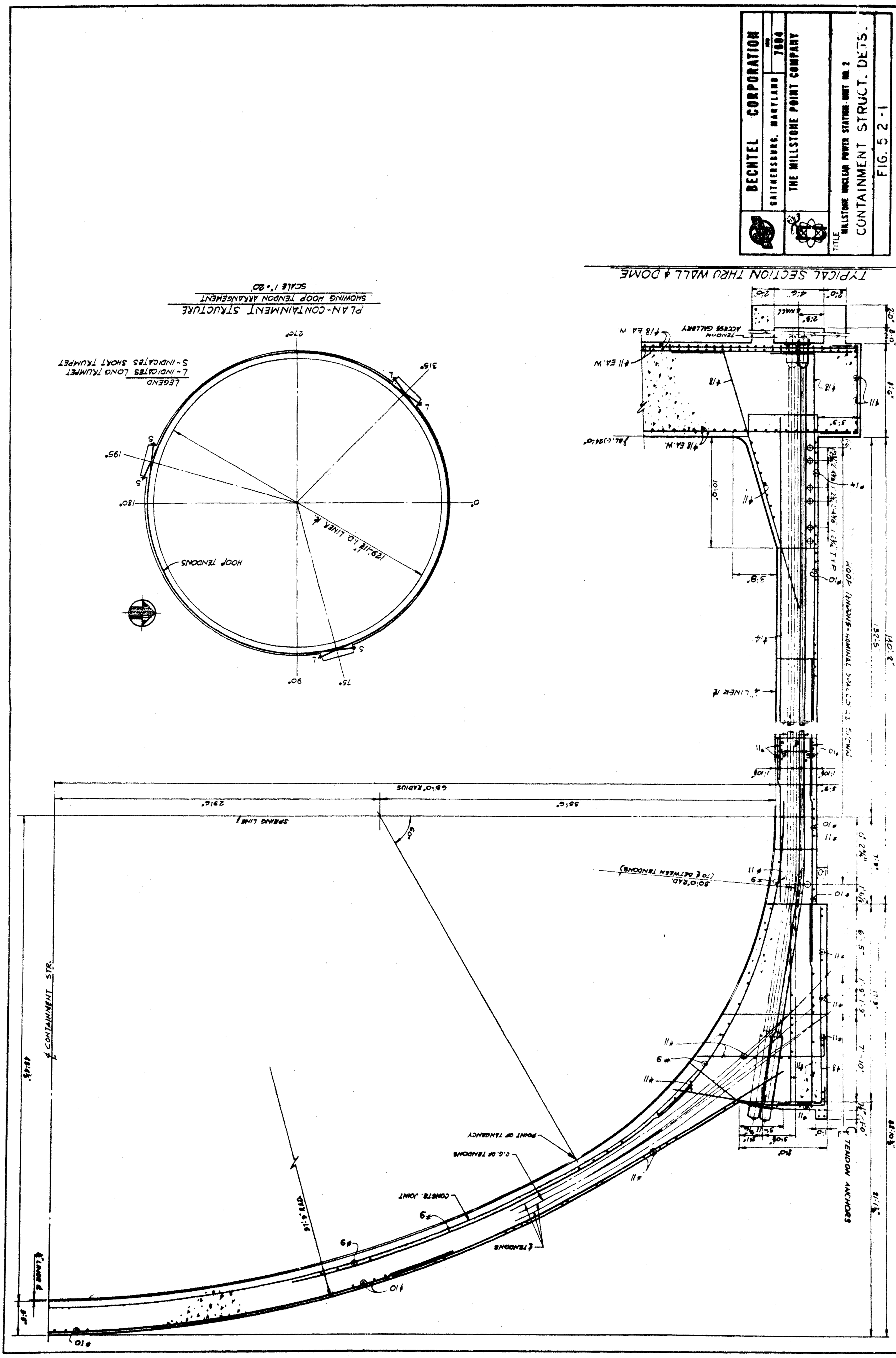


FIGURE 5.2–1 CONTAINMENT STRUCTURE DETAILS

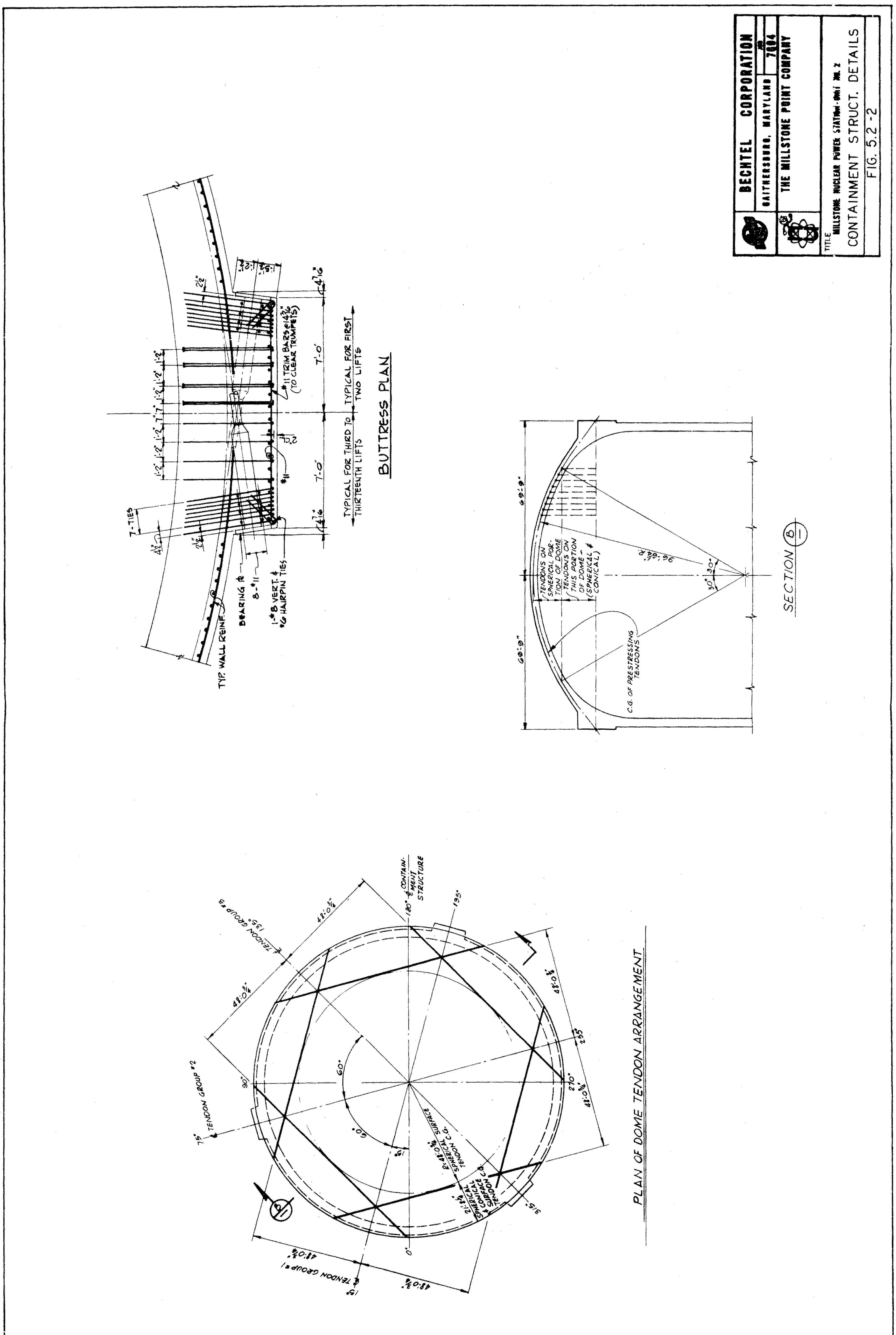
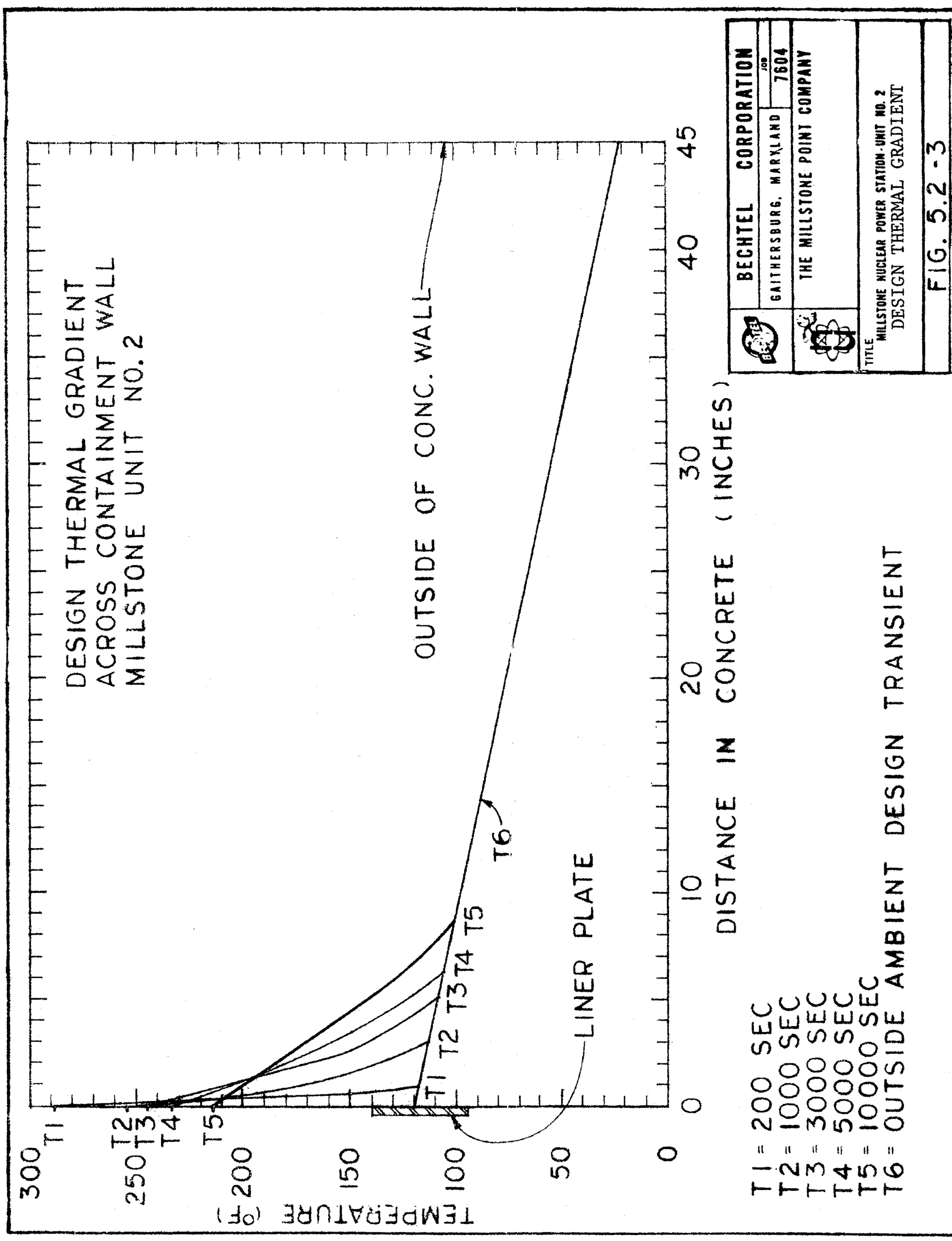


FIGURE 5.2–2 CONTAINMENT STRUCTURE DETAILS

FIGURE 5.2–3 DESIGN THERMAL GRADIENT



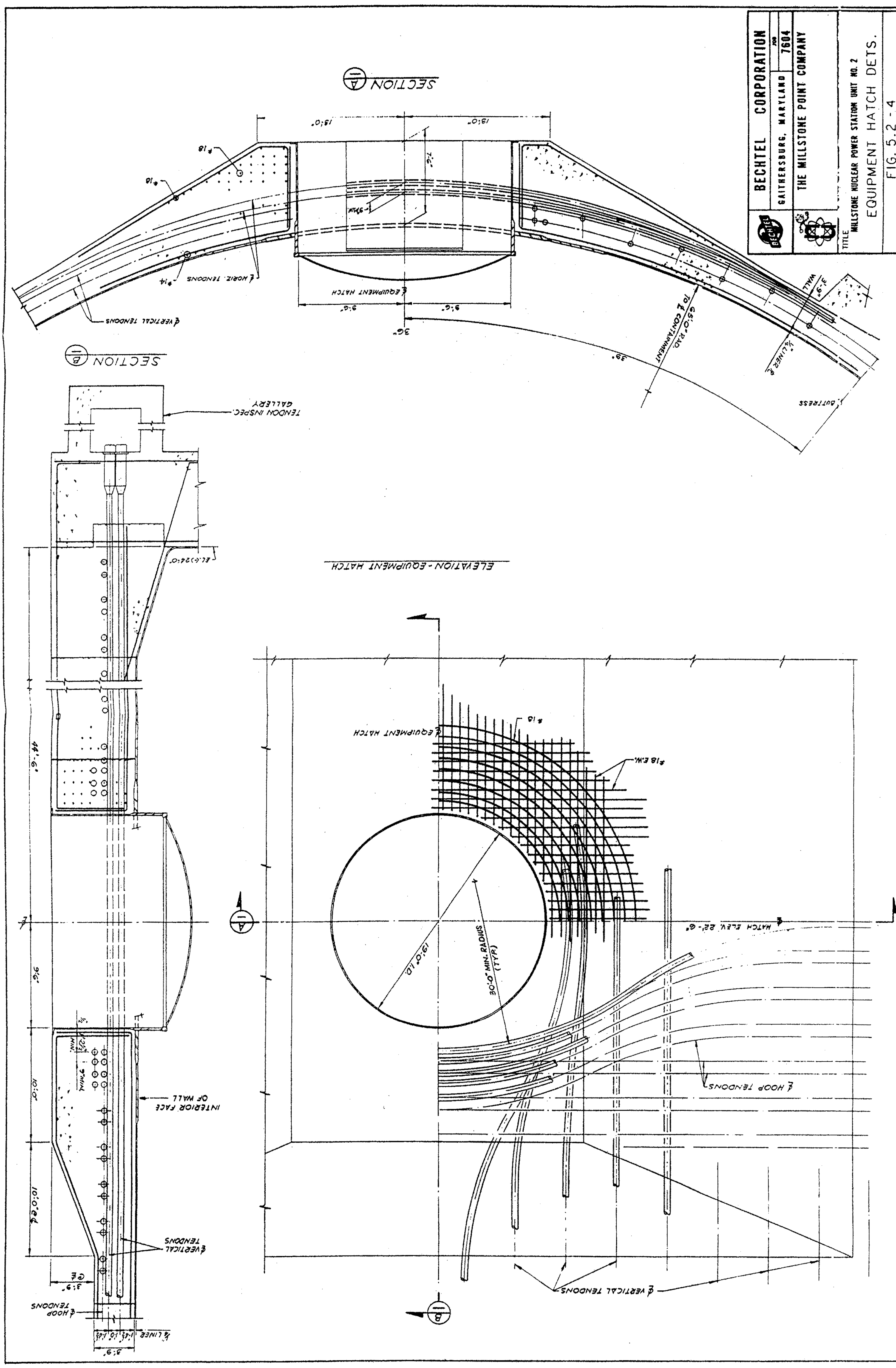


FIGURE 5.2–4 EQUIPMENT HATCH DETAILS

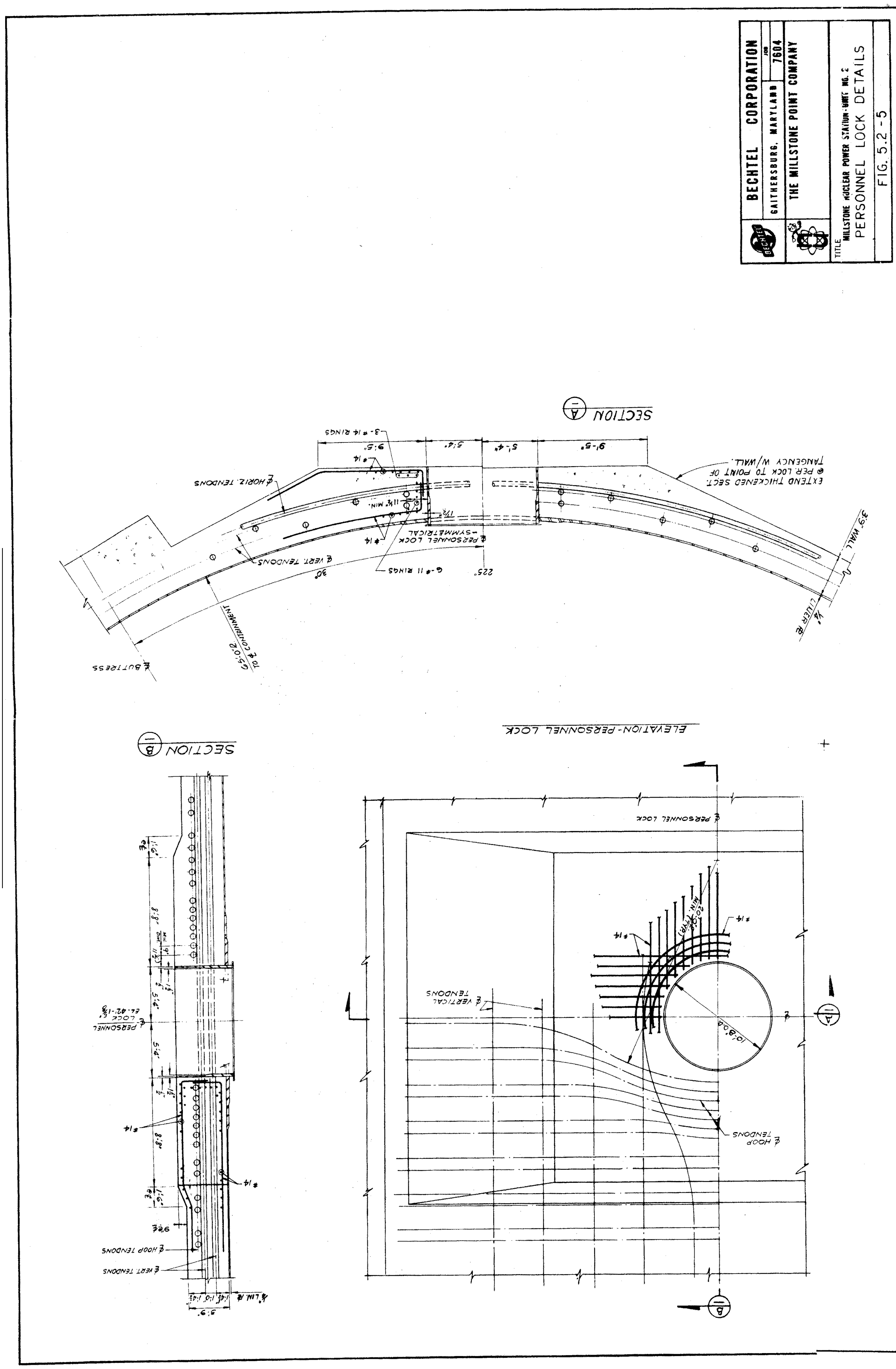


FIGURE 5.2–5 PERSONNEL LOCK DETAILS

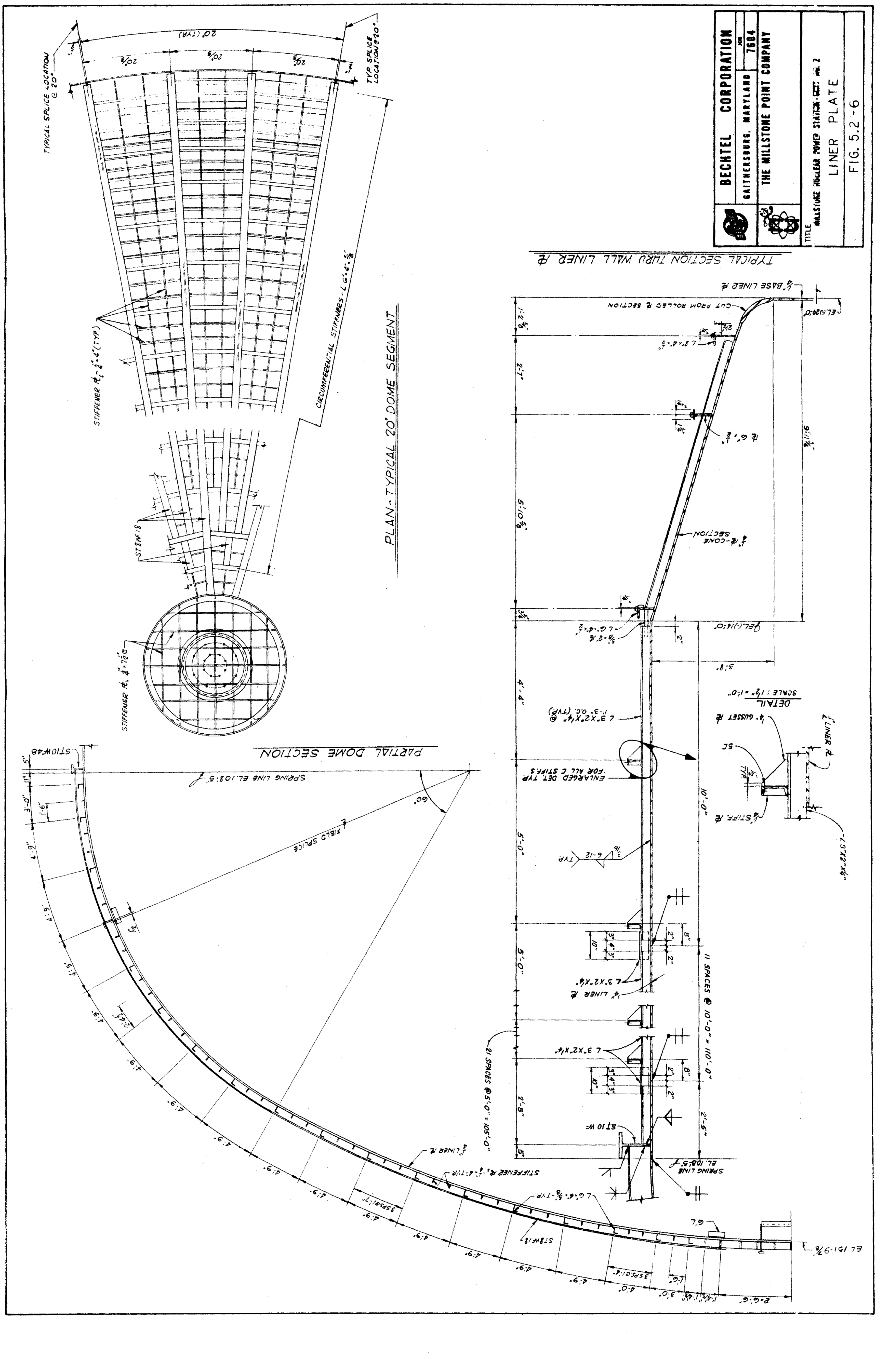


FIGURE 5.2–6 LINER PLATE

FIGURE 5.2–7 LEAK CHASE CHANNELS

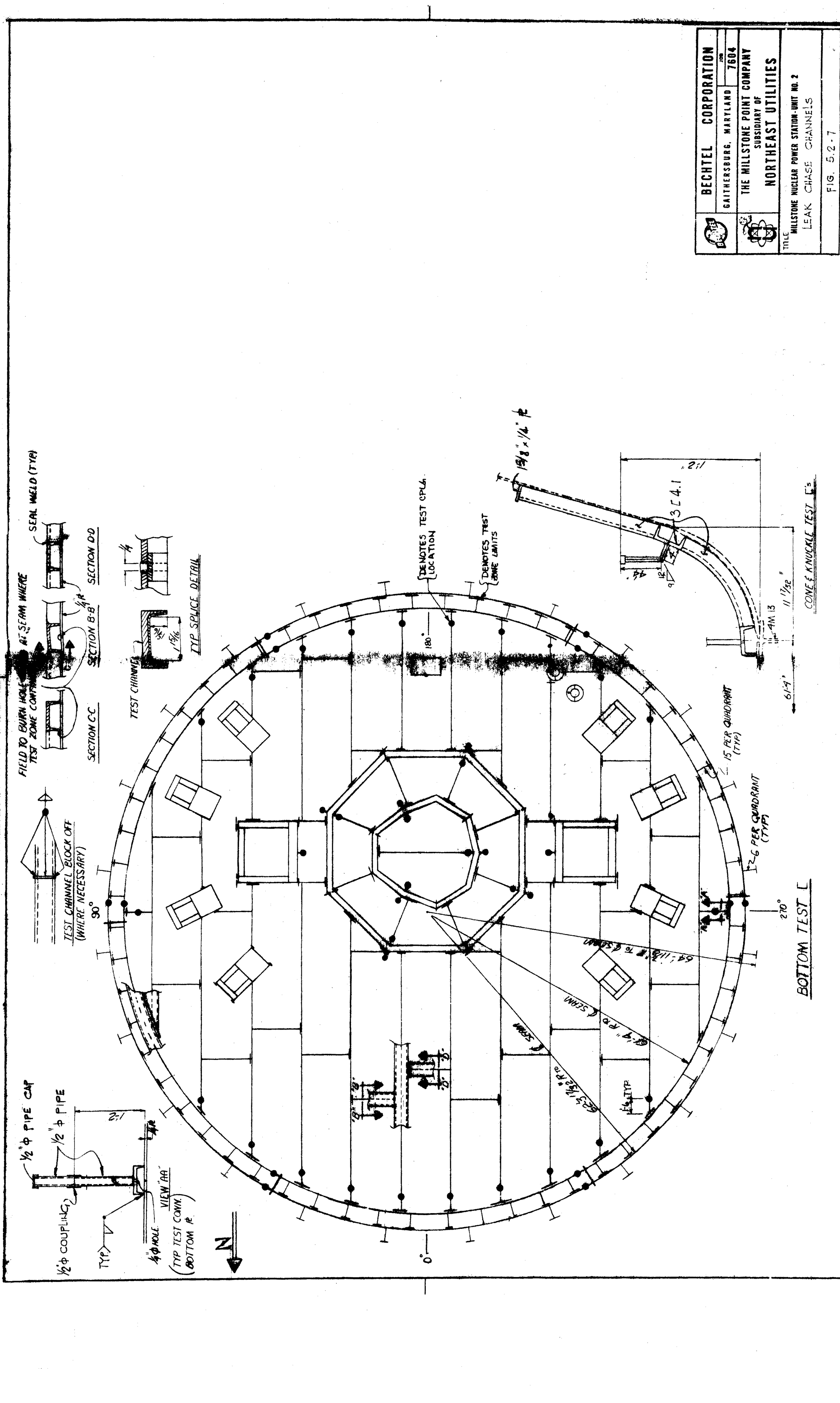


FIGURE 5.2–8 TYPICAL PENETRATIONS

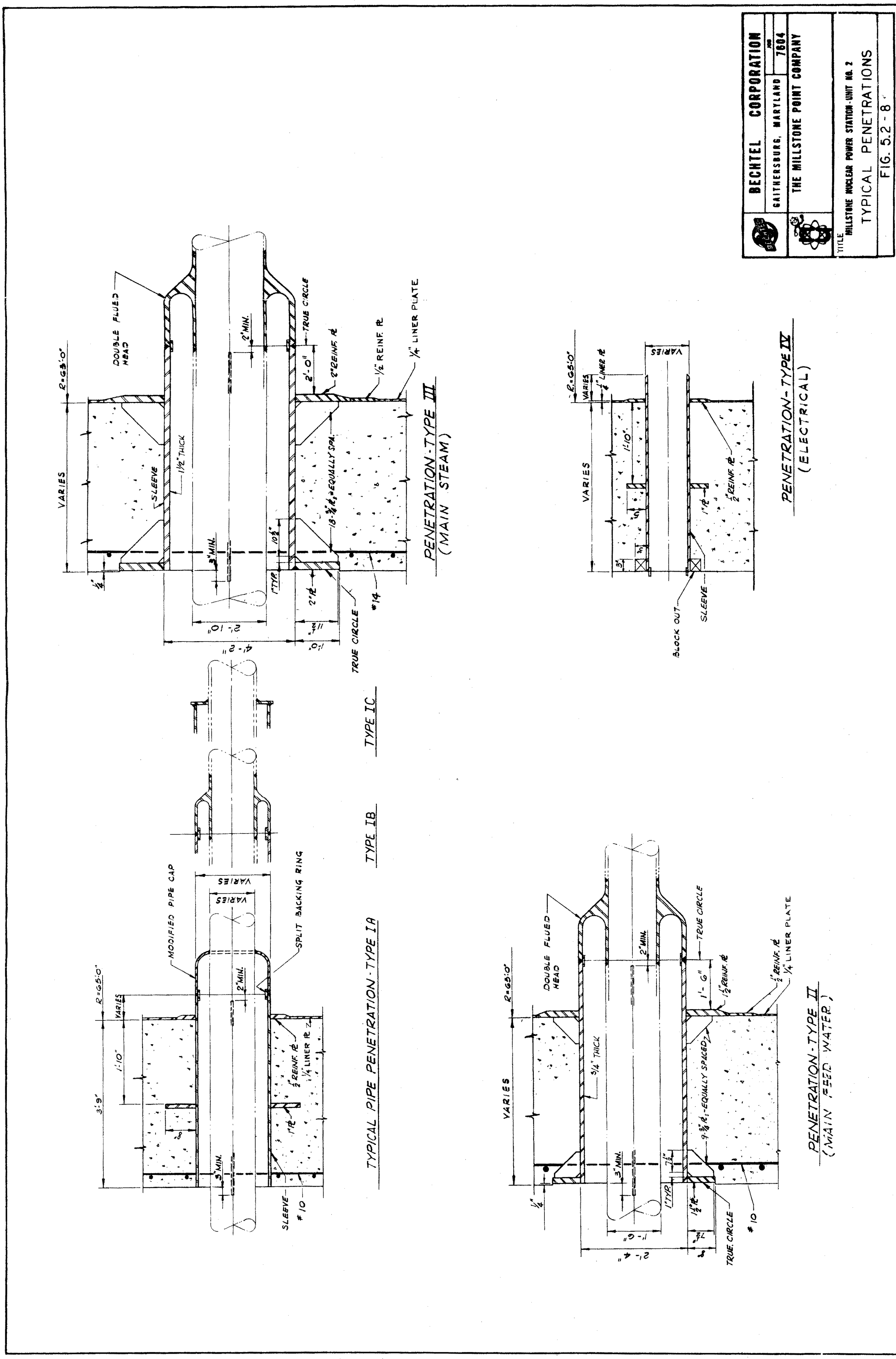


FIGURE 5.2–9 BRACKET DETAILS

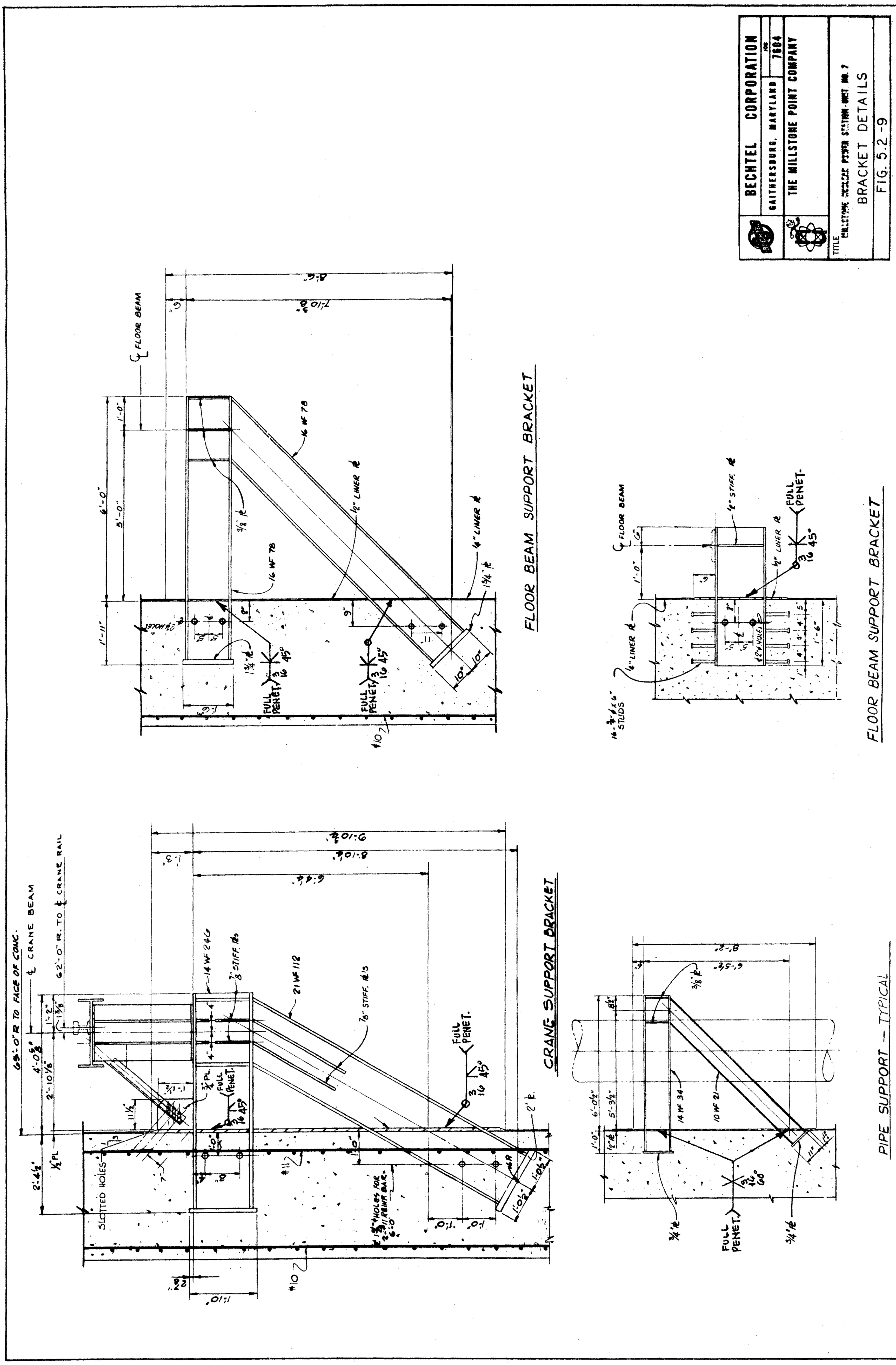
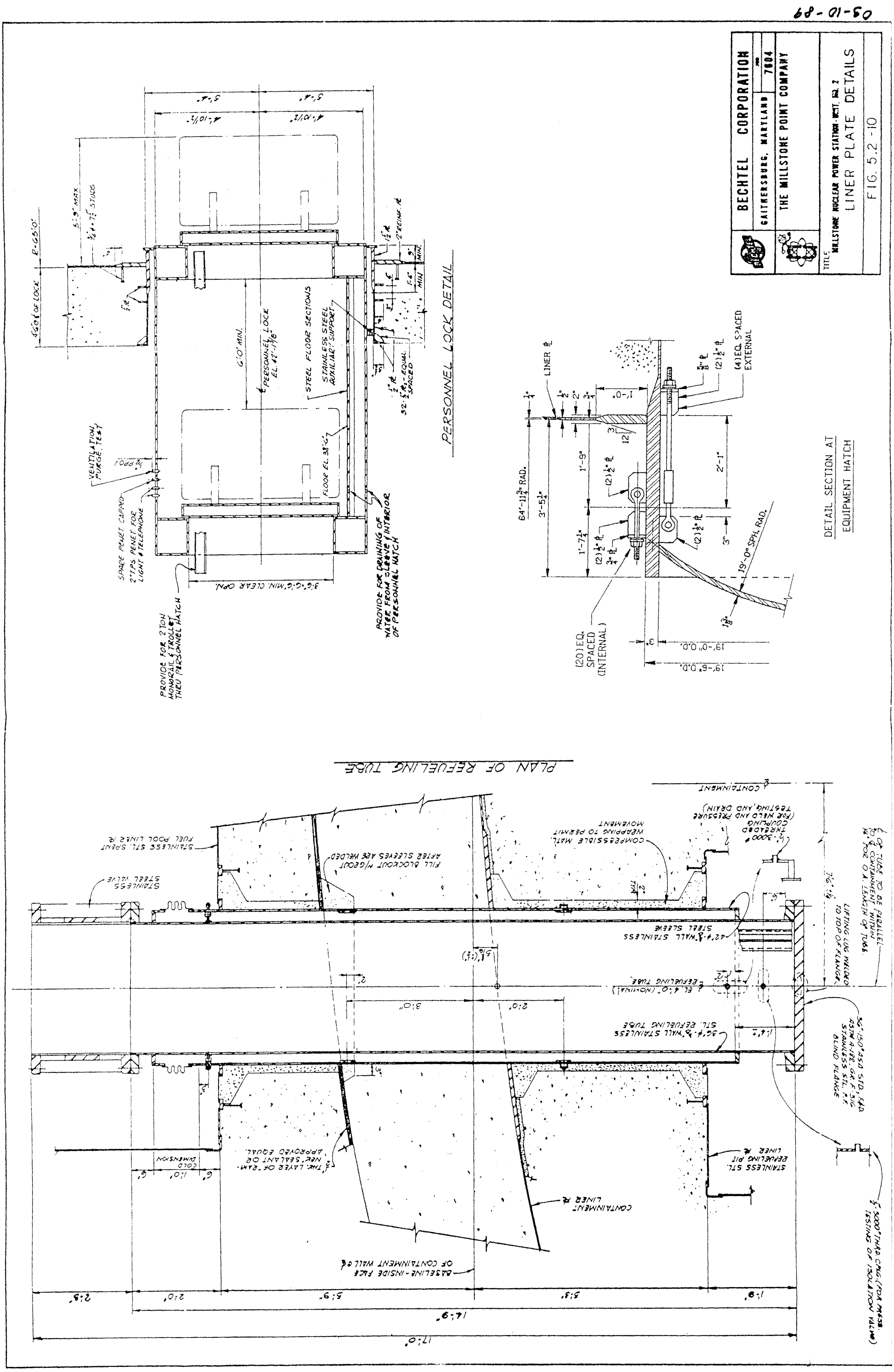


FIGURE 5.2–10 LINER PLATE DETAILS



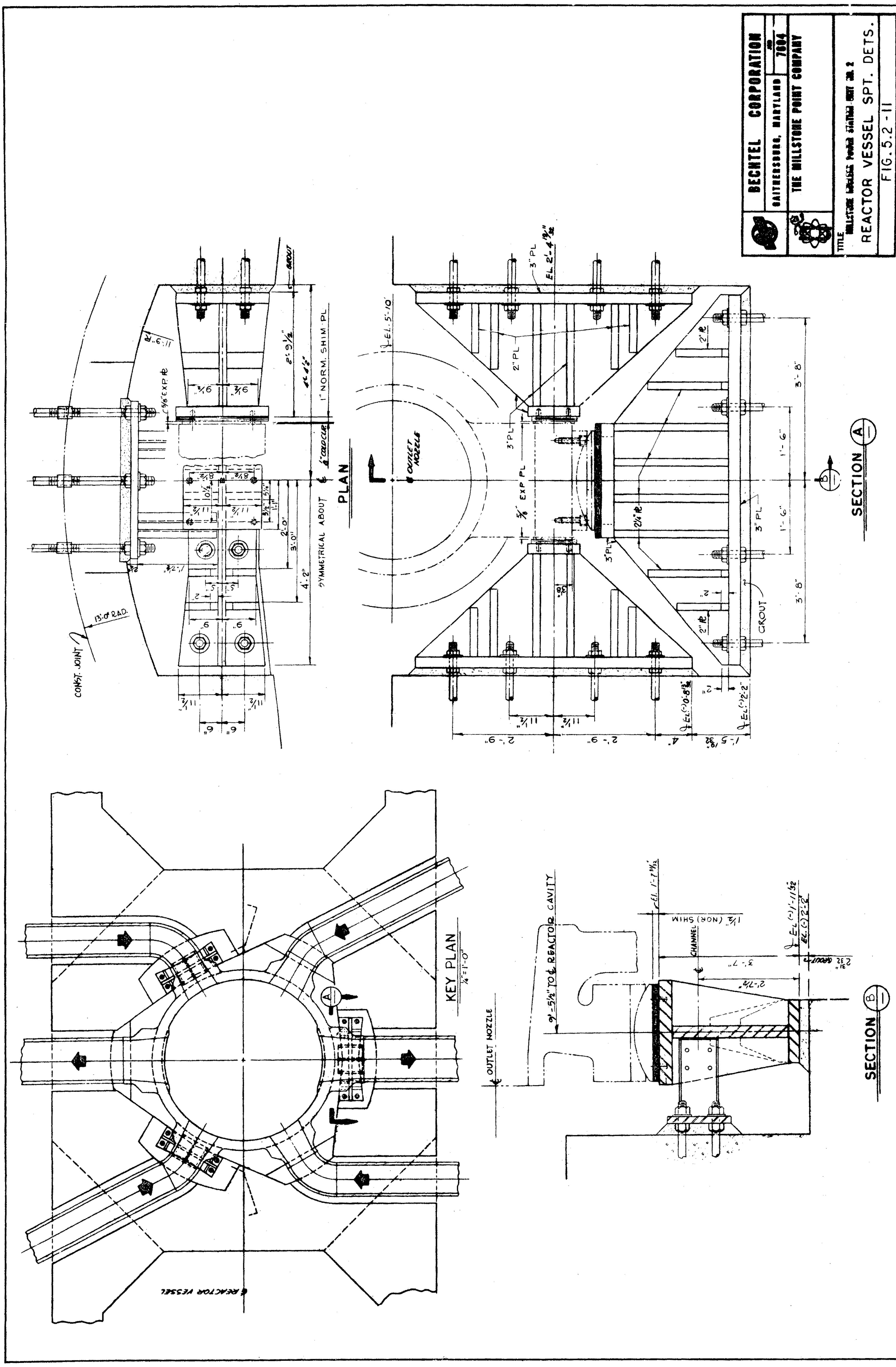


FIGURE 5.2–11 REACTOR VESSEL SUPPORT DETAILS

FIGURE 5.2–12 LOWER STEAM GENERATOR SUPPORT DETAILS

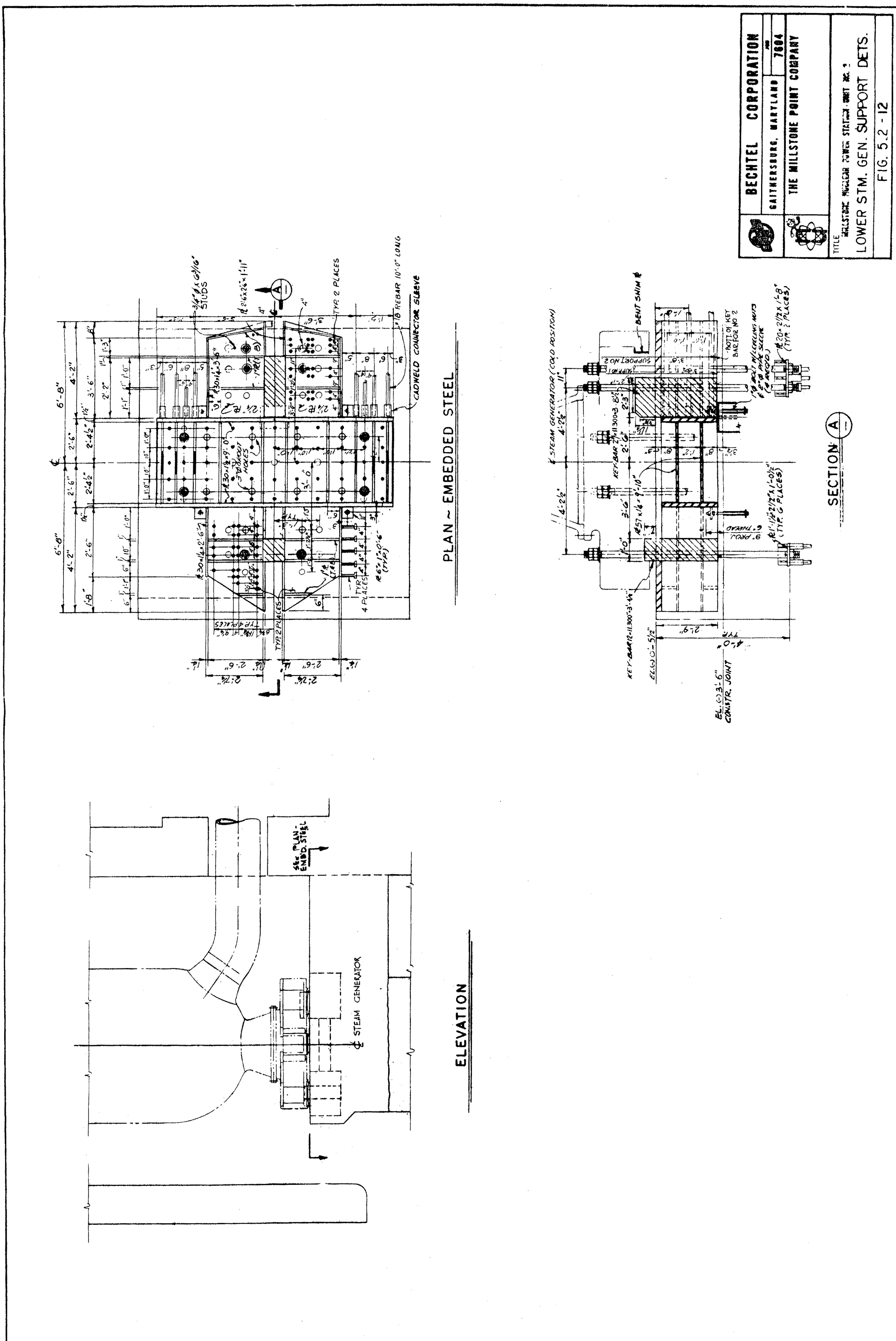
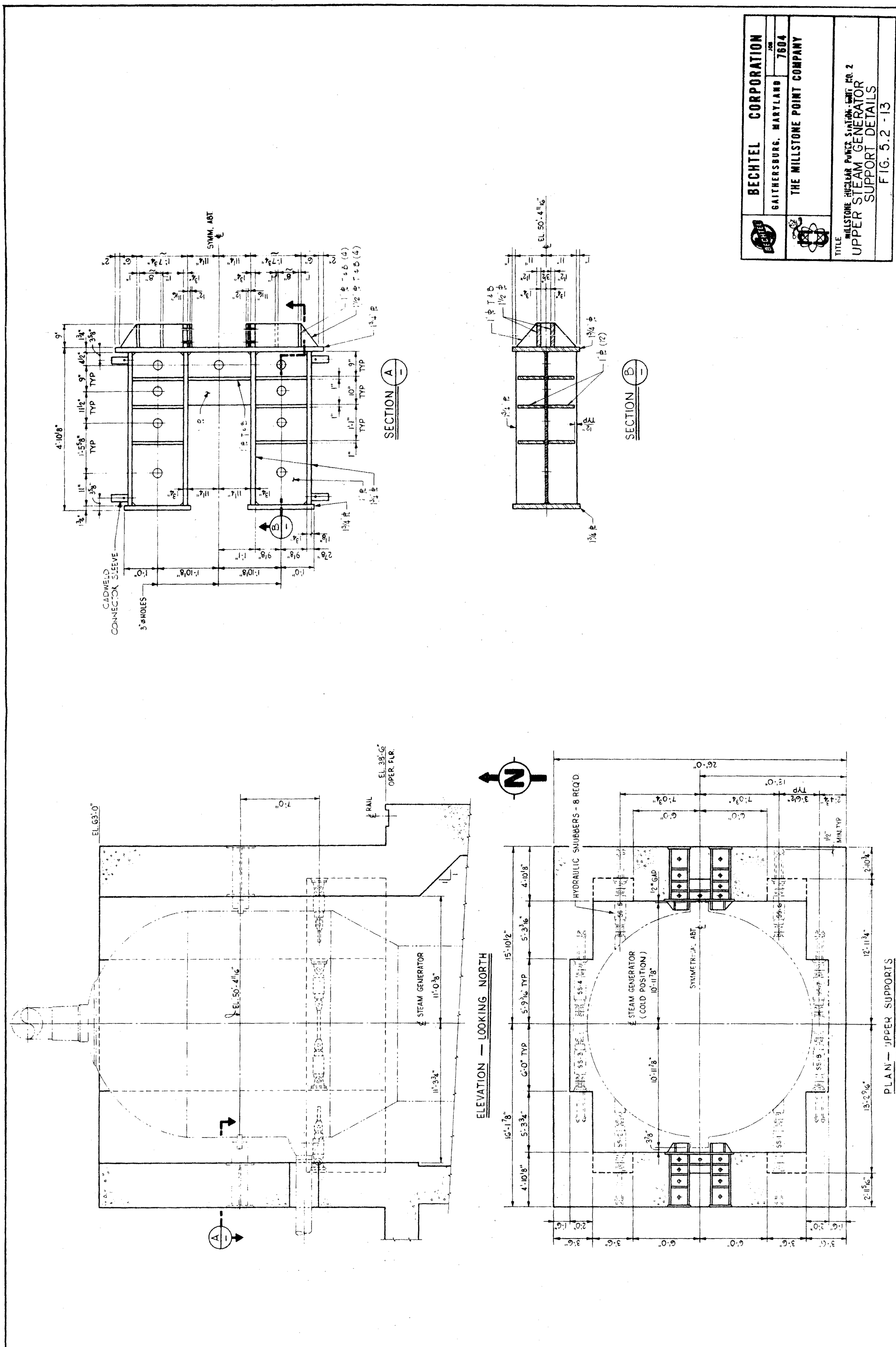


FIGURE 5.2–13 UPPER STEAM GENERATOR SUPPORT DETAILS



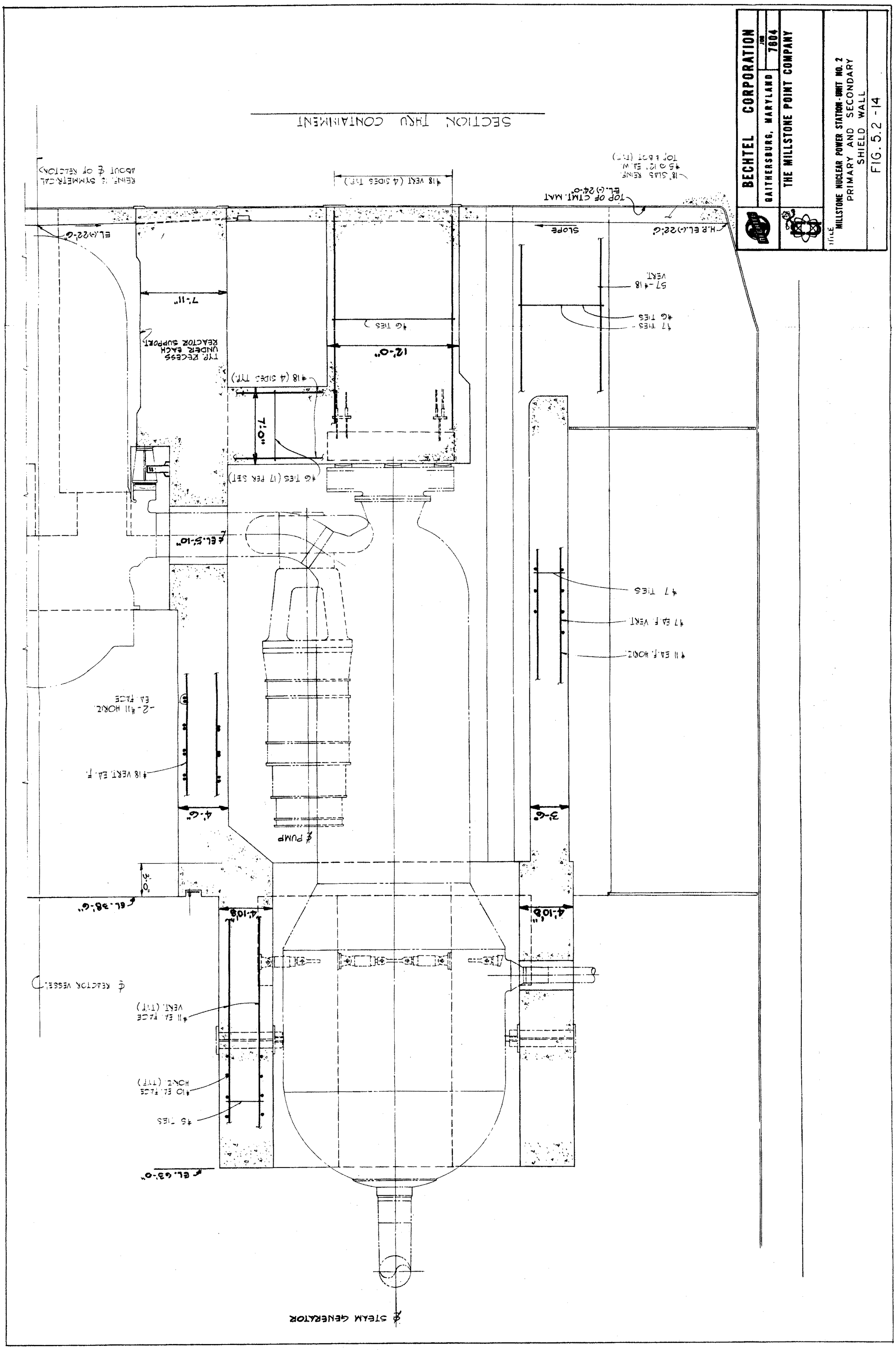
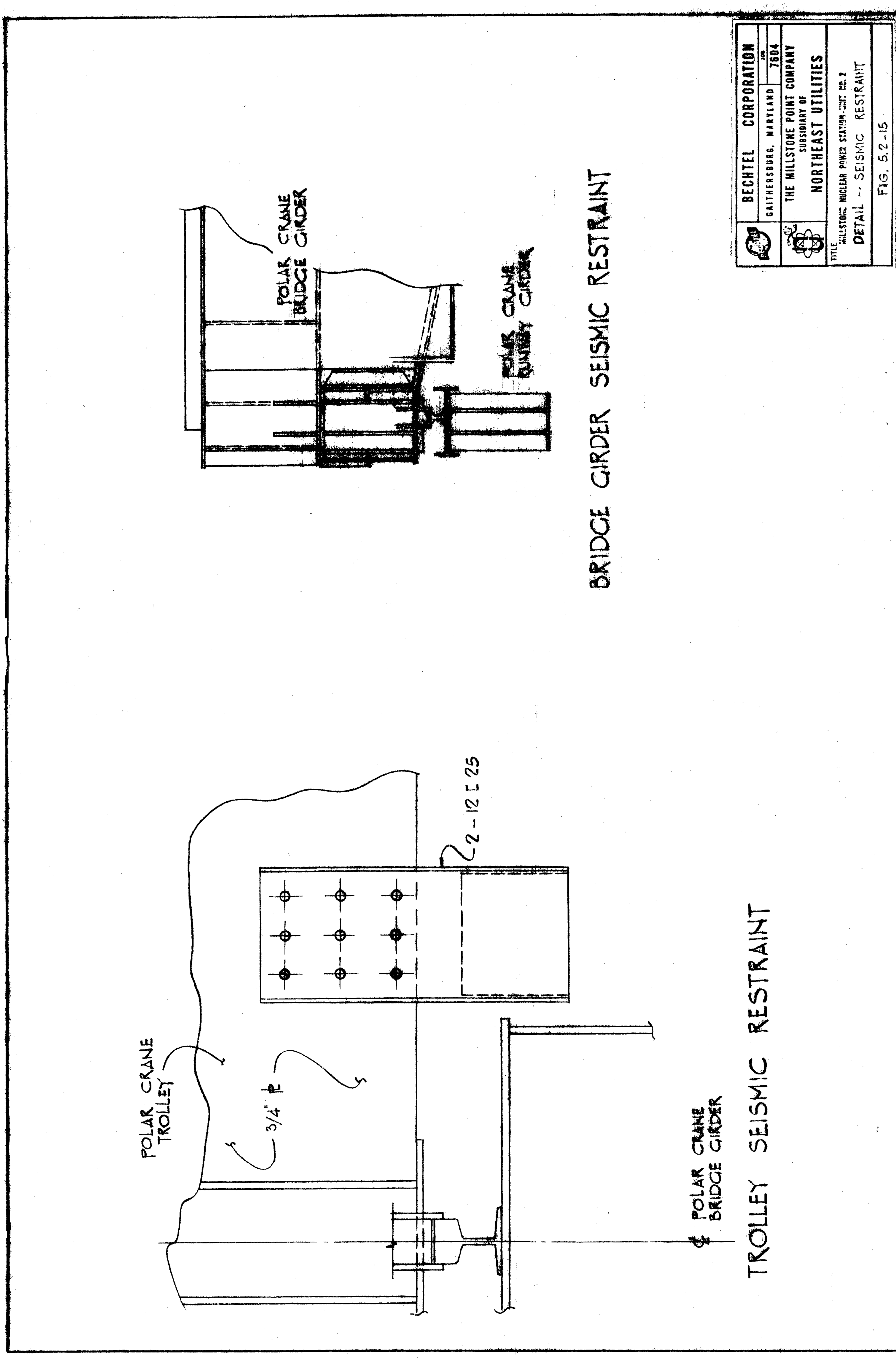


FIGURE 5.2–14 PRIMARY AND SECONDARY SHIELD WALL

FIGURE 5.2–15 DETAIL - SEISMIC RESTRAINT



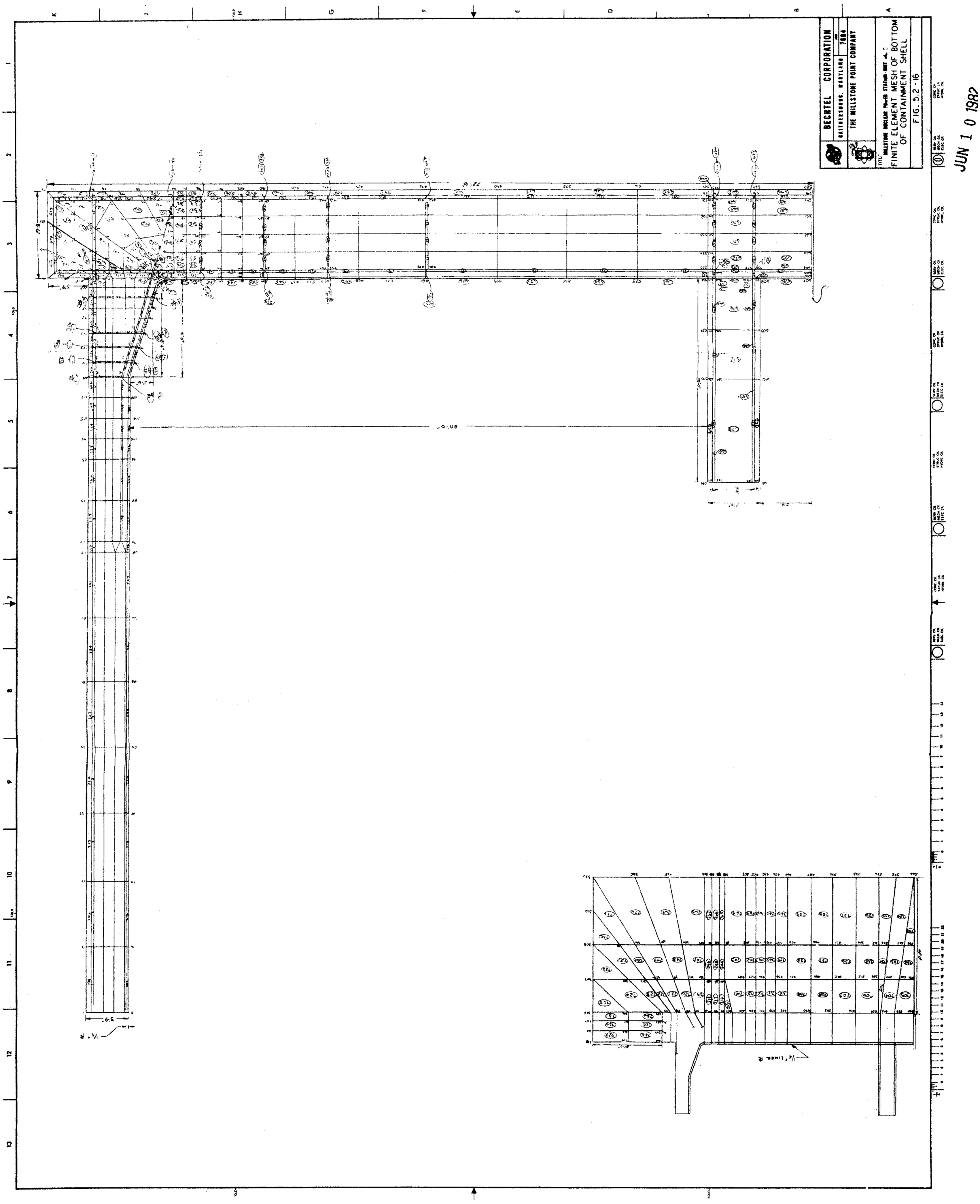


FIGURE 5.2–16 FINITE ELEMENT MESH OF BOTTOM OF CONTAINMENT SHELL

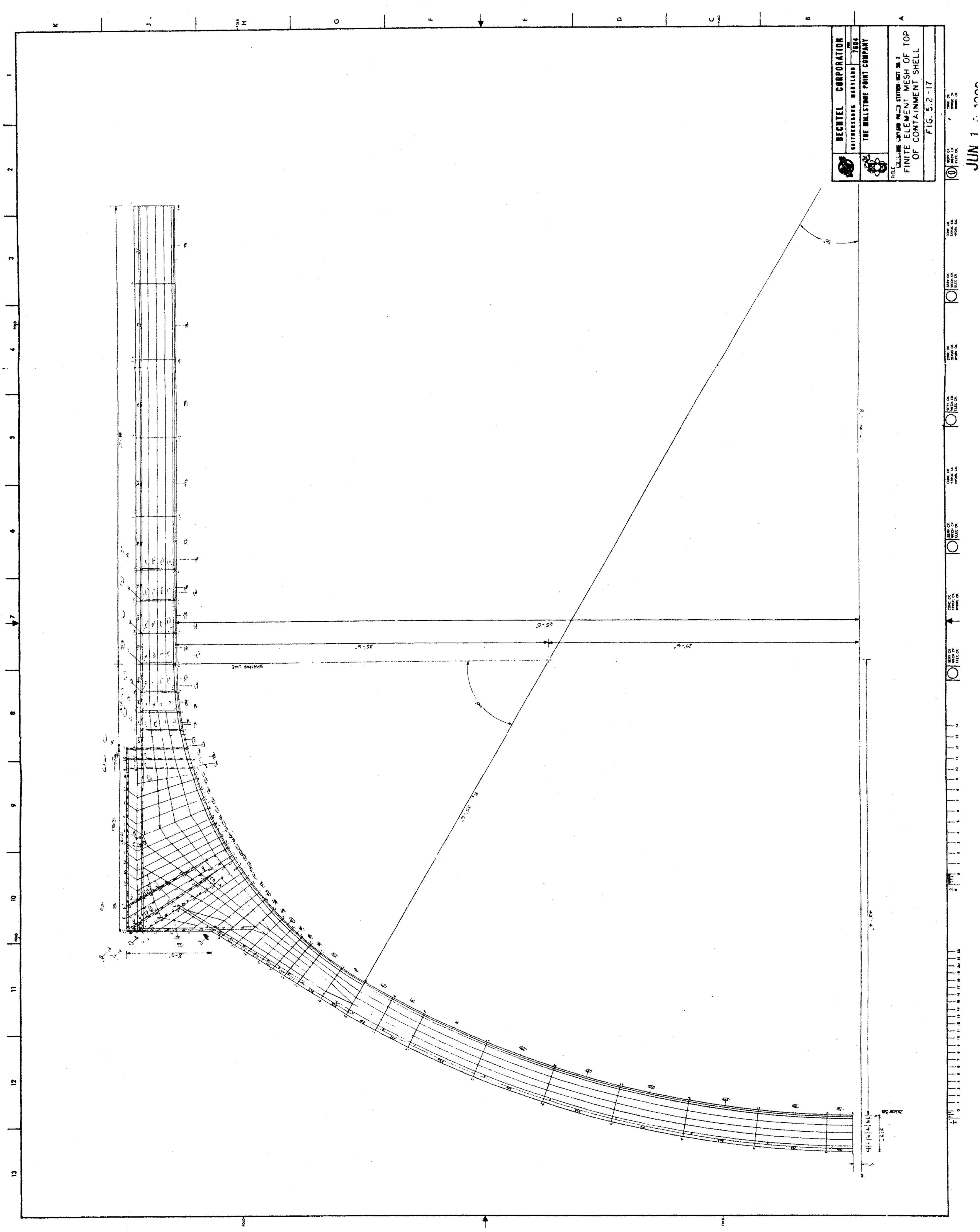


FIGURE 5.2–17 FINITE ELEMENT MESH OF TOP OF CONTAINMENT SHELL

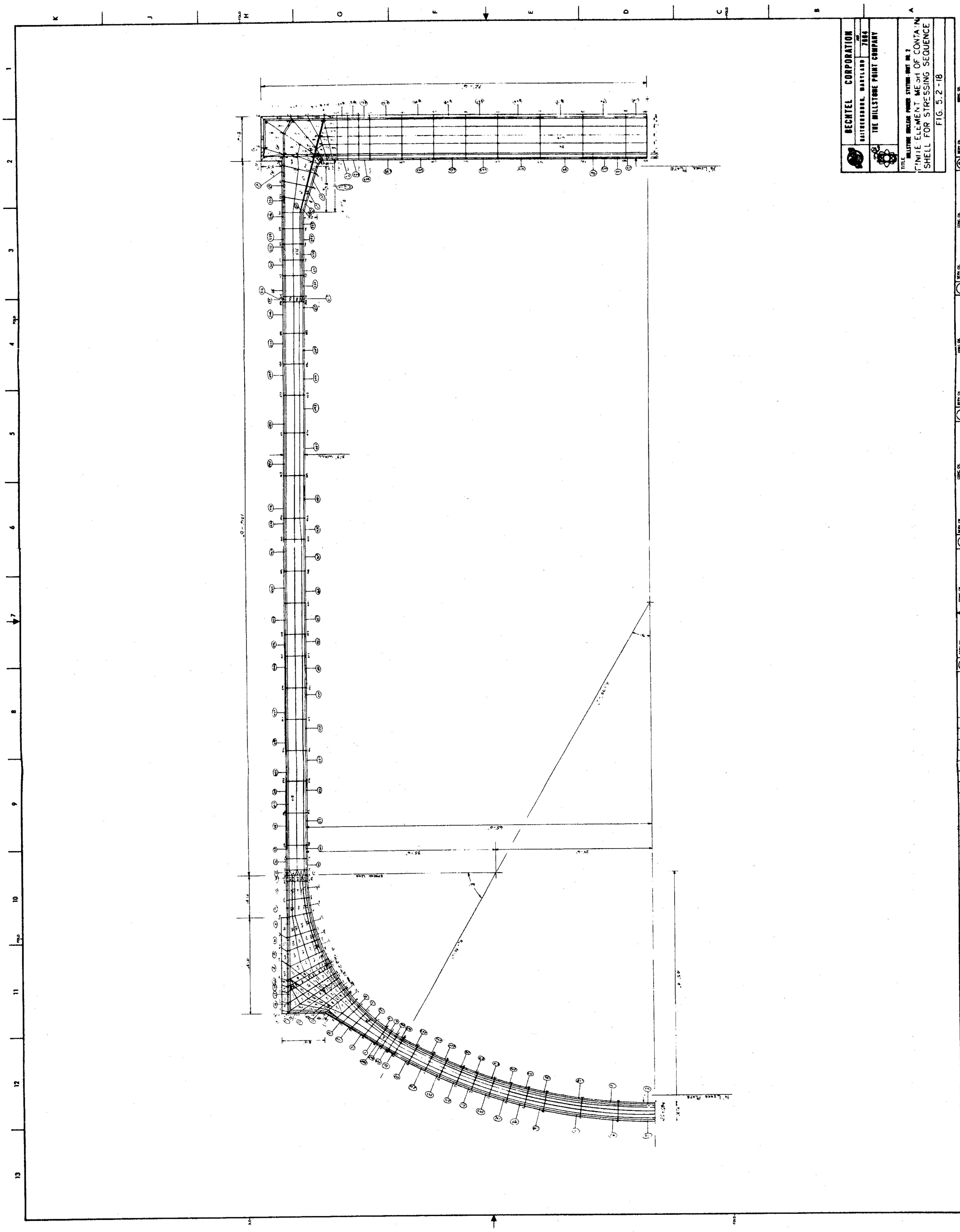


FIGURE 5.2–18 FINITE ELEMENT MESH OF CONTAINMENT SHELL FOR STRESSING SEQUENCE

FIGURE 5.2–19 CONTAINMENT STRUCTURE STRESS ANALYSIS SUMMARY, DEAD LOAD AND INITIAL PRESTRESS, LIVE LOAD

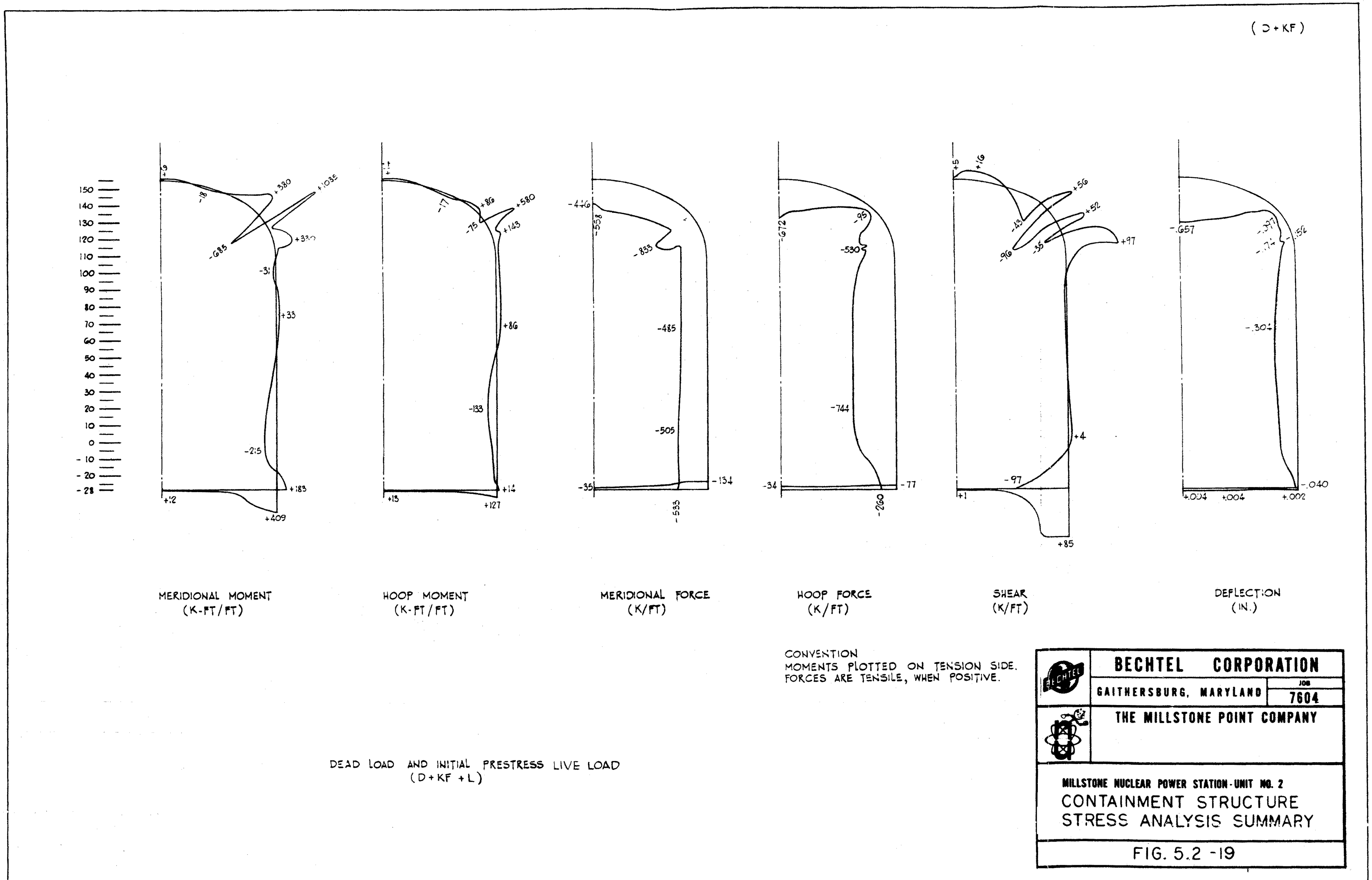


FIGURE 5.2–20 CONTAINMENT STRUCTURE STRESS ANALYSIS SUMMARY, DEAD LOAD, LIVE LOAD, PRESTRESS AND TEST PRESSURE

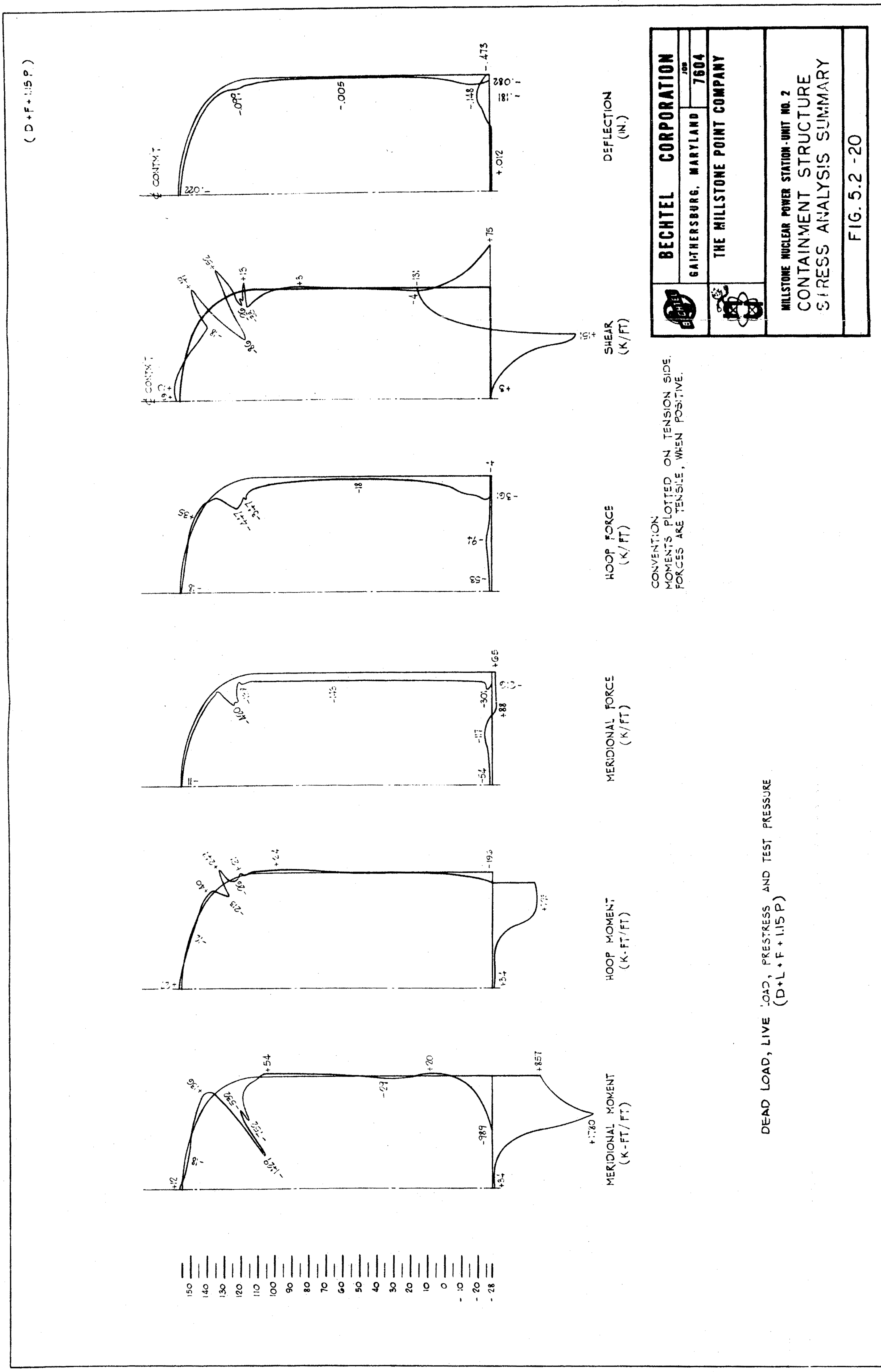
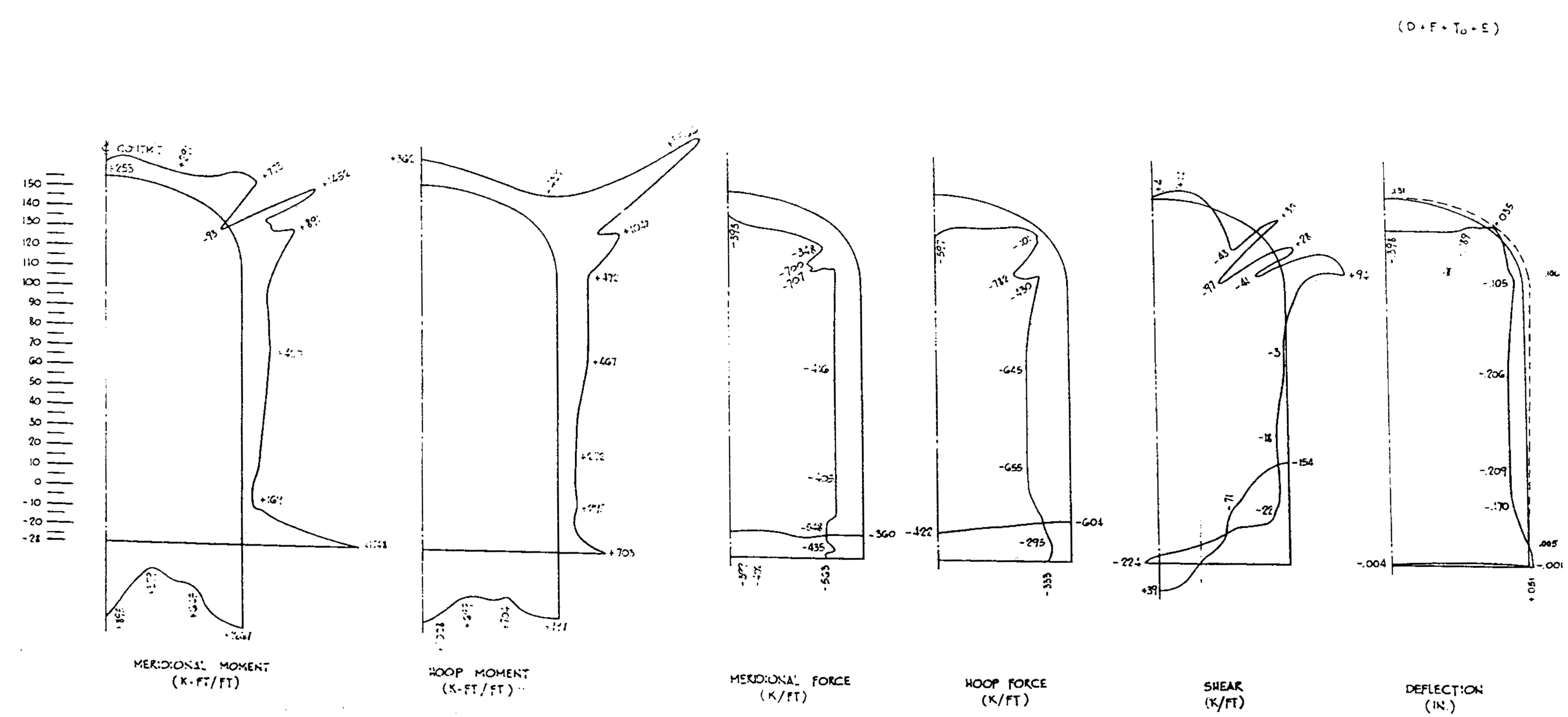


FIGURE 5.2–21 CONTAINMENT STRUCTURE STRESS ANALYSIS SUMMARY, DEAD LOAD, LIVE LOAD, PRESTRESS, OPERATING TEMPERATURE AND DBE

MNPS-2 FSAR



DEAD LOAD, LIVE LOAD, PRESTRESS, OPERATING TEMPERATURE AND OBE
(D + L + F + To + E)

CONVENTION
MOMENTS PLOTTED ON TENSION SIDE.
FORCES ARE TENSILE, WHEN POSITIVE.

BECHTEL CORPORATION
GAITHERSBURG, MARYLAND 7604
THE MILLSTONE POINT COMPANY

NOTE:
Changes resulting from the revised seismic analysis performed in 1998-99 have not been incorporated in this Figure, but the controlling stresses/strains resulting from the revision are shown in Table 5.2-4.

FIGURE 5.2-21
CONTAINMENT STRUCTURE STRESS ANALYSIS SUMMARY

MARCH 1999

FIGURE 5.2–22 CONTAINMENT STRUCTURE STRESS ANALYSIS SUMMARY, DEAD LOAD, LIVE LOAD, PRESTRESS, 100% ACCIDENT PRESSURE AND ACCIDENT TEMPERATURE

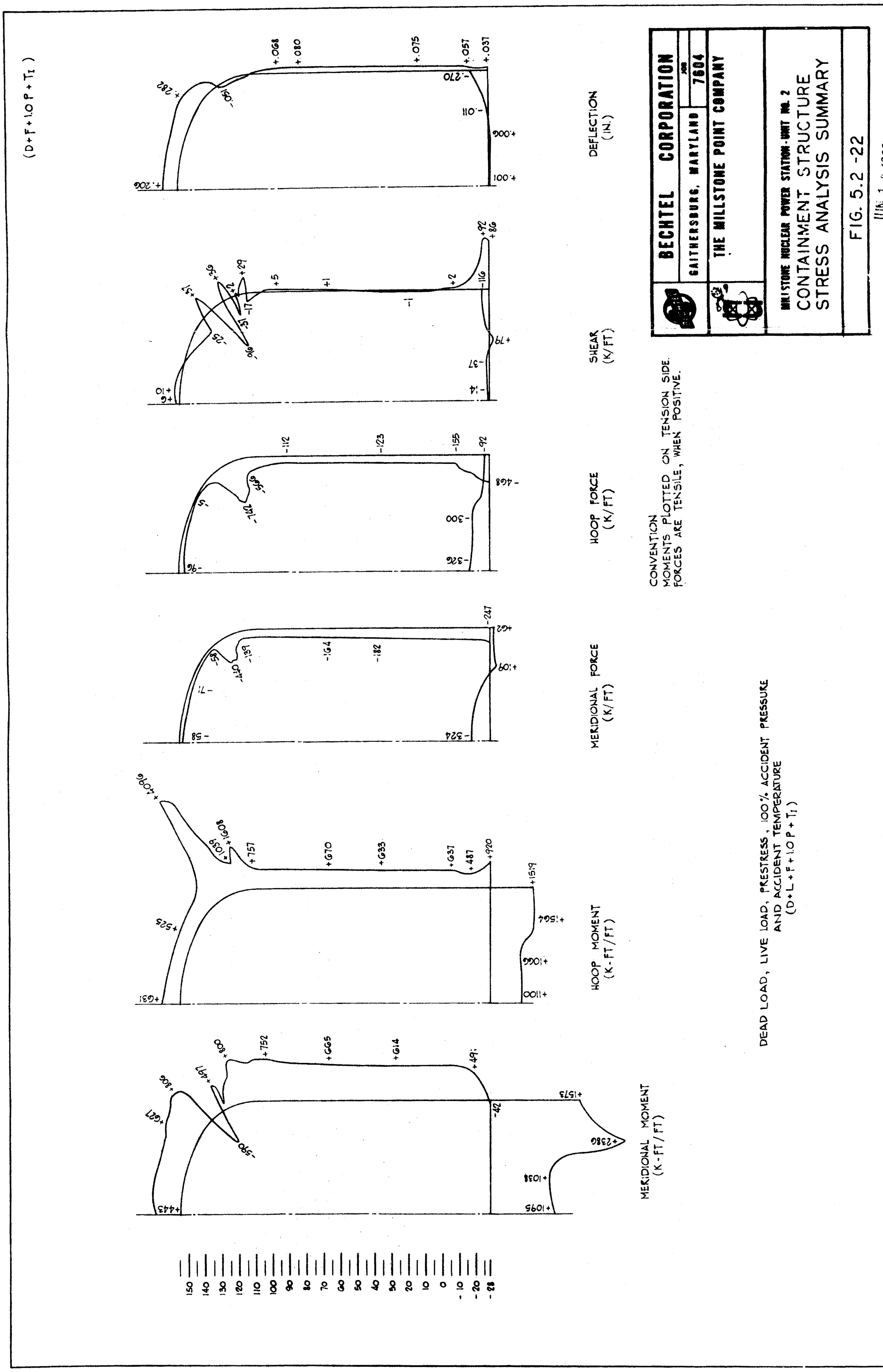
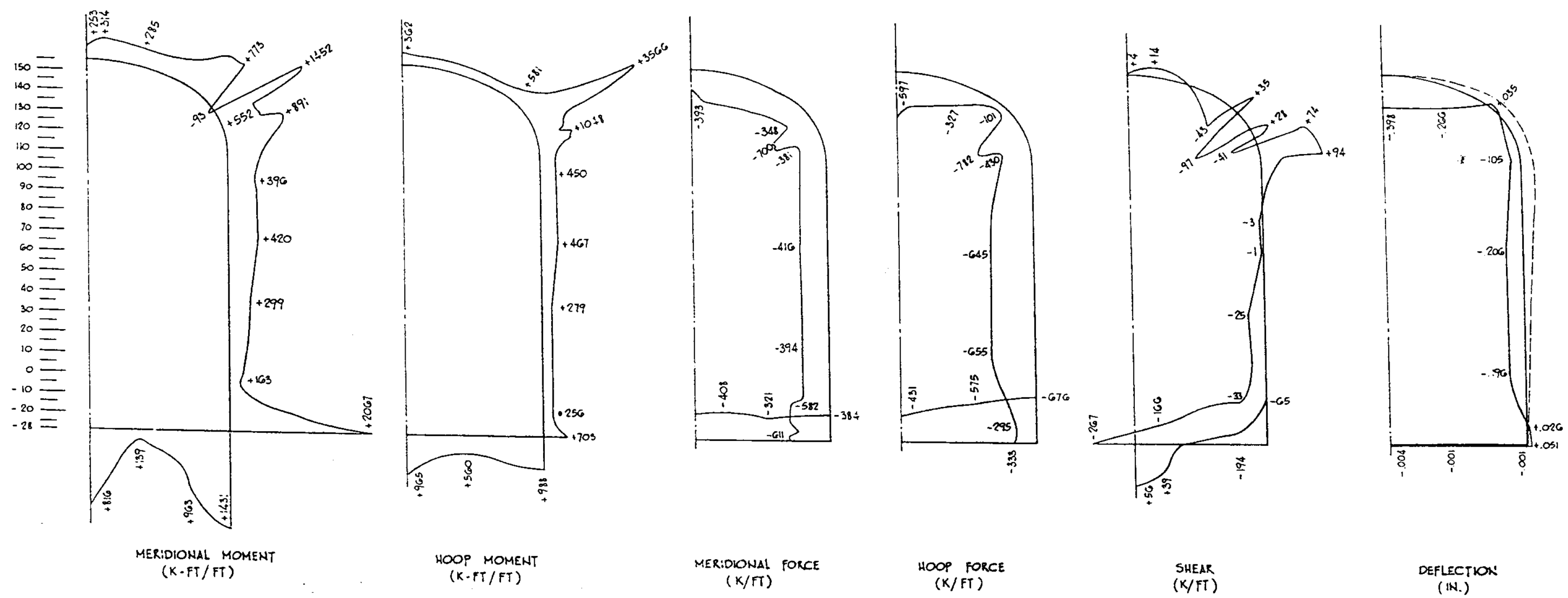


FIGURE 5.2–23 CONTAINMENT STRUCTURE STRESS ANALYSIS SUMMARY, DEAD LOAD, PRESTRESS, OPERATING TEMPERATURE, THERMAL EXPANSION FORCES OF PIPES, PIPE RUPTURE FORCES AND DBE



DEAD LOAD, PRESTRESS, OPERATING TEMPERATURE, THERMAL EXPANSION FORCES OF PIPES, PIPE RUPTURE FORCES AND DBE.
(D + F + T_O + H + R + E')

CONVENTION
MOMENTS PLOTTED ON TENSION SIDE.
FORCES ARE TENSILE, WHEN POSITIVE.

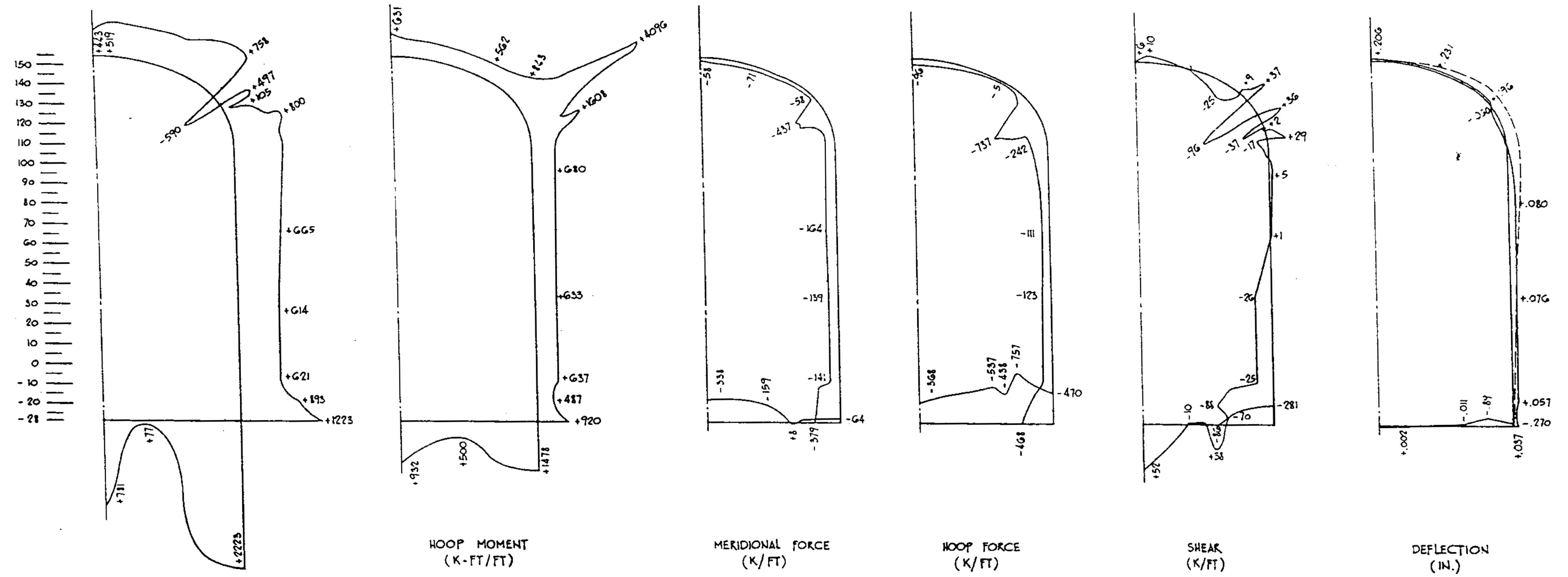
BECHTEL CORPORATION
GAITHERSBURG, MARYLAND 7604
THE MILLSTONE POINT COMPANY

NOTE:
Changes resulting from the revised seismic analysis performed in 1998-99 have not been incorporated in this Figure, but the controlling stresses/strains resulting from the revision are shown in Table 5.2-6.

FIGURE 5.2-23
CONTAINMENT STRUCTURE STRESS ANALYSIS SUMMARY

MARCH 1999

FIGURE 5.2–24 CONTAINMENT STRUCTURE STRESS ANALYSIS SUMMARY, DEAD LOAD, PRESTRESS, 100% ACCIDENT PRESSURE, THERMAL EXPANSION FORCES OF PIPES, ACCIDENT TEMPERATURE AND DBE



MERIDIONAL MOMENT (K-FT/FT)

HOOP MOMENT (K-FT/FT)

MERIDIONAL FORCE (K/FT)

HOOP FORCE (K/FT)

SHEAR (K/FT)

DEFLECTION (IN.)

CONVENTION
MOMENTS PLOTTED ON TENSION SIDE.
FORCES ARE TENSILE, WHEN POSITIVE.

DEAD LOAD, PRESTRESS, 100 % ACCIDENT PRESSURE, THERMAL EXPANSION FORCES OF PIPES, ACCIDENT TEMPERATURE AND DBE.
(D + F + 1.0 P + H + T_1 + E')

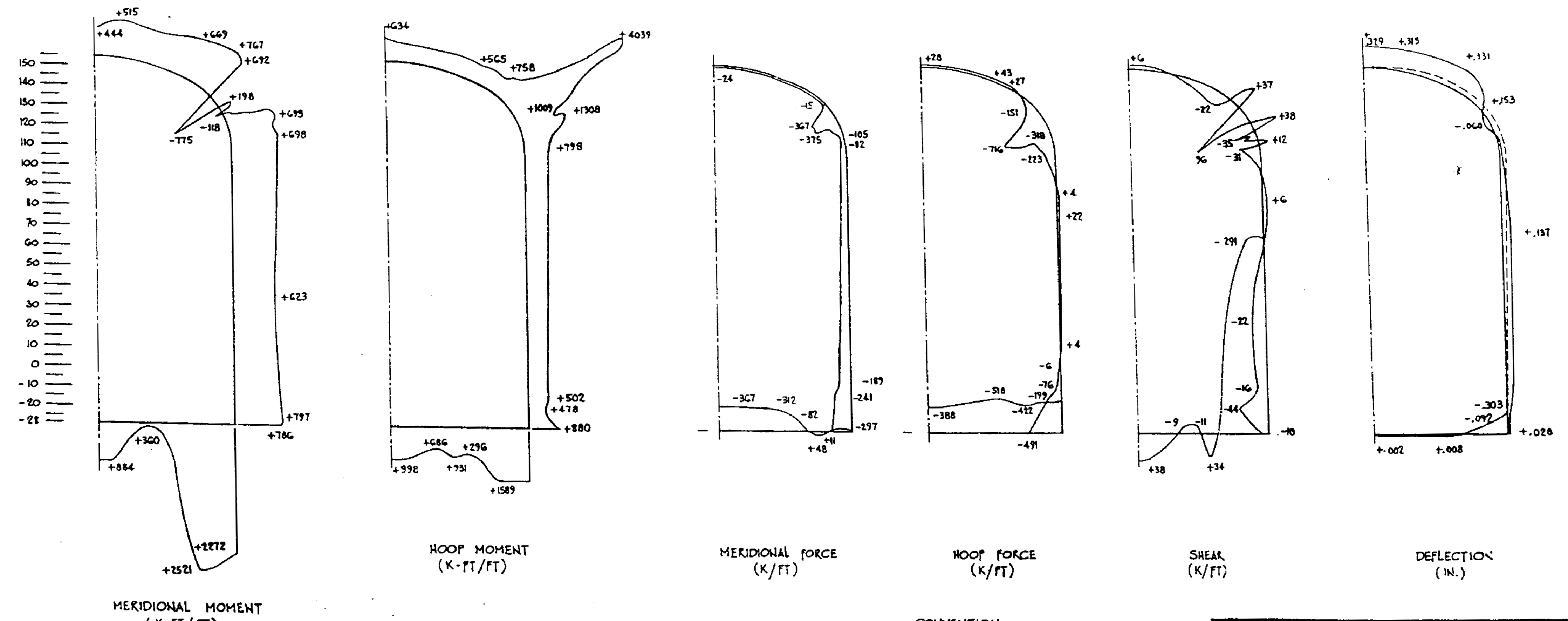
BECHTEL CORPORATION	
GAITHERSBURG, MARYLAND	JOB 7604
THE MILLSTONE POINT COMPANY	

NOTE:
Changes resulting from the revised seismic analysis performed in 1998-99 have not been incorporated in this Figure, but the controlling stresses/strains resulting from the revision are shown in Table 5.2-7.

FIGURE 5.2-24
CONTAINMENT STRUCTURE STRESS ANALYSIS SUMMARY

MARCH 1999

FIGURE 5.2–25 CONTAINMENT STRUCTURE STRESS ANALYSIS SUMMARY, DEAD LOAD, PRESTRESS, 125% ACCIDENT PRESSURE, 125% THERMAL EXPANSION FORCES OF PIPES, ACCIDENT TEMPERATURE AND 125% OBE



DEAD LOAD, PRESTRESS, 125 % ACCIDENT PRESSURE, 125 % THERMAL EXPANSION FORCES OF PIPES, ACCIDENT TEMPERATURE AND 125 % OBE
(D + F + 1.25 P + 1.25 H + T_t + 1.25 E)

CONVENTION
MOMENTS PLOTTED ON TENSION SIDE.
FORCES ARE TENSILE, WHEN POSITIVE.

BECHTEL CORPORATION
GAITHERSBURG, MARYLAND
JOB 7604
THE MILLSTONE POINT COMPANY

NOTE:
Changes resulting from the revised seismic analysis performed in 1998-99 have not been incorporated in this Figure, but the controlling stresses/strains resulting from the revision are shown in Table 5.2-8.

FIGURE 5.2-25
CONTAINMENT STRUCTURE STRESS ANALYSIS SUMMARY

MARCH 1999

FIGURE 5.2–26 CONTAINMENT AND STRUCTURE STRESS ANALYSIS SUMMARY, DEAD LOAD, PRESTRESS, 150% ACCIDENT PRESSURE AND ACCIDENT TEMPERATURE

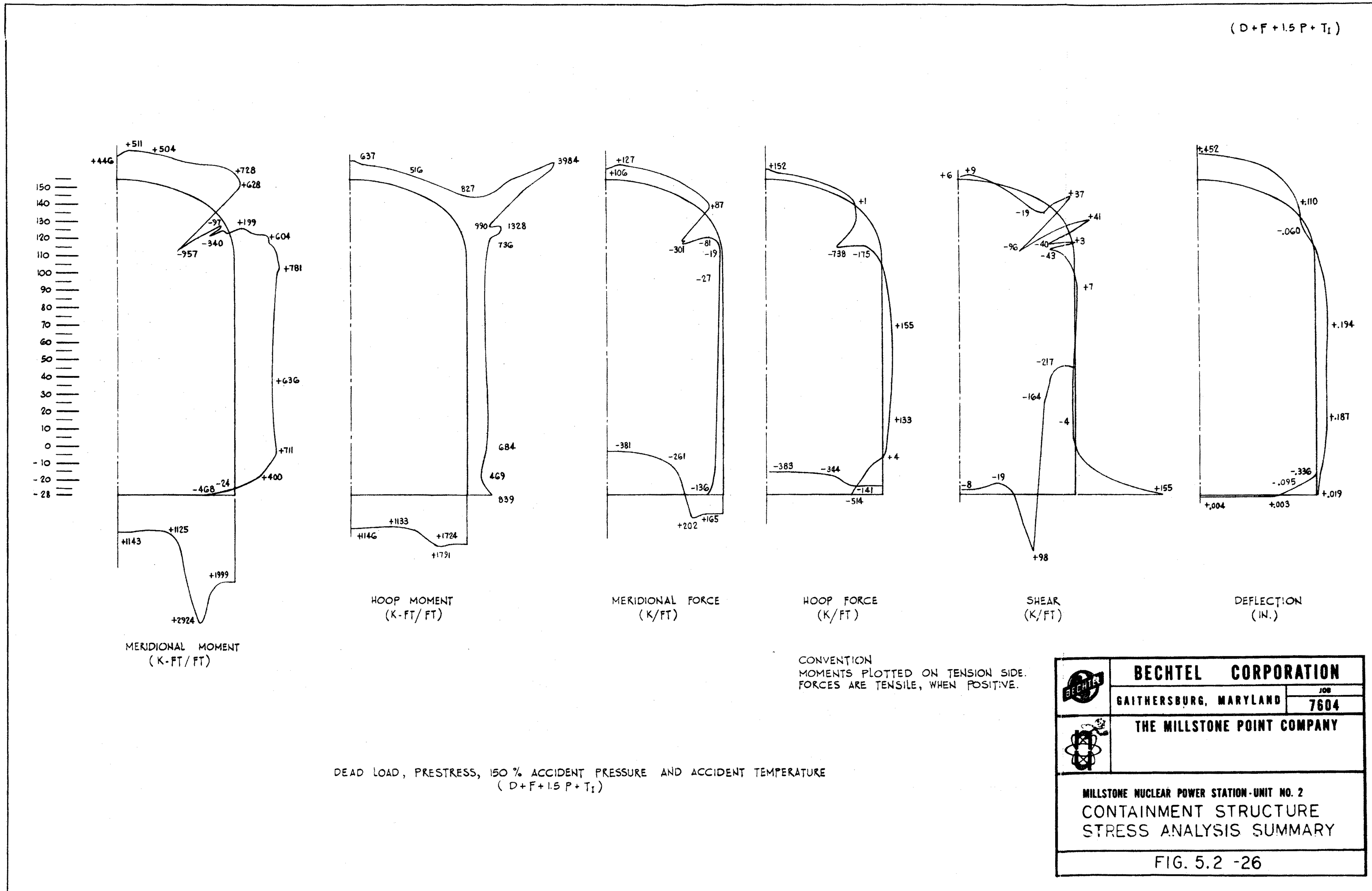


FIGURE 5.2–27 ISOLATION VALVE ARRANGEMENTS

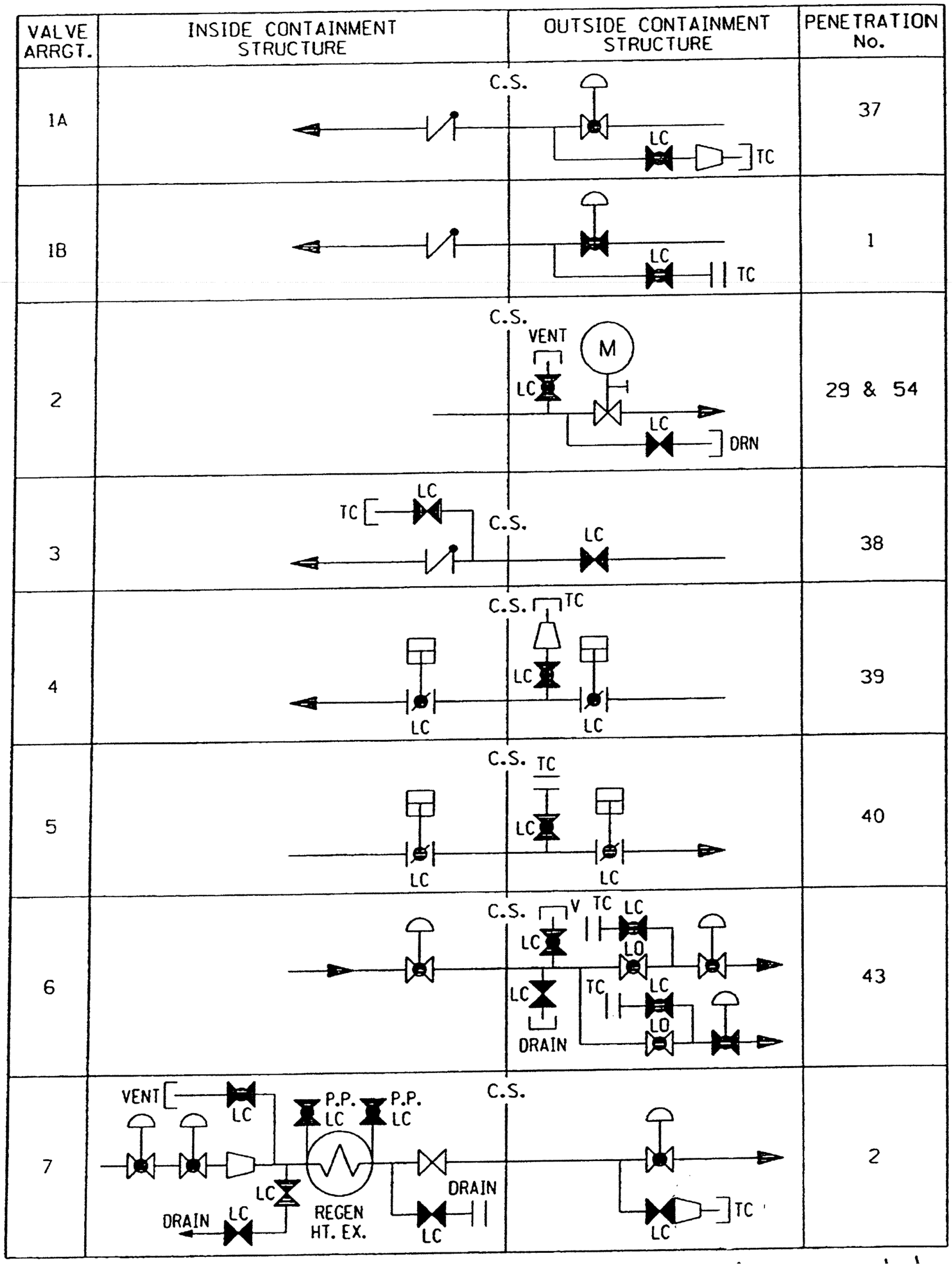
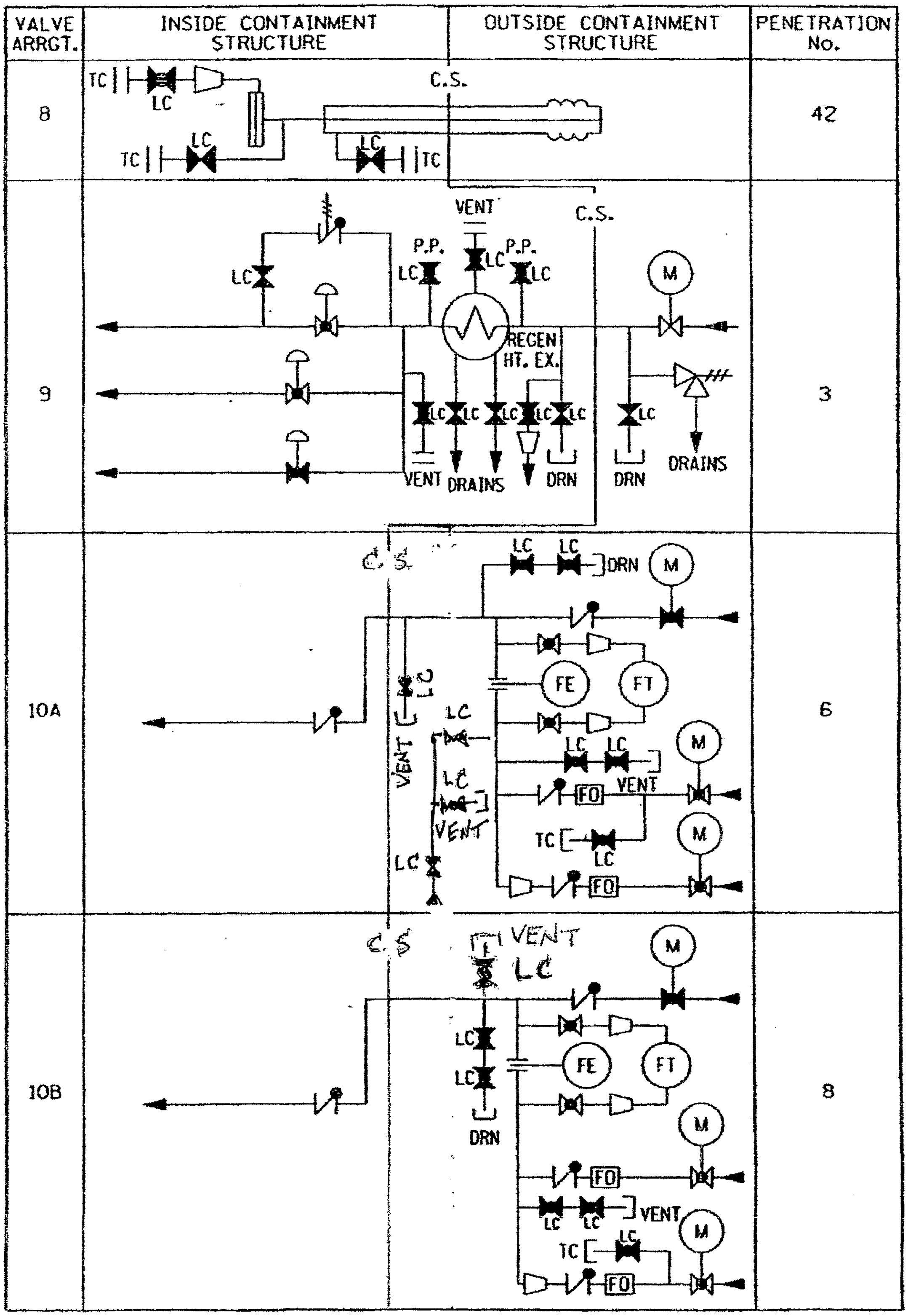
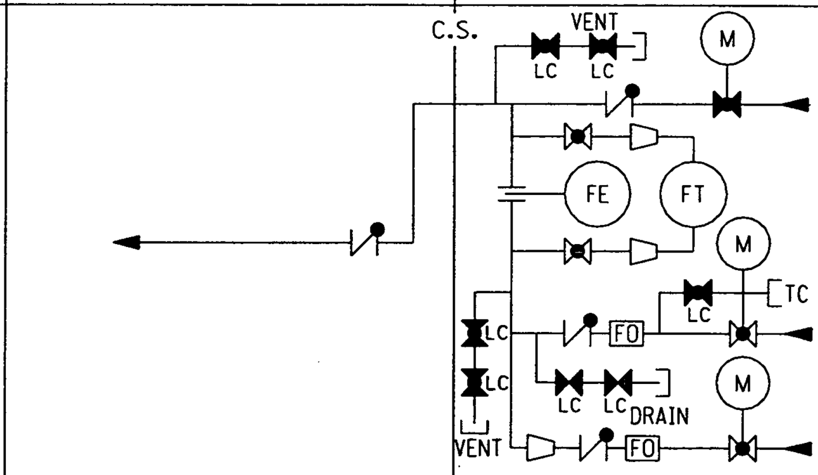
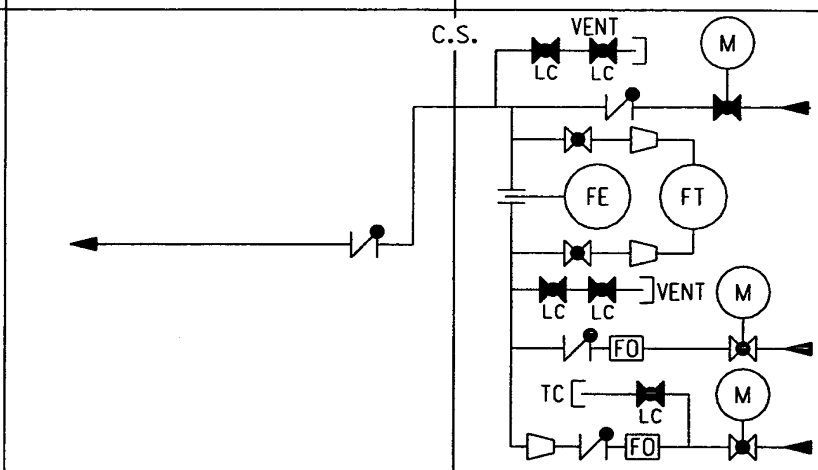
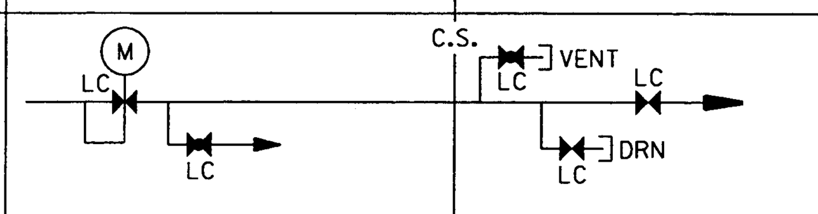
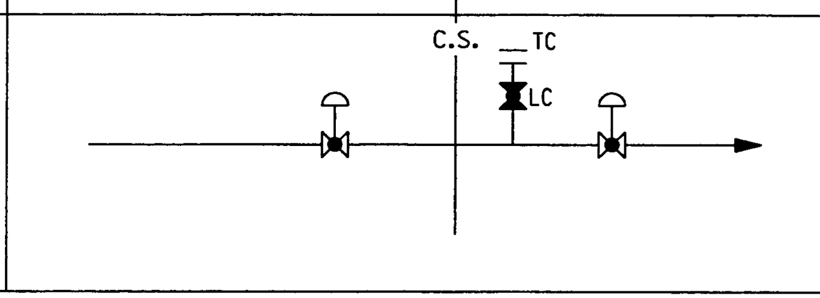


FIGURE 5.2–28 ISOLATION VALVE ARRANGEMENTS



* MISSILE WALL

FIGURE 5.2–29 ISOLATION VALVE ARRANGEMENTS

VALVE ARRGT.	INSIDE CONTAINMENT STRUCTURE / OUTSIDE CONTAINMENT STRUCTURE	PENETRATION No.
10C		7
10D		9
11		10
12		51

* MISSILE WALL

FIGURE 5.2–30 ISOLATION VALVE ARRANGEMENTS

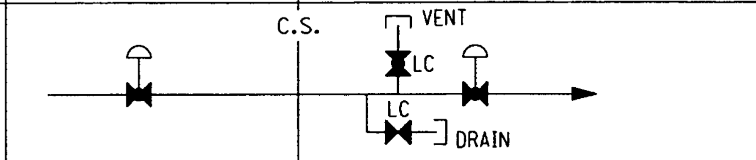
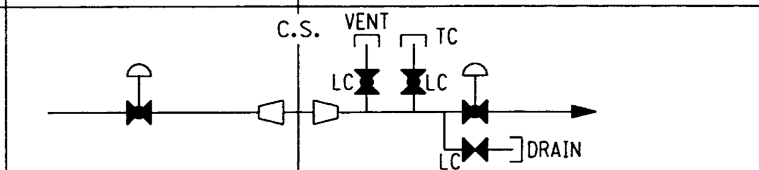
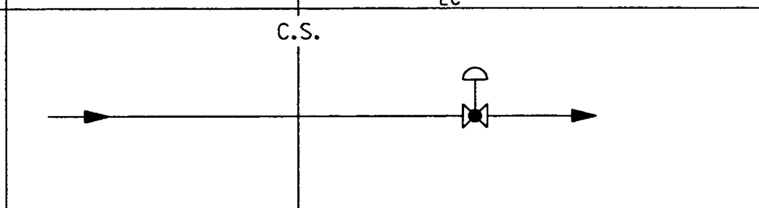
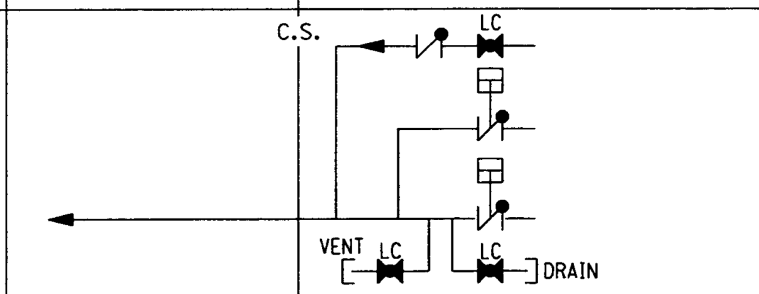
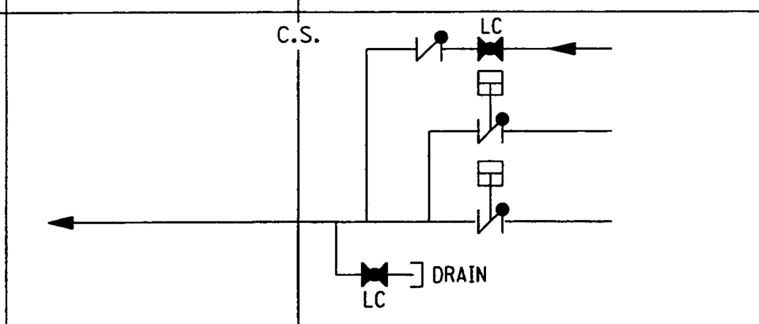
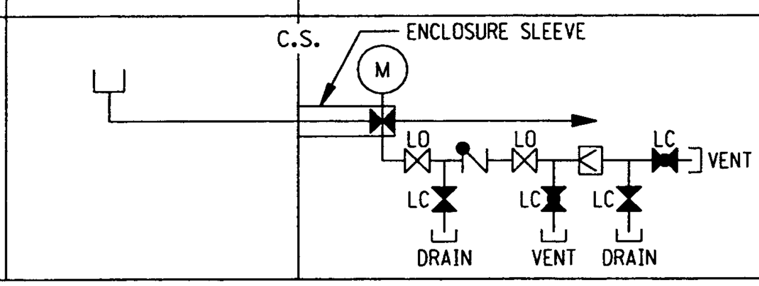
VALVE ARRGT.	INSIDE CONTAINMENT STRUCTURE	OUTSIDE CONTAINMENT STRUCTURE	PENETRATION No.
13A			14
13B			35
14			22, 23, 65 & 72
15A			15
15B			16
16			12 & 13

FIGURE 5.2–31 ISOLATION VALVE ARRANGEMENTS

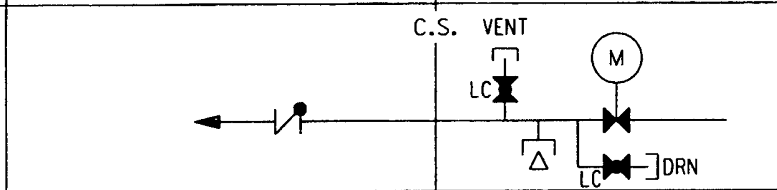
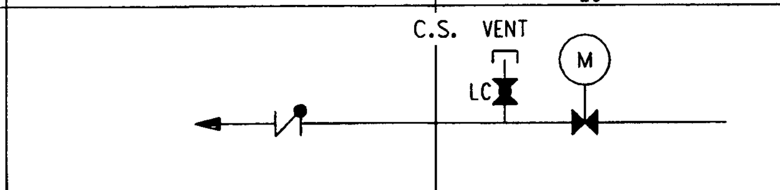
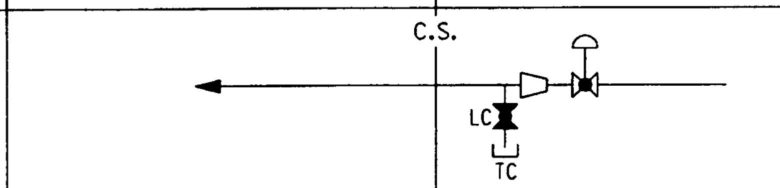
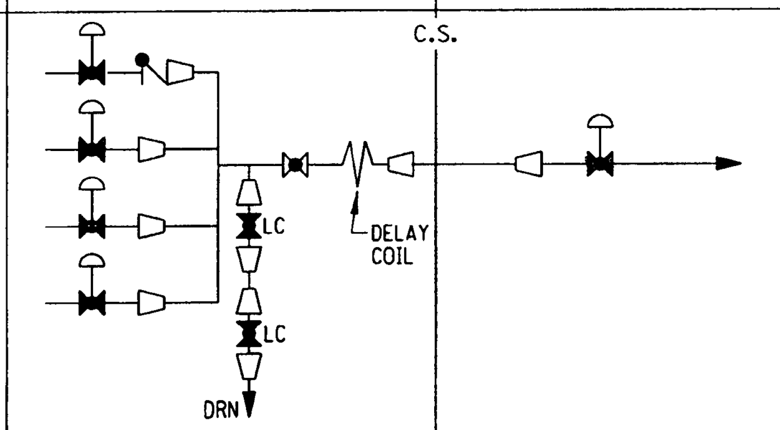
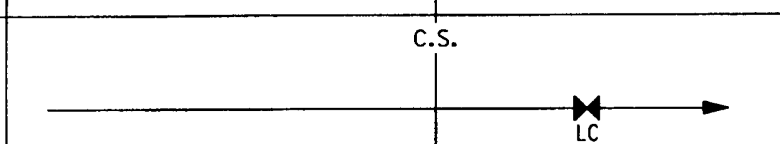
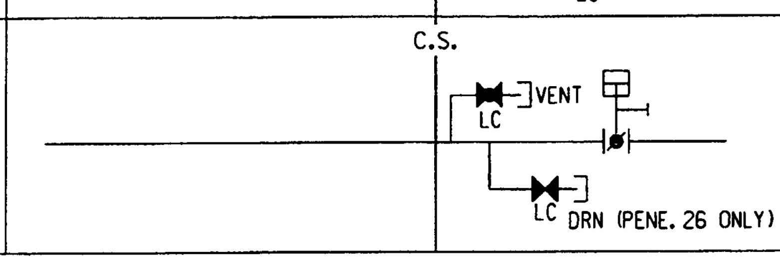
VALVE ARRGT.	INSIDE CONTAINMENT STRUCTURE	OUTSIDE CONTAINMENT STRUCTURE	PENETRATION No.
17A			4
17B			5
18			34
19			21
20			11
21A			25 & 26

FIGURE 5.2–32 ISOLATION VALVE ARRANGEMENTS

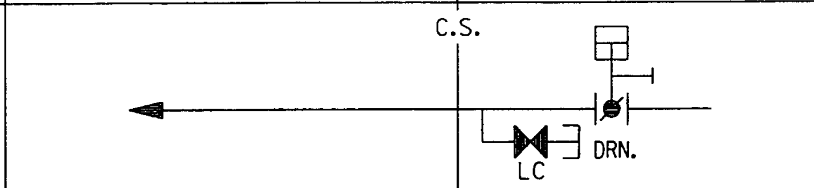
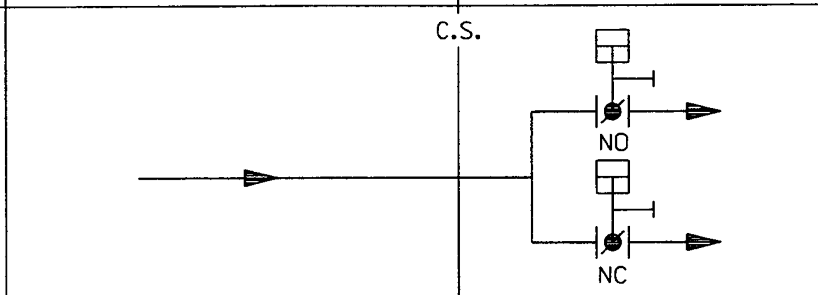
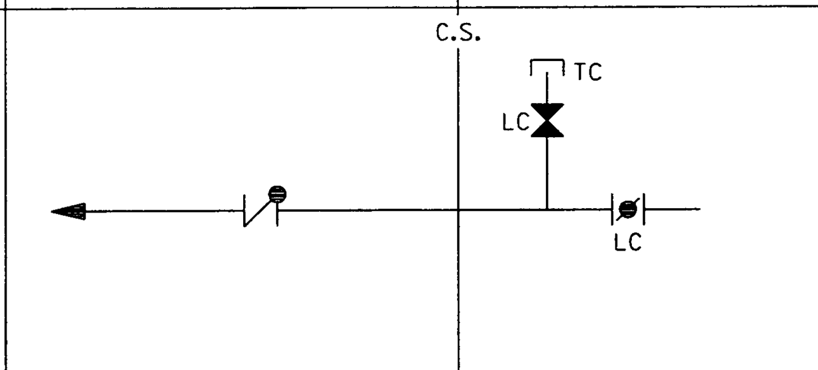
VALVE ARRGT.	INSIDE CONTAINMENT STRUCTURE	OUTSIDE CONTAINMENT STRUCTURE	PENETRATION No.
21B			27, 28
22			30, 31 32, 33
34			49

FIGURE 5.2–33 ISOLATION VALVE ARRANGEMENTS

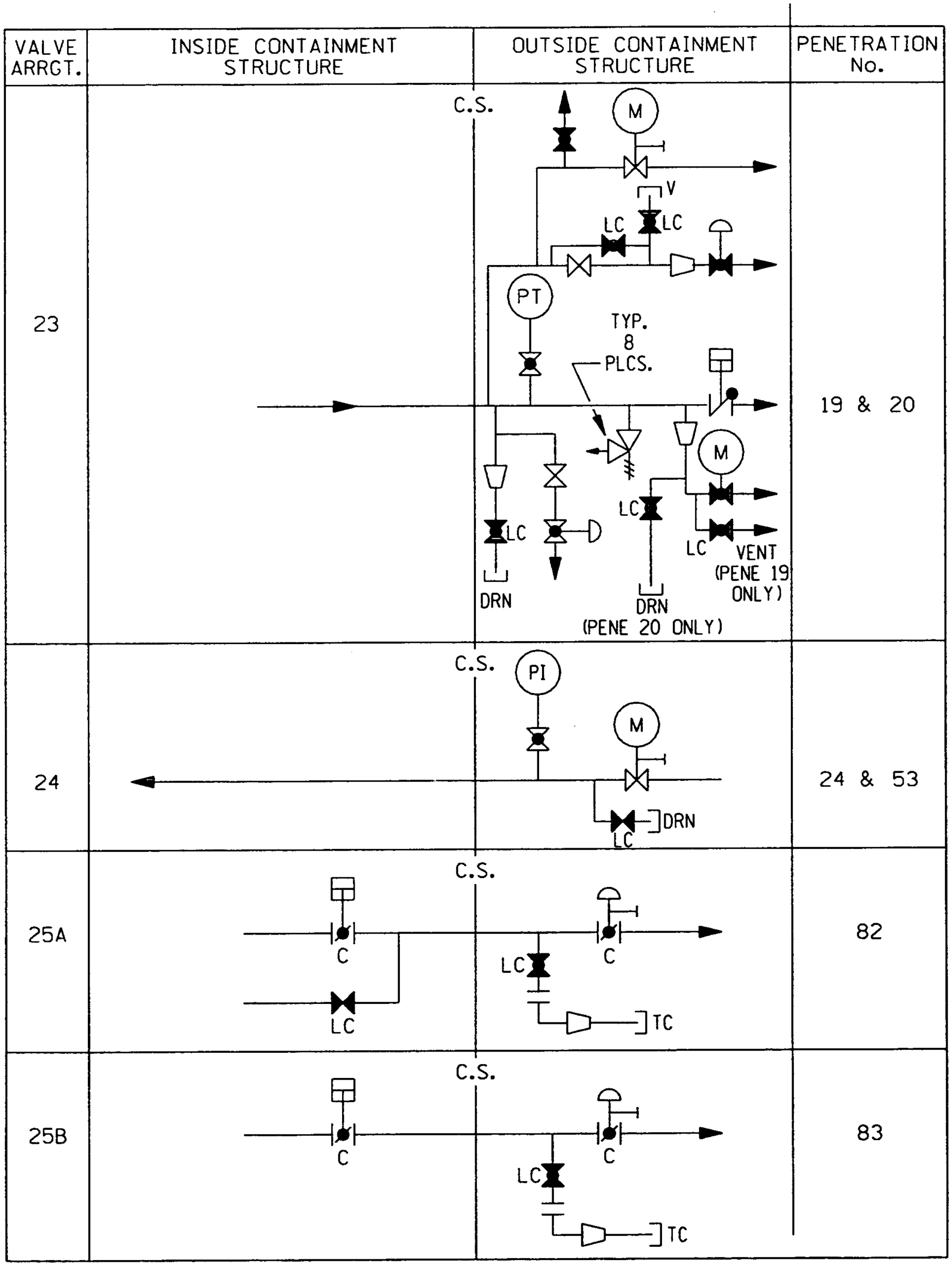


FIGURE 5.2–34 ISOLATION VALVE ARRANGEMENTS

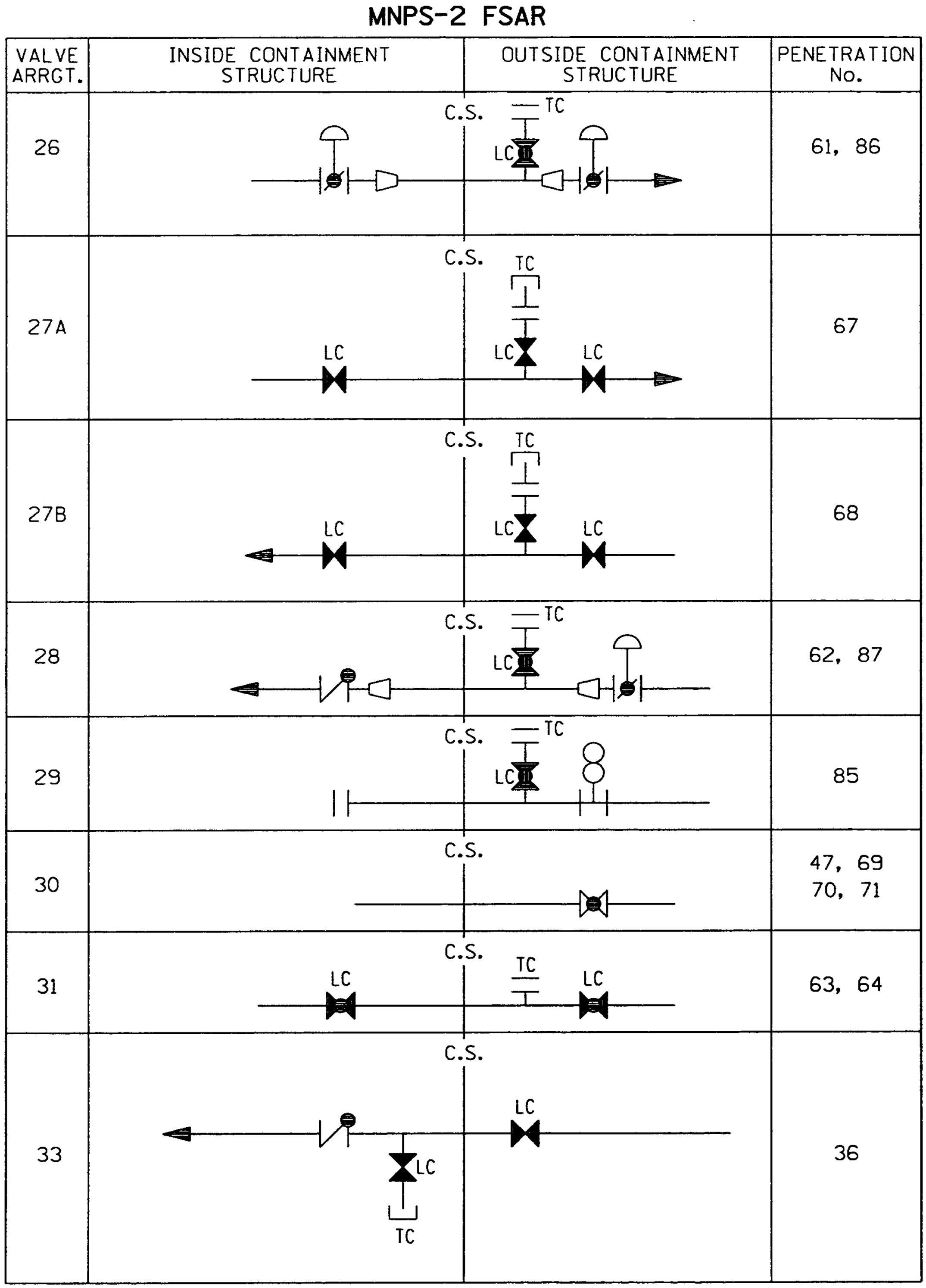
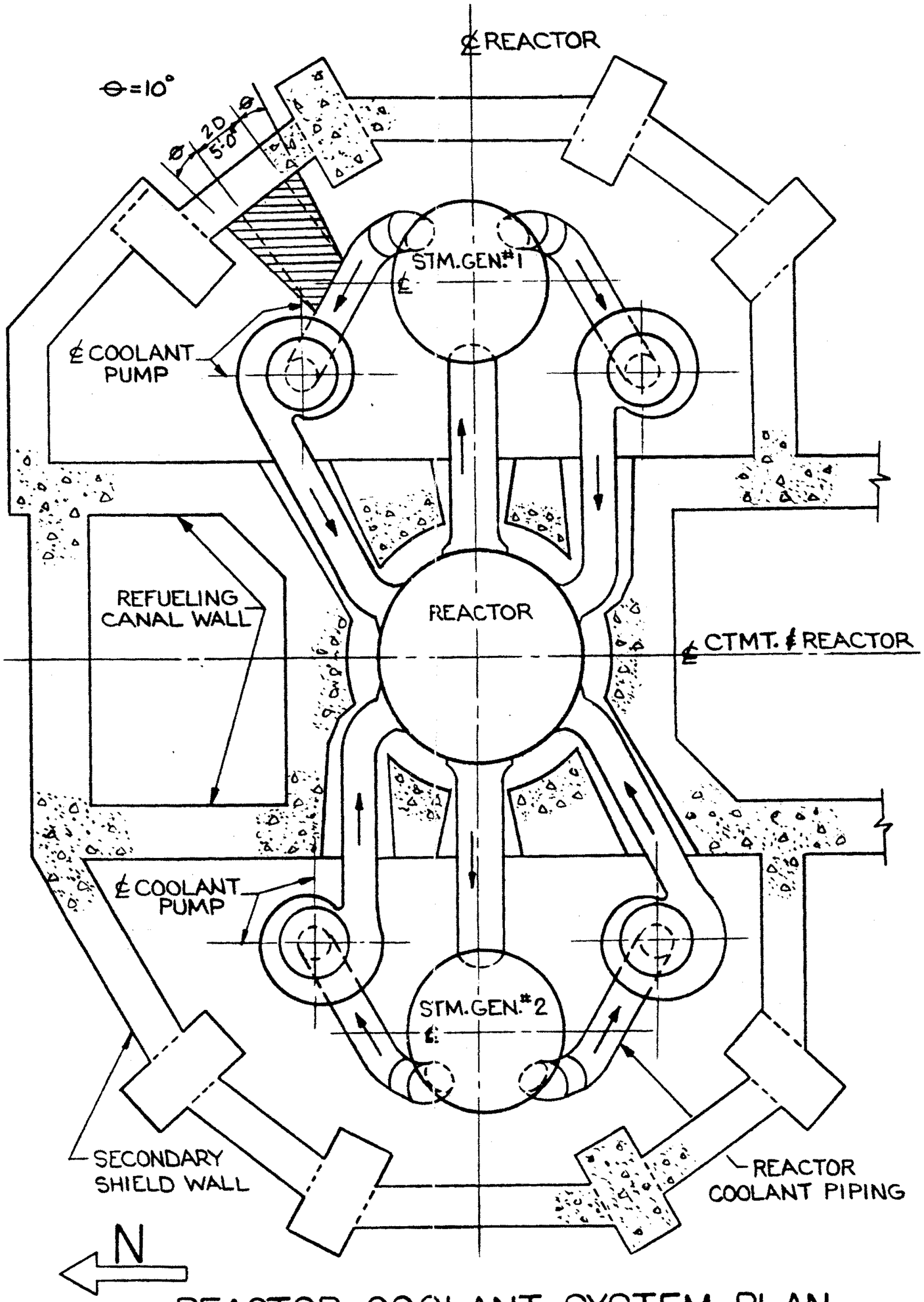
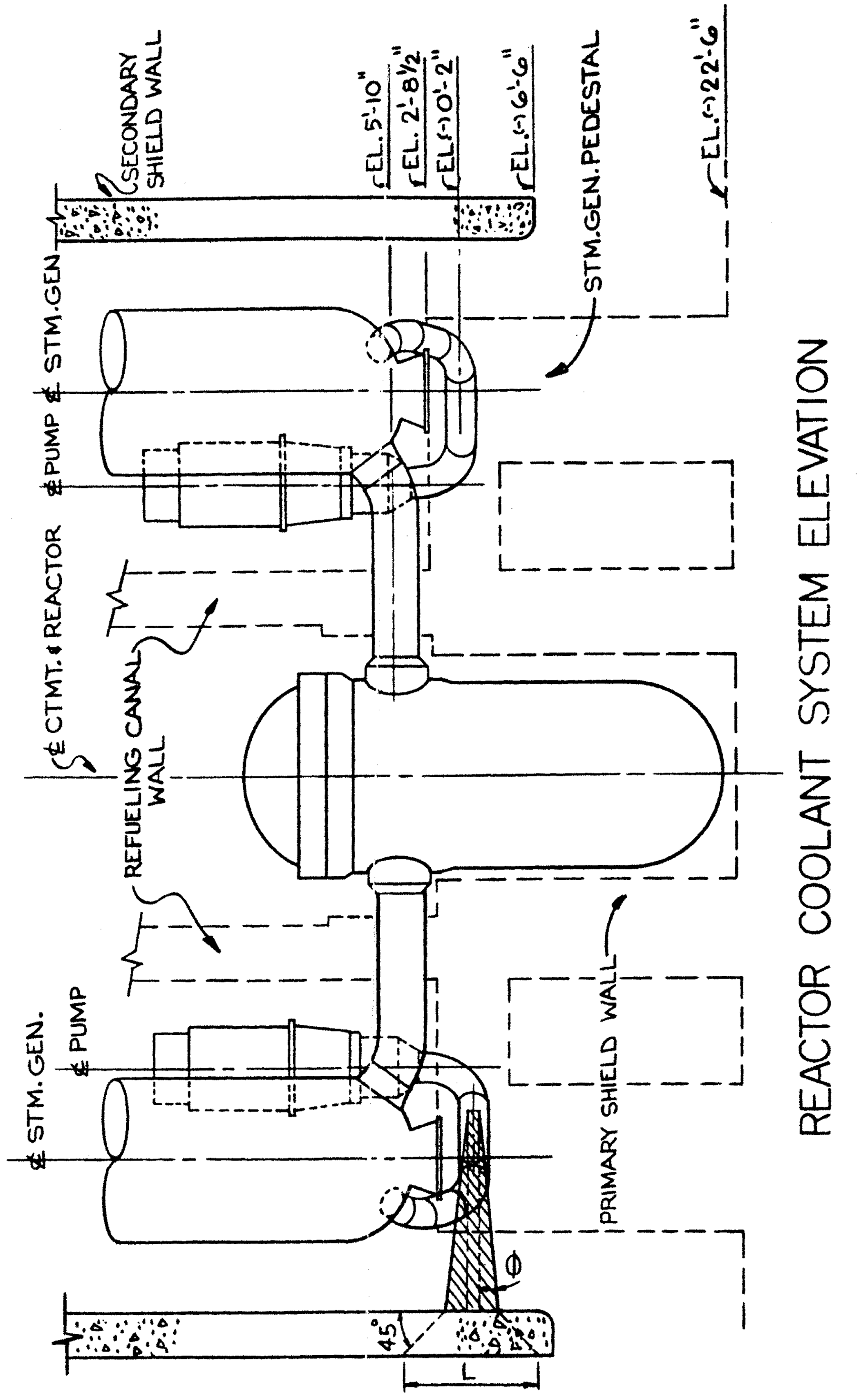


FIGURE 5.2–35 REACTOR COOLANT SYSTEM PLAN



REACTOR COOLANT SYSTEM PLAN

FIGURE 5.2–36 REACTOR COOLANT SYSTEM ELEVATION



MILLSTONE NUCLEAR POWER STATION
MILLSTONE UNIT 2
FIGURE 5.2-36

FIGURE 5.2–37 DELETED BY FSARCR 04-MP2-016

5.3 ENCLOSURE BUILDING

5.3.1 GENERAL DESCRIPTION

The enclosure building is a limited leakage steel framed structure with uninsulated metal siding and an insulated roof deck. It also includes those portions of the auxiliary building adjacent to the containment, as shown in Figures 5.3–1 and 5.3–2. The enclosure building completely surrounds the containment above grade and is designed and constructed to ensure that an acceptable upper limit of leakage of radioactive materials to the environment would not be exceeded in the unlikely event of a loss-of-coolant incident.

The use of a suitable formed gasket material at all joints will provide assurance that the required degree of air tightness and partial vacuum will be maintained within the enclosure building. Two continuous lines of caulking are provided at all lap joints of both siding and decking, with the exception of the east and west wall blowout panels. A single line of caulking shall be provided at the bottom end lap and side laps of the blowout panels. The caulking may be applied to either the interior building seam or the exterior building seam.

Principal dimensions of the enclosure building are as follows:

Length (feet)	153.0
Width (feet)	147.0
Height (feet)	147.0
Decking (gauge)	20
Siding (gauge)	22

The enclosure building is supported partially on concrete grade beams and caissons, partially on the roof of the auxiliary and turbine buildings, and partially on the dome of the containment. The interior of the enclosure building contains permanent ladders, stairways and catwalks which provide access to the upper exterior regions of the containment and to equipment in this building. In addition, permanent work platforms are furnished for the periodic surveillance of the post-tensioned prestressing tendons.

Concrete floor slabs are provided at grade between the enclosure building and the containment, and also at Elevations 36–6 and 38–6. A waterproofing membrane is provided under the slabs at grade and is extended down and around the containment below grade, as shown on Figure 5.3–3. Between the waterproofing membrane and the containment wall, corrugated asbestos-cement siding is installed as shown in Figure 5.3–4, to provide a passage of least resistance for possible leakage from the containment below grade to the enclosure building.

There are two stacks that exhaust radioactive effluents from the Millstone Unit 2 operations. Radioactive effluents are piped to the Millstone stack that was provided for the Millstone Nuclear Power Station, Unit 1. This stack provided for future expansion to accept effluent gases from the Unit 2 plant. The physical features of the stack are provided in Section 3.8 of the FSAR for the Millstone Nuclear Power Station, Unit 3. Gas volume increase is less than one percent, resulting in an exit gas velocity of 5,723 feet per minute. Section 3.8, of the Unit 3, FSAR lists the

Millstone stack as a Class I structure. This section outlines the design criteria for earthquake loading and the dynamic analysis applied to the structure. Stack failure would not directly impact any safety related equipment.

The only other stack that exhausts radioactive effluents to the atmosphere from Unit 2 is the stack located atop the enclosure building. The stack is constructed of one-quarter inch steel plate and standard structural shapes. Overall height is 13 feet. This is a seismic Class I stack, designed in accordance with the criteria contained in Section 5.3.3 of the FSAR. The stack has a constant rectangular cross section which has dimensions of 4 feet 0 inches by 9 feet 6 inches. Exit velocity of effluents is 1,684 feet per minute with two fans operating and 2,526 feet per minute with three fans operating. During normal plant operation, two fans are operating.

5.3.2 CONSTRUCTION MATERIALS

The following materials are used in the construction of the enclosure building:

a. Structural steel	ASTM A-36
b. Concrete (psi)	
Grade beams and Caissons	4,000
Slabs at grade	3,000
Floor slabs	3,000
c. Reinforcing steel	ASTM A615, Grade 60
d. Metal siding	22 gauge
e. Metal roof decks	20 gauge

5.3.3 DESIGN BASES

The design of the enclosure building provides the required features as outlined in General Design Criteria 1, 2, 3, 4, 5, 60, Appendix A of 10 CFR Part 50.

5.3.3.1 Bases for Design Loads

The following loads are considered in the design of the enclosure building:

a. Dead loads

b. Live loads including external pressures

c. Earthquake loads

d. Wind and tornado loads

5.3.3.1.1 Dead Loads

The dead loads consist of the weight of the steel frame, roof, metal siding, and access stairs and ladders.

5.3.3.1.2 Live Loads

The design live loads for the enclosure building are as follows:

a. Roof, snow loads (psf) 60

b. Slabs at grade

Equipment hatch area	AASHO H-20 truck load
Other areas (psf)	500
External pressure (independent of wind and tornado loads) (psf)	10

Weights of equipment as indicated on drawings supplied by the manufacturer are included as live loads.

5.3.3.1.3 Earthquake Loads

The earthquake loads are predicated on an operating basis earthquake (OBE) at the site having a horizontal ground surface acceleration of 0.09 g. In addition, a design basis earthquake (DBE) having a horizontal ground surface acceleration of 0.17 g is used to check the design of the enclosure building to ensure no loss of structural function. The seismic design spectrum curves are given in Figures 5.8–1 and 5.8–2. A vertical component two-thirds of the magnitude of the horizontal ground surface component is applied at the base simultaneously.

A dynamic analysis including the effects of the attachments to the other structures is used to arrive at the equivalent static loads for the design.

5.3.3.1.4 Wind and Tornado Loads

Winds loads for the design of the enclosure building are based on a wind velocity of 115 mph with gusts up to 140 mph. The ASCE Paper 3269 is used to determine the shape factors. However, the provisions in the paper for gust factors and variations of wind velocity with respect to height are not applied.

The entire enclosure building is designed to resist the effects of the 140 mph wind.

Tornado loads on the enclosure building are based on a tornado funnel having a peripheral tangential velocity of 300 mph and a translational velocity of 60 mph. These velocities are combined, resulting in a design basis tornado wind velocity of 360 mph. The enclosure building, adjacent to structures which house safety related equipment, is designed so that its structural framing will withstand tornado winds, but the siding will be blown away.

The wind velocity is assumed to be uniformly distributed over the height of the structure. Probable missiles in the form of siding are less critical than the design missiles as specified in Section 5.2.5.1.2 of the FSAR. The siding when blown off may induce superficial damage to the adjacent structures, but the structural integrities of the adjacent structures will be maintained. The design requirements for tornado loads for structures which house safety related equipment for shutdown are given in Section 5 of the FSAR.

The design of the enclosure building for tornado loads assumes that tornado wind is not coincident with a loss-of-coolant accident (LOCA) or earthquake.

5.3.3.2 Design Load Combination and Structural Analysis

The enclosure building is designed to meet the performance and strength requirements of the following loading combinations:

a. At design loads

b. At factored loads

The design of structural steel is in accordance with the AISC Manual of Steel Construction. The design of concrete is in accordance with the ACI Code 318-63.

5.3.3.2.1 At Design Loads

The enclosure building is analyzed for the following specific loading conditions:

a. D + L = Construction

b. D + l + E = Operating

c. D + L + W = Operating

Where

D = dead loads

L = live loads

E = operating basis earthquake loads (0.09 g)

W = wind or tornado loads

5.3.3.2.2 At Factored Loads

The enclosure building is analyzed for the factored load combination to ensure that its structural integrity is not affected.

$$C = \frac{1}{\varnothing}[(1.0)(D) + E']$$

Where

C = required capacity of the structure

∅ = 0.90 for fabricated structural steel

D = dead loads

E′ = design basis earthquake (0.17 g)

The stresses of the members of the structure at factored loads are limited to the yield stresses of the structural steels.

5.3.3.2.3 Seismic Analysis

The seismic analysis of the enclosure building is made on a mathematical model which consists of the lumped masses of the containment structure and the enclosure building. The seismic response of the combined model is obtained in accordance with the procedures outlined in Section 5.8.

5.3.4 THROUGH-LINE LEAKAGE EVALUATION

To evaluate the through-line leakage that can bypass the enclosure building filtration region (EBFR), the fluid systems penetrating containment are categorized as follows:

a. Piping Systems open to the containment post-accident atmosphere.

b. Piping Systems which are closed and therefore not exposed to the containment post-accident atmosphere.

The following assumptions are made to postulate the maximum hypothetical conditions:

a. There is either a seismic occurrence and all Seismic Class 2 lines are broken, or there is no seismic occurrence and all Seismic Class 2 lines are intact.

b. The single failure criterion applies to Seismic Class 1 components only.

The condition of a seismic occurrence is not considered. Should the pipe break within the EBFR, all potential containment leakage would be processed by the enclosure building filtration system (EBFS) as per design. The EBFS has ample capacity for this event.

From the basis formulated, systems which are not normally opened to the containment atmosphere, or normally closed systems, either do not leak (assuming no seismic event) or are vented to the EBFR (assuming a seismic event). Normally, closed systems which may be opened to the atmosphere during accident conditions, such as lines connected to the reactor coolant pressure boundary, are not considered. These systems are either operating at a higher pressure or form closed loops. Therefore, assuming a single failure, these lines either prevent leakage by the higher pressure or contain the leakage by the closed loop.

From this basis, an evaluation was performed to establish and document those containment penetrations that have the potential for providing leakage pathways from the reactor containment to areas beyond the EBFR. These leakage pathways could result in Post-Accident Containment Atmosphere bypassing the EBFR and discharging directly to the atmosphere thereby increasing on-site and off site doses under radiological accident conditions. Leakage through these pathways is referred to as “bypass leakage.”

For a leakage pathway to viably result in bypass leakage, the pathway must be open to the containment atmosphere post-accident and provide a means of transporting the containment atmosphere beyond the EBFR as well as a means for the containment atmosphere to escape the piping or duct. For containment penetrations that are confirmed to contribute to bypass leakage, leakage rates may be based on measured values as opposed to the recommended or maximum allowable values used for testing.

In cases where measured values are used, steps are taken to ensure that degradation of the valve sealing capabilities are taken into account commensurate with the severity of service and the required time intervals between valve maintenance.

The evaluation concluded the following penetrations are considered to qualify as systems that are open both inside containment and outside containment, extend beyond the EBFR, and could contribute to bypass leakage:

Penetration	System
14	Containment Sump Pump Discharge
37	Instrument Air System
38	Station Air System
42	Fuel Transfer Tube
61	Hydrogen Monitoring System
62	Hydrogen Monitoring System
67	Refueling Cavity Drain
68	Refueling Cavity Skimmer
85	Containment Pressure Test Connection
86	Hydrogen Monitoring System
87	Hydrogen Monitoring System

The off-site and control room dose analyses are based on a calculated maximum bypass leakage (refer to Section 14.8.4). For each verified bypass leakage pathway, a recommended leak rate is provided based on the limits used to satisfy the leakage limits established for the testing required by 10 CFR 50, Appendix J. Total leakage from all verified bypass leakage pathways will be summed and compared to the total limit. The control room and off-site radiological dose calculations establish the maximum limit for total bypass leakage. In the event that total bypass leakage exceeds this value, repairs will be performed to reduce bypass leakage to an acceptable level.

The provisions for initial and periodic leak testing of containment penetrations and maximum allowable leakage are specified in Table 5.2-11 of the FSAR and Section 3.6.1.2.c of the Technical Specifications respectively.

FIGURE 5.3–1 ENCLOSURE BUILDING

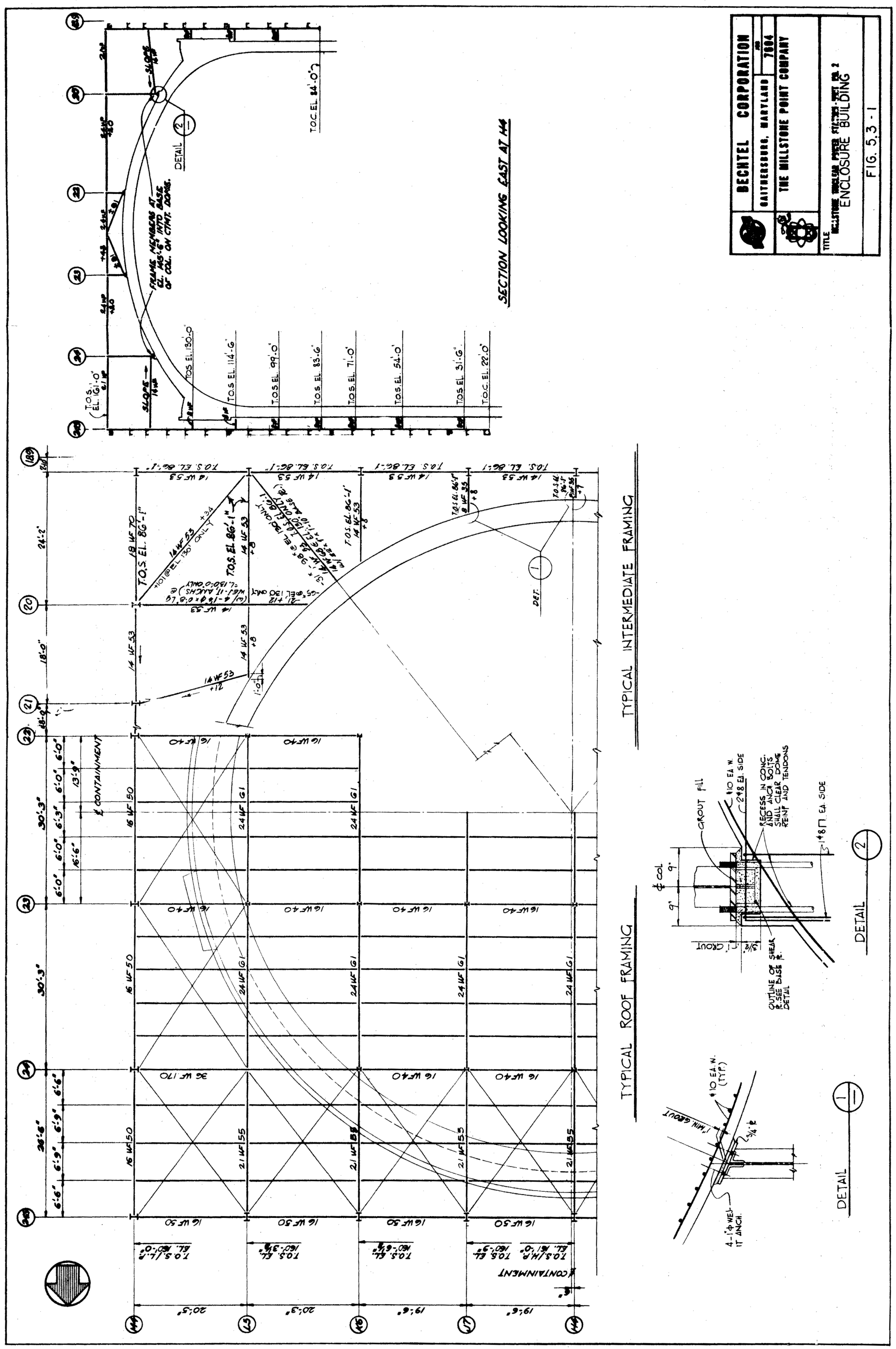


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FIGURE 5.3–2 ENCLOSURE BUILDING LAYOUT

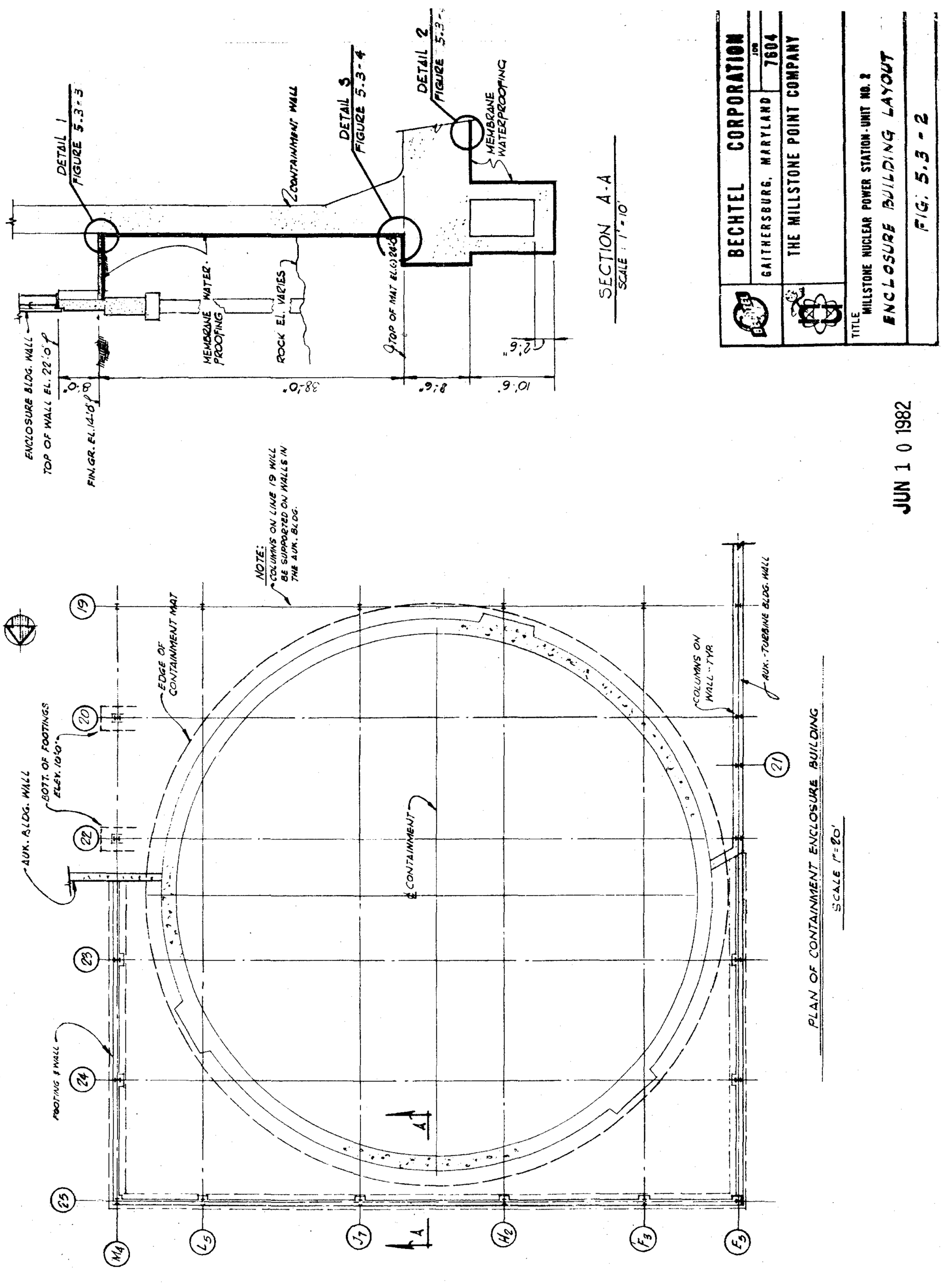


FIGURE 5.3–3 WATERPROOF MEMBRANE DETAILS

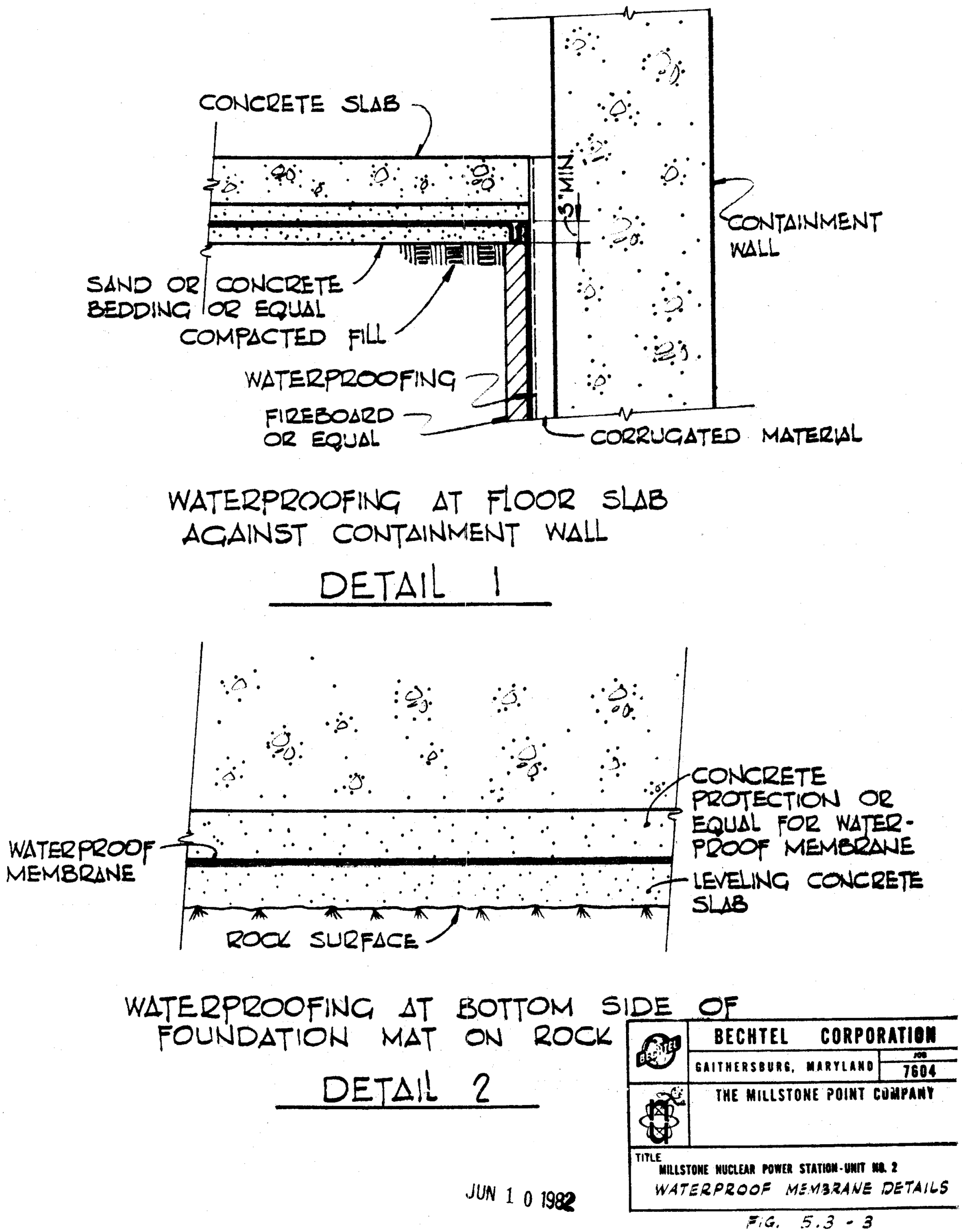


FIGURE 5.3–4 WATERPROOF MEMBRANE DETAILS

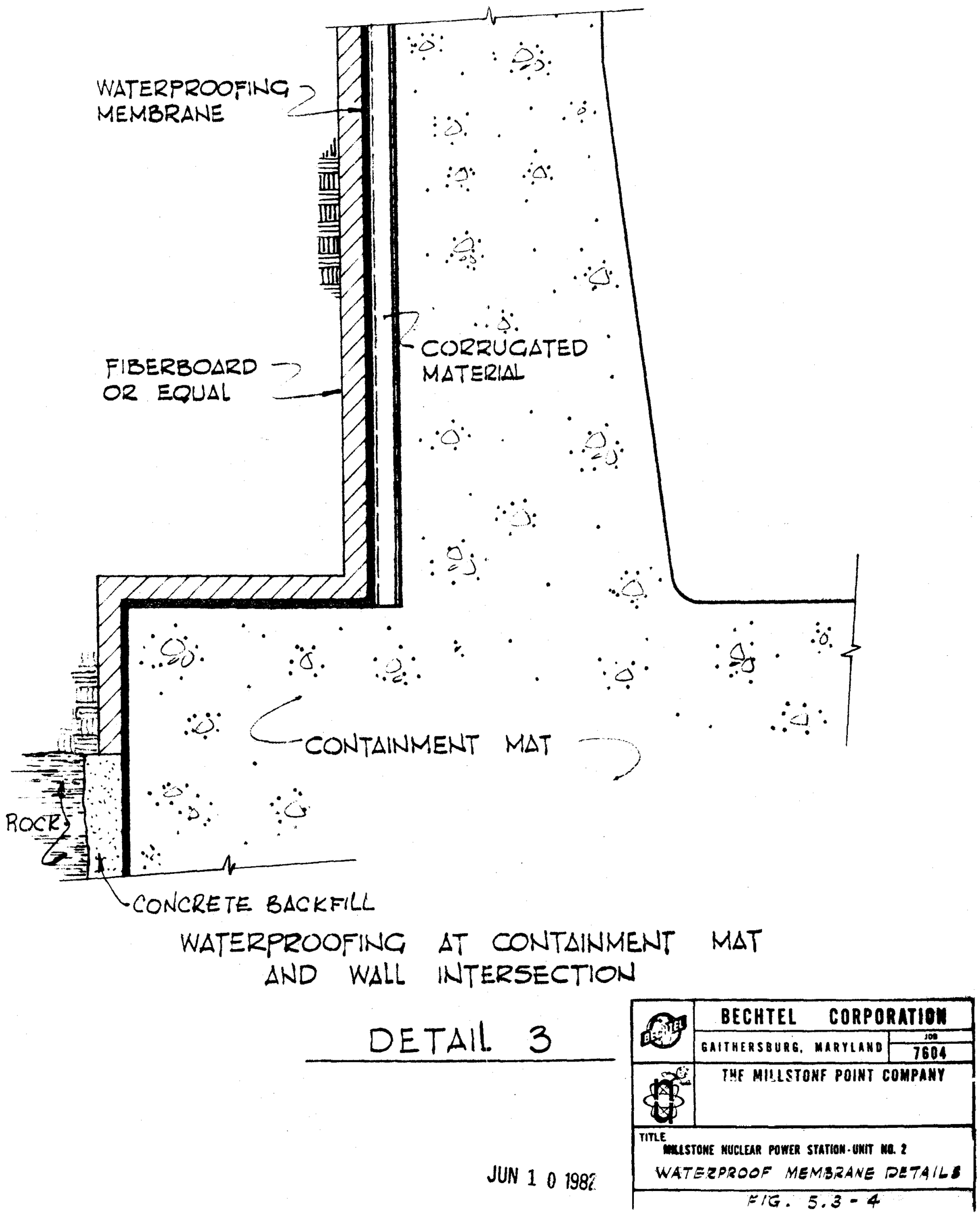
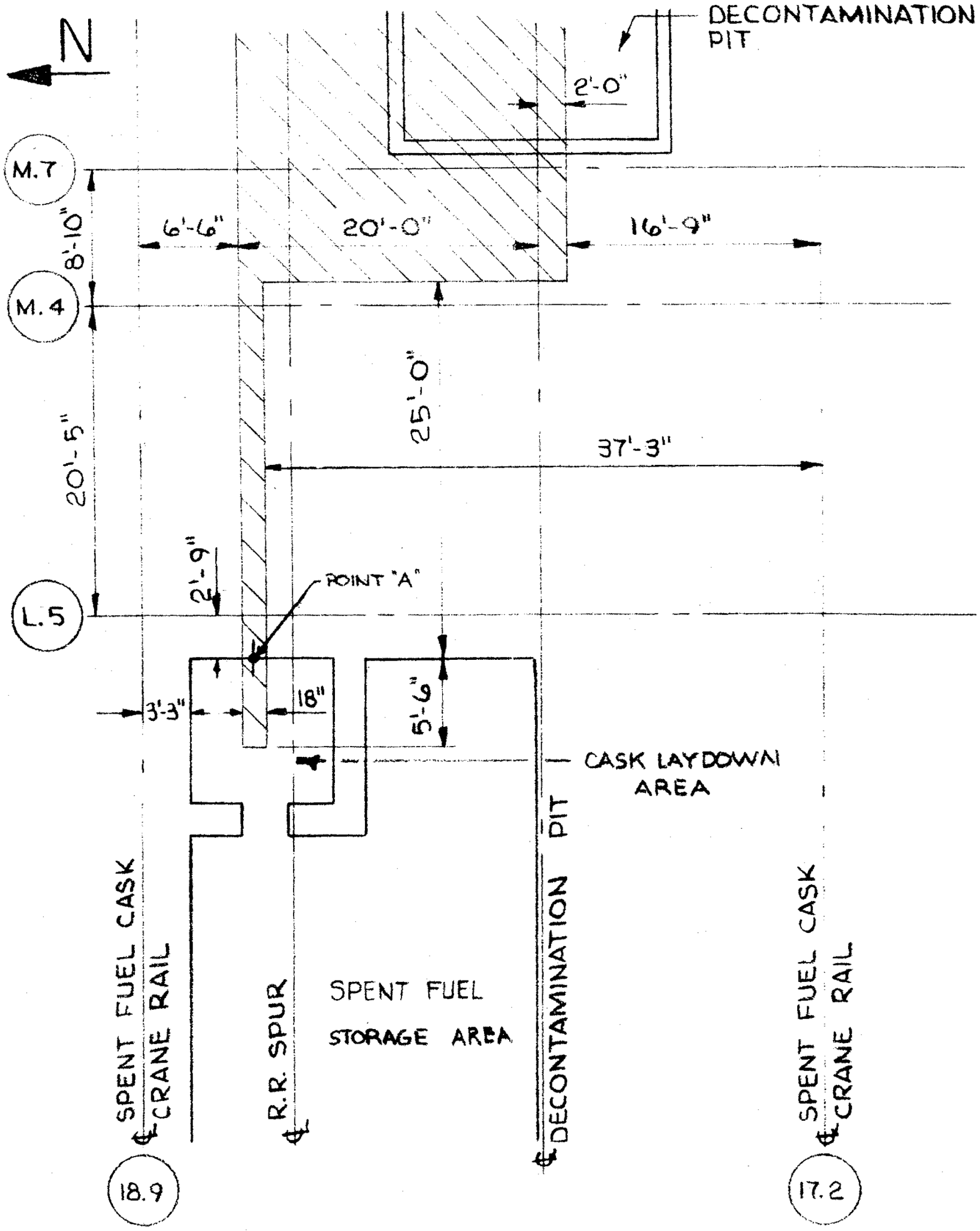


FIGURE 5.3–5 SPENT FUEL CASK TRAVEL LIMITS



NOTE:
THE HATCHED AREA DEFINES THE LIMITS OF THE CENTERLINE OF THE SPENT FUEL CASK WHILE BEING HANDLED BY THE SPENT FUEL CASK CRANE IN THE CASK HANDLING MODE.

SPENT FUEL CASK TRAVEL LIMITS FIGURE 5.3-5

2 - 16 - 73

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5.4 AUXILIARY BUILDING

5.4.1 GENERAL DESCRIPTION

The auxiliary building is a multistory, reinforced concrete structure with flat slabs and shear walls. Some open areas of the building are supported by structural steel columns to preserve space and allow flexibility in the design. The portion of the building west of column line M.7 is founded on bedrock approximately 60 feet below the ground surface, while the eastern end of the building is supported by compacted structural backfill. These two portions of the building are separated from each other by an expansion joint at line M.7 to allow for differential movements.

The auxiliary building is separated from the containment, which is to the north, by an expansion joint and from the turbine building to the west by slotted connections. Although the control rooms of Units 1 and 2 are combined in one area, the buildings are separated by Teflon lined sliding bearings. These isolation joints provide the auxiliary building with structural independence from the surrounding buildings in the lateral direction.

General layouts at the various elevations and sections through the auxiliary building are shown on Figures 1.2–7 and 1.2–14.

5.4.1.1 Fuel Storage Facility

5.4.1.1.1 New Fuel Storage

The new fuel storage is bounded by column lines 17.2 and 18.9 and column lines S and N.3 at Elevation 38-6.

5.4.1.1.2 Spent Fuel Storage

Spent fuel storage is provided between column lines 17.2 and 18.9 and column lines H.4 and L.5 at Elevation (-)2–0. The storage area consists of a reinforced concrete pool lined with one-fourth inch thick stainless steel plate to Elevation 38-6. Normal water level is to Elevation 36-6. The spent fuel is protected from a main steam line rupture by a reinforced concrete wall which is located north of and parallel to the pool.

A leak chase system consisting of channels embedded behind the liner plate at all seams and connected to a collector system is used to monitor and control any possible leakage from the pool. A description of the monitoring system is provided in Section 5.4.3.3.4.

An overhead crane with the capacity of 125 tons is employed to handle the spent fuel cask. The crane is designed to meet the single failure requirements in accordance with NUREG-0612 and NUREG-0554. A system of interlocks is also provided on the crane that defines a safe load path. This safe load path is indicated on Figure 5.3–5.

5.4.1.1.3 Compliance with Safety Guide 13

The design of the fuel storage structures complies with the structural requirements of Safety Guide 13.

5.4.2 CONSTRUCTION MATERIALS

The following materials are used in the construction of the auxiliary building.

a. Structural and miscellaneous steel	
Rolled shapes, plates, and bars	ASTM A-36
Crane rails	Bethlehem Steel
High strength bolts	ASTM A-325 or A-490
Stainless Steel	ASTM A-240 Type 304
b. Reinforcing steel	
Deformed bars	ASTM A-615 Grade 60
c. Concrete, 28 day strength (psi)	
Lean concrete backfill	2,000
Foundation mat slab	
Auxiliary Building	3,000
Warehouse portion of the Auxiliary Building	4,000
All other concrete	3,000
d. Interior coatings (Original Construction)	
1. Concrete and masonry surfaces	
Primer	Keeler & Long Number 7107 epoxy white primer
Finish coat (floor and wainscot)	Keeler & Long Number 7107 epoxy white enamel (tinted)
Finish coat (above wainscot)	Keeler & Long Number 7107 epoxy white enamel
2. Carbon steel	
Primer (wainscot)	Keeler & Long Number. 7107 epoxy white primer
Finish coat (wainscot)	Keeler & Long Number. 7475 epoxy white enamel (tinted)

Primer (above wainscot)	Keeler & Long Tri-Polar Primer
Finish coat (above wainscot)	Keeler & Long Tri-Polar Enamel

e. Coating materials used in the auxiliary building represent current day technology and comply with current day environmental requirements regarding volatile organic compound (VOC) content and hazardous material constituents such as lead, asbestos, and hexavalent chrome. Specific products are selected to provide service performance comparable to or improved over that of the original coating materials. Coating colors are chosen to help maintain a safe working environment.

5.4.3 DESIGN BASES

The design of the auxiliary building provides the required features as outlined in General Design Criteria 1, 2, 3, 4, 5, 61, 62, 63, Appendix A of 10 CFR Part 50.

5.4.3.1 Bases for Design Loads

The auxiliary building is designed for all credible combinations of loading, including loads under normal operation, loads during a steam line rupture, and loads due to adverse environmental conditions. The following loadings are considered:

a. Dead loads

b. Live loads

c. Thermal loads

d. Earthquake loads

e. Lateral pressure loads

f. Wind and tornado loads

g. Pipe restraint loads

h. Pipe whipping loads

i. Cask drop loads

j. Fuel transfer tube bellows

5.4.3.1.1 Dead Loads

These loads consist of the weight of all structural materials, including all partitions, hangers, trays, pads and pedestals, and equipment dead loads. These are specified on the drawings supplied by the manufacturers of the equipment installed within the building.

5.4.3.1.2 Live Loads

Live loads consist of design floor loads, pool and tank liquid weights, piping loads, and equipment live loads as specified on the drawings supplied by the manufacturers of the equipment installed within the building. A snow load of 60 psf is applied on the roof.

5.4.3.1.3 Thermal Loads

Thermal loads are those induced in the spent fuel pool floor and walls due to the thermal gradients across these elements. Thermal gradients may be caused by an increase in water temperature during operating conditions or by an accident. The interior temperatures of the pool are assumed to be 150°F at operating conditions and 212°F during an accident. The ambient temperature exterior to the pool is assumed to be 55°F for computation of stresses.

5.4.3.1.4 Earthquake Loads

These loads are as defined in Sections 5.2.2.1.5 and 5.8.

5.4.3.1.5 Lateral Pressure Loads

The static lateral soil pressure loads are as follows:

Active -	above water table	55 (psf/ft)
	below water table	95 (psf/ft)
Passive -	above water table	370 (psf/ft)
	below water table	200 (psf/ft)

A surcharge of 200 psf, or an 8,000 pound wheel load, is considered in all cases.

Buoyant forces resulting from the displacement of ground water are supplied to the structure. The following water levels are considered:

Ground water	Elevation 5–0
Flood water	Elevation 18–1

The dynamic soil and hydrodynamic pressures are discussed in Section 5.8.2.2 and are considered to act on the structural elements below grade, where applicable.

5.4.3.1.6 Wind and Tornado Loads

Wind loads for the auxiliary building are determined on the basis of the American Society of Civil Engineers (ASCE) Paper 3269, “Wind Forces on Structures,” using the highest wind velocity at the site for a 100 year recurrence period. The ASCE Paper 3269 is used mainly to determine the shape factors. Based upon the site location and the structure classification, the design wind velocity is taken to be 115 mph with gusts up to 140 mph.

The auxiliary building has been analyzed for tornado loads (not coincident with an accident or earthquake) on the following basis:

a. Differential bursting pressure between the interior and exterior of the structure is assumed to be three psi pressure occurring in three seconds (1 psi/second), followed by a calm for two seconds and a re pressurization.

b. Lateral loads on the structure are based on a tornado funnel which is conservatively assumed to have a peripheral tangential velocity of 300 mph and a translational velocity of 60 mph. The applicable portions of the wind design methods described in the ASCE Paper 3269 are used, particularly for the shape factors. The provisions in the paper for gust factors and variation of wind velocity with respect to height are not applied. The wind velocity is assumed to be uniformly distributed over the height of the structure.

c. Tornado driven missiles as defined in Section 5.2.5.1.2.

With the exception of the missile impact area, the allowable stresses to resist the effects of tornadoes are 90 percent of the yield strength of the reinforcing steel and 85 percent of the ultimate strength of the concrete.

A discussion of the probability of tornado occurrence is presented in Section 2.3.

5.4.3.1.7 Pipe Restraint Loads

These are the loads imparted to the structure from the pipe restraints produced by either a postulated pipe rupture or an earthquake. See Section 6.1.4 for pipe rupture criteria.

5.4.3.1.8 Pipe Whipping Loads

These are the loads imposed on the structure due to whipping from a postulated pipe rupture. See Section 6.1.4 for pipe rupture criteria.

5.4.3.1.9 Cask Drop Loads

The upgrade of the MP2 Spent Fuel Cask Crane to single failure criteria has precluded the need to postulate a Spent Fuel Cask Drop accident. The crane upgrade will give positive control of the lifted loads even with the worst single failure. This section has been retained to provide the historical background for the design and analysis of the spent fuel pool.

The following design criteria were used in the analysis of the spent fuel pool in the event that a cask is accidentally dropped:

a. Weight of cask in air (lb.)	200,000	
b. Length of cask (feet)	19	
c. Diameter of cask (feet)	8	
d. Distance of drop (feet)		
In air		2.75
In water		35.5

As shown on Figure 5.3–5, the only area of the spent fuel pool into which the cask could be dropped directly is the cask laydown area. The cask laydown area is isolated from the spent fuel storage area by two-foot thick, permanent, reinforced concrete walls and a temporary gate placed in the fuel transfer slot. The base slab of the cask laydown area is composed of seven feet of reinforced concrete resting on a mass of monolithic concrete which, in turn, rests on bedrock. Therefore, a cask dropped in this area would travel vertically downward as restrained by the surrounding walls. Any damage would be limited to rupturing of the spent fuel pool liner and local superficial crushing of concrete in the area of impact of the end of the cask. Leakage through the ruptured liner would be detected in the control room and would be stopped by closing the valve that connects the leak collection channel for the ruptured zone(s) to the leak detection instrumentation.

If during handling, the cask is dropped on or near point “A,” as shown on Figure 5.3–5, there exists a possibility that the cask could fall or tumble into the spent fuel storage area. The fall would provide some local concrete crushing in the spent fuel pool and laydown area walls at elevation (+) 38 feet 6 inches. The cask would then slide into the spent fuel storage area of the pool. The cask would crush the spent fuel rack module(s) that it landed on, but the buoyant effect of the water combined with the crushing of the rack would dissipate most of the kinetic energy of the falling cask. Therefore, the probable damage would be limited to rupture of the spent fuel pool liner and local crushing of concrete where the cask impacted. The dose impact of damage to spent fuel stored in the pool (both intact and consolidated) would be mitigated by administratively controlling the age of the stored fuel in the affected area around the cask laydown area of the spent fuel pool. Whenever a shielded cask is on the refueling floor, all stored fuel within a specified distance of the cask laydown area shall have decayed a minimum amount of time from subcritical reactor operation in accordance with the Technical Specifications. The seven foot thick base and six foot thick walls, of reinforced concrete, would remain intact. Leakage would be detected and stopped as described above.

Makeup water would be available as discussed in Section 9.5.

5.4.3.1.10 Fuel Transfer Tube Bellows

The following loads were used in the design of the fuel transfer tube and bellows:

Design pressure, internal (psi)	60
Design temperature (°F)	290

Lateral movement (inches) 0.14

Axial movement, expansion or contraction (in.) 0.5

Displacements are selected to accommodate an assumed differential settlement of one-eighth inch between the buildings. Since both the containment and auxiliary buildings are founded on rock, this motion is minimal.

5.4.3.2 Design Load Combinations

To ensure the structural integrity of the auxiliary building, the working stress method of design is used for the various loading combinations. For the operating conditions, normal allowable stresses given in the American Institute of Steel Construction (AISC) Manual of Steel Construction 1963, and the American Concrete Institute (ACI)-318-63, “Building Code Requirements for Reinforced Concrete” are used. These allowable stresses are increased by 33-1⁄3 percent for the 115 mph base wind loads and the operating basis earthquake (OBE) loads. For the tornado wind and the design basis earthquake (DBE), the allowable stresses are 90 percent of the yield strength of the reinforcing, and 85 percent of the ultimate strength of concrete.

The load combinations are listed:

a. $D + L$

b. $D + L + W_w$

c. $D + L + W_t$

d. $D + L + E$

e. $D + L + E'$

f. $D + L + P_e + W_t + H_w$

g. $D + L + T + E$

h. $D + L + T + E'$

i. $D + L + P_e + E$

j. $D + L + P_e + E'$

k. $D + L + F_p + E'$

l. $D + L + F_c$

m. $D + L + F_r + E'$

where:

D = dead loads

L = live loads

W_w = wind loads (115 mph base)

W_t = tornado loads (360 mph base

E = OBE

E′ = DBE

P_e = soil pressure

F_p = pipe whipping loads

F_c = cask drop loads

F_r = pipe restraint loads

H_w = hydrostatic pressure

T = thermal loads

These load combinations are applied to the portions of the structure housing or associated with the various systems as follows:

	Item	
a.	Cask crane structure	a, b, c, d, e
b.	Chemical addition and sampling system	a, c, d, e, f, i, j
c.	Chemical and volume control system (CVCS)	a, c, d, e, f, i, j
d.	Containment spray pumps	a, c, d, e, f, i, j
e.	Control room	a, b, c, d, e
f.	Diesel generator room and day tanks	a, c, d, e, f
g.	Electrical distribution system	a, b, c, d, e
h.	New fuel storage	a, b, c, d, e
i.	Reactor building closed cooling water (RBCCW) system	a, c, d, e, f, i, j
j.	Safety injection systems (SIS)	a, c, d, e, f, i, j
k.	Spent fuel cooling system	a, c, d, e, f, i, j
l.	Spent fuel pool	a, b, c, d, e, g, h, k, l
m.	Waste processing systems	a, c, d, e, f, i, j

The design loads and stresses for all concrete walls and columns are within the allowable values for each wall or column. Design stresses are not applicable for the columns inasmuch as the interaction diagrams are used to proportion reinforcing and concrete required to support a given load with its corresponding eccentricity. Exterior walls having vertical and lateral loads are designed for bending and axial loadings, and the resulting combined stresses are kept within the code allowables. There are numerous walls, slabs, columns, and beams within the buildings, each element of which was designed for the pertinent loading. Design stresses in the various components are recorded in the design calculations. The allowable loading, or combined loading, depends on the reinforcing which was added and varies considerably depending upon the applied conditions.

The maximum combined stress ratio in the steel cask crane frame is 0.904, resulting from an axial stress of 3.53 ksi and a bending stress of 31.8 ksi. This stress occurs under loading combination e., above.

In addition to the various load combinations included, all Category I structures outside the containment that could be pressurized in the event of a postulated pipe rupture are designed to satisfy the following load combinations:

(1) $U = D + L + T + R_a + 1.5P_a$

(2) $U = D + L + T + R_a + 1.25P_a + F_r + F_c + F_j + 1.25E$

(3) $U = D + L + T + R_a + P_a + F_r + F_c + F_j + E'$

where:

U = total design load

D = dead loads

L = live loads

R_a = pipe reactions under thermal condition generated by a postulated break

F_r = equivalent static pipe restraint loads

F_c = equivalent static pipe whipping loads, including the effects of missiles

F_j = equivalent static jet impingement loads

P_a = equivalent static differential pressure load generated by a postulated pipe break

E = OBE loads

E' = DBE loads

T = thermal loads under thermal conditions generated by a postulated break

For the above loading conditions, the allowable stresses are as follows:

Concrete Construction:

90 percent of the yield strength for reinforcing steel

85 percent of the ultimate strength of concrete

Steel Construction:

Allowable stresses specified in Part 2 of the AISC "Specification for the Design, Fabrication and Erection of Structural Steel for Building," April, 1963.

5.4.3.3 Structural Analysis

5.4.3.3.1 Seismic Analysis

Seismic analysis is performed in accordance with Section 5.8.

5.4.3.3.2 Wind and Tornado Analysis

The design wind loads on the auxiliary building are a function of the kinetic energy per volume of the moving air mass. The product of one-half of the air density and the square of the resultant design velocity results in a pressure corresponding to the design wind.

Determination of the design wind pressure on the structure is in accordance with the ASCE Paper 3269, "Wind Forces on Structures."

The pressure corresponding to the standard air at 0.07651 pcf at 15°C and 760 mm of mercury in terms of the velocity at the appropriate height zone is given by:

$$q = 0.002558V^2$$

Similarly, the design pressure, including the effect of the shape coefficient (C_d), is given by:

$$p = q \times C_d = 0.002558V^2C_d$$

Using these equations and the wind velocities given in Section 5.4.3.1.6, the wind forces are calculated for the various parts of the auxiliary building and are then applied to the structure.

The spent fuel pool is protected from the tornado missile, which is described in Section 5.4.3.1.6, by a concrete roof over the cask crane frame and missile proof metal siding, as per Section 5.4.3.3.6.

5.4.3.3.3 Cask Drop in Spent Fuel Pool

The design criteria stated in Section 5.4.3.1.9 is applied using the following assumptions:

Water density at 120°F (lb/cubic foot)	61.7 borated water density
Strength of concrete (psi)	3,000
Modulus of concrete (E) (ksf)	4.78×10^5

The striking velocity at collision is determined by the use of the formula as defined in the paper "Tornado Protection for the Spent Fuel Storage Pool" by Miller and Williams, APED-5696 Class I, November 1968. The friction factor is ignored. A dynamic pressure factor (C_q) of unity is assumed in the analysis.

To determine the effects of accidentally dropping the cask on the bottom of the spent fuel pool in the cast laydown area, the kinetic energy was computed. This energy is considered to be dissipated as elastic strain energy within the bounds of the lean concrete mass supporting the laydown area. The contact area between the cask and the concrete is small.

However, the actual contact stress is calculated and found to be less than 1,000 psi. The impact of this contact may cause some local damage to the liner plate and/or concrete. However, the extent of the damage is small and will not result in any significant structural damage to the floor. The maximum stress in the concrete occurs at Elevation (-)2-0 and diminishes rapidly as the stress profile extends downward.

5.4.3.3.4 Stainless Steel Liner Plate for Spent Fuel Pool

[Note: Section 5.4.3.3.4 describes preoperational testing and repair of the spent fuel pool liner. It is retained, without change, for a historical record. The spent fuel pool leak monitoring and detection system is described in Section 9.5.2.1.]

Provision is made for ensuring the leak tightness of the spent fuel pool and refueling canal liner plate.

The test consists of two parts. In the first part of the test, a halogenated hydrocarbon gas is forced through the leak monitoring channels and a halogenated hydrocarbon detector is used to locate leaks in the liner plate weld seams inside the spent fuel pool. All leak indications are marked and repaired after the halogenated hydrocarbon gas is removed from the leak monitoring channels. All weld repairs are checked by a liquid penetrant test.

Upon completion of the repair, the pool is filled with water to the design level and monitored for 48 hours.

If no water is detected in the leak monitoring system, the pool is considered acceptable.

5.4.3.3.5 Fuel Transfer Tube

One expansion joint is installed in the fuel transfer tube. It is a bellows located in the fuel transfer canal in the auxiliary building.

The loads used in its design are given in Section 5.4.3.1.10.

The outside of the expansion joint in the transfer canal may be visually inspected by draining the transfer canal or by remote means. A test connection on the bellows provides a means of testing for bellows integrity. Repair would require draining the transfer canal.

A detail of the fuel transfer tube is shown in Figure 5.2–10.

5.4.3.3.6 Spent Fuel Pool Missile Protection

Withhold under 10 CFR 2.390 (d) (1)

[redacted]

[redacted]

[redacted]

[redacted]

Although a large portion of the missile energy would be spent in penetrating the conventional siding (if, indeed, penetration would occur), there is no possible trajectory that would allow the missile to directly impact the fuel assembly elements located at the bottom of the spent fuel pool.

Withhold under 10 CFR 2.390 (d) (1)

[redacted]

[redacted]

[redacted]

FIGURE 5.4–1 PROBABLE MISSILE TRAJECTORIES INSIDE AUXILIARY BUILDING

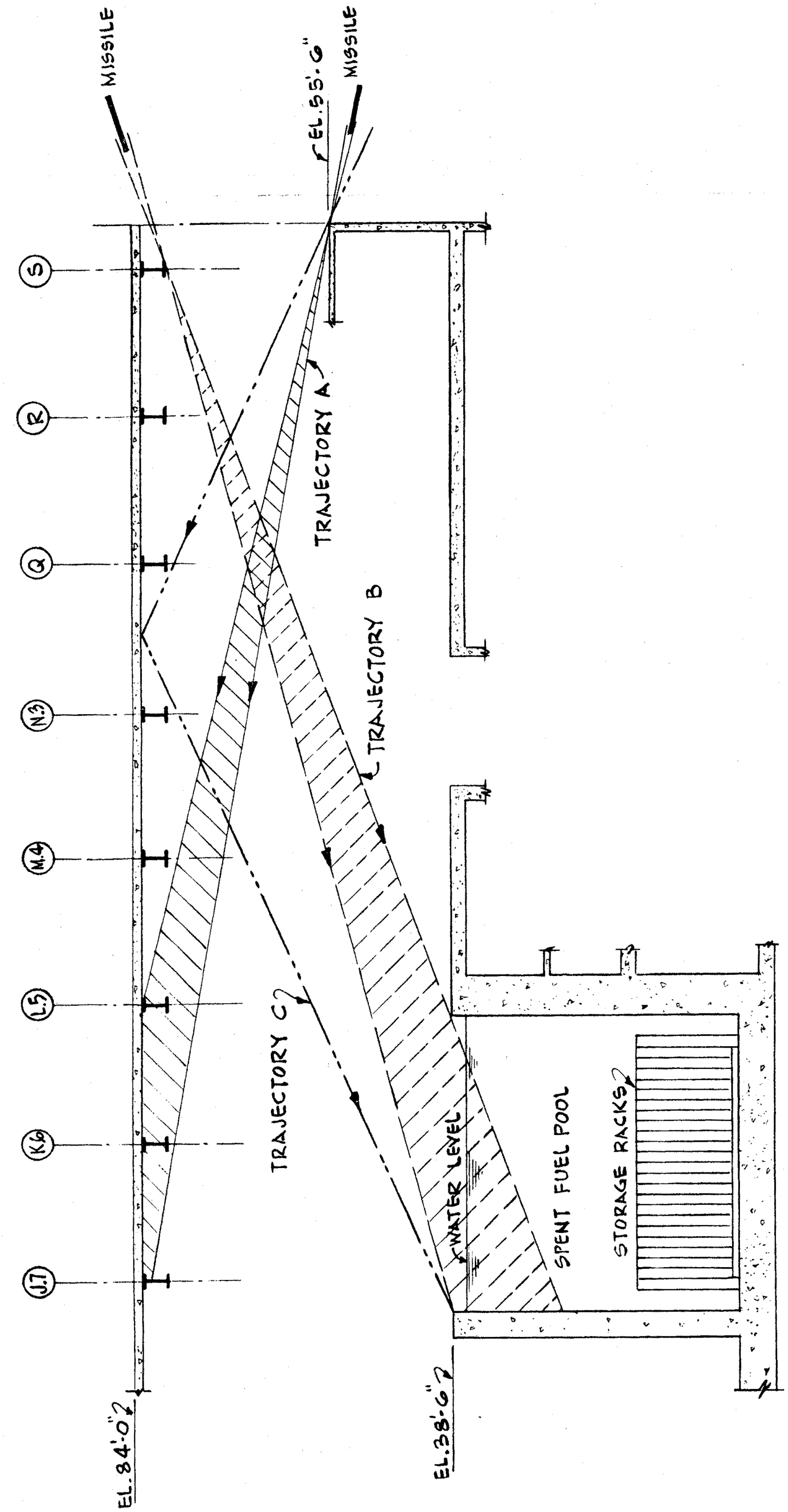


FIGURE 5.4–2 MISSILE RESISTANT SIDING DETAIL

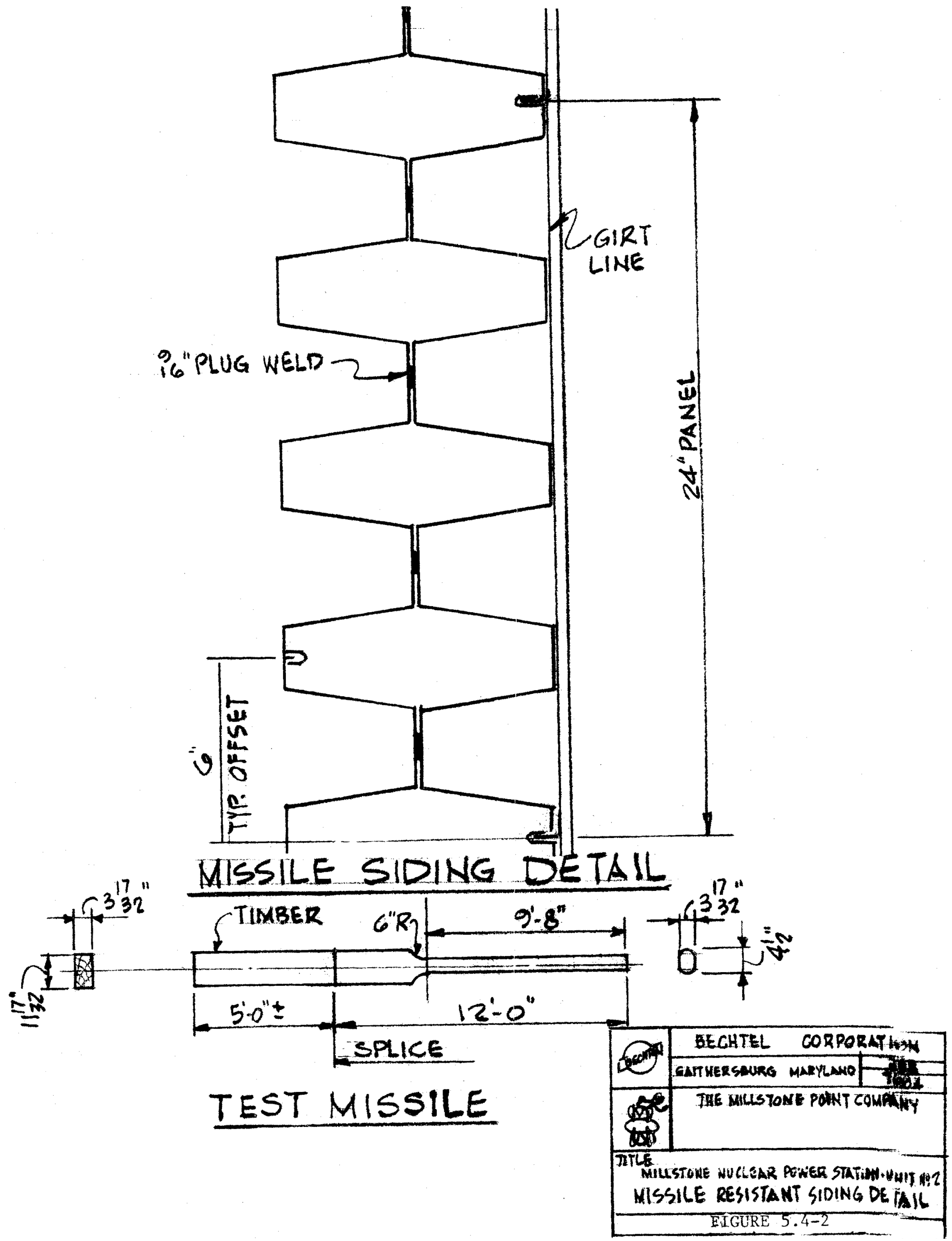
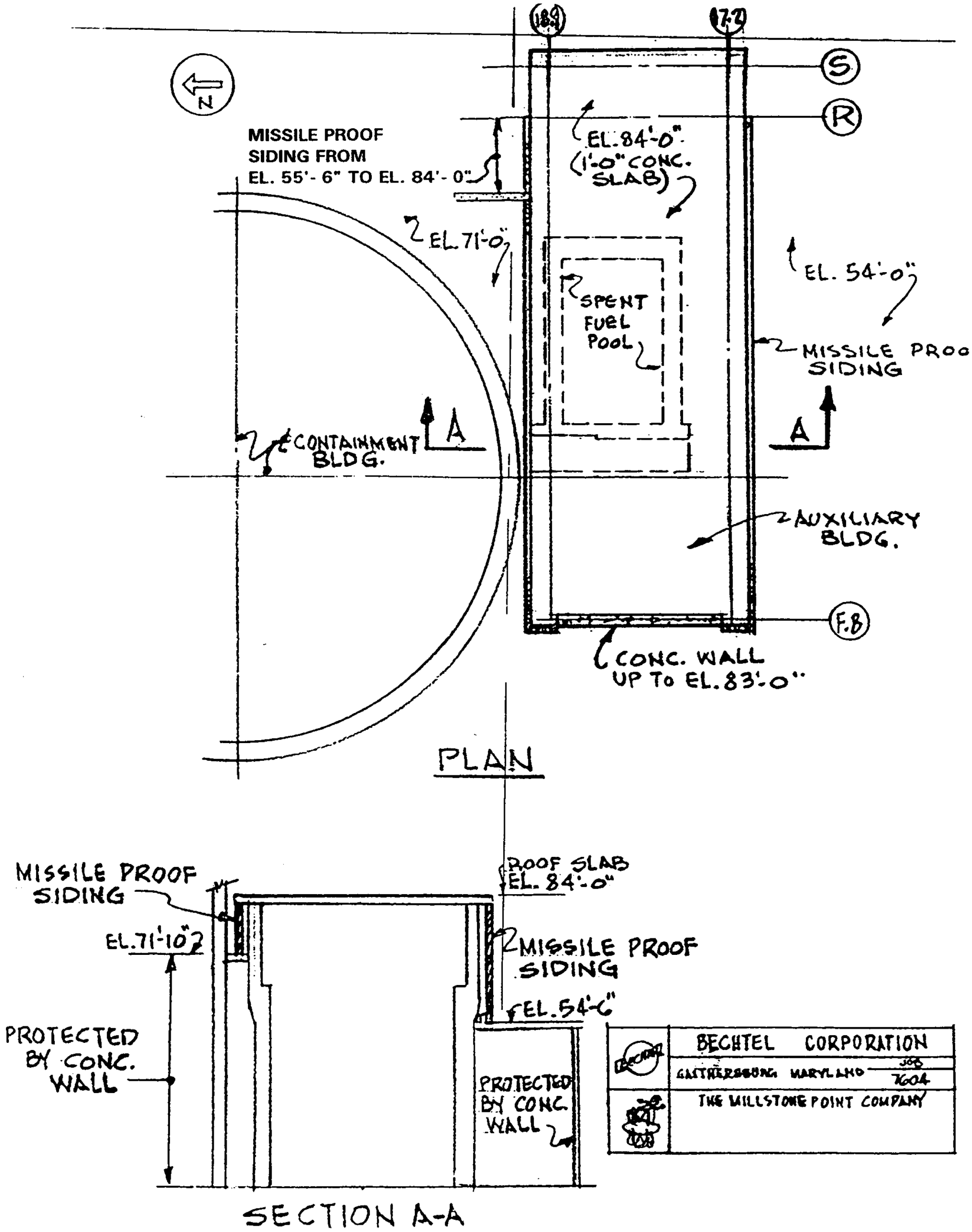


FIGURE 5.4–3 LOCATION OF MISSILE RESISTANT SIDING



5.5 TURBINE BUILDING

5.5.1 GENERAL DESCRIPTION

The turbine building is a rigid framed steel structure with metal siding and precast concrete panels on the exterior. Blowout panels are located on the east wall, column line “E” on the upper portion of the metal siding. The foundations for the frame are spread footings bearing on lean concrete backfill which extends to rock. The turbine-generator pedestal is a low-tuned mass concrete structure which is also founded on lean concrete backfill which extends to rock.

The pedestal is separated from the surrounding floor slabs by teflon lined sliding bearings as indicated on Figure 5.5–1. The turbine building main frame is connected to the Unit 1 turbine building by sliding connections so that it is an independent structure.

As shown on Figures 1.2–3, 1.2–4, and 1.2–5, the heater bay running between lines E and E.5, separates the turbine building from the auxiliary building. This bay is connected to the auxiliary building for lateral support, but is separated from the turbine building main frame by sliding connections as indicated on Figure 5.5–2. Sections through the turbine building are shown on Figures 1.2–15 and 1.2–16.

The Unit 1 and Unit 2 turbine buildings are on the same centerline so that the crane runway system is continuous through both units. A 195 ton overhead crane operates on the runway system. The crane is designed to meet the loading requirements of the applicable portions of Crane Manufacturers Associated of America (CMAA) Specification 70.

5.5.2 CONSTRUCTION MATERIALS

The following materials are used in the construction of the turbine building:

a. Structural and Miscellaneous Steel

Rolled shapes, plates and bars	ASTM A-36
Crane rails	Bethlehem Steel
High strength bolts	ASTM A-325 or A-490

b. Reinforcing Steel

Column ties in turbine pedestals	ASTM A-615 Grade 40
All other deformed bars	ASTM A-615 Grade 60

c. Concrete, 28 day strength (psi)

Turbine pedestals, operating floor slabs, and column footings	4000
All other concrete	3000

d. Interior coatings (Original Construction)

Concrete and masonry surfaces in switchgear room

Floor	Keeler & Long Number 7107 Epoxy Grey

e. Interior maintenance coatings

Coating materials used in the turbine building represent current day technology and comply with current day environmental requirements regarding volatile organic compound (VOC) content and hazardous material constituents such as lead, asbestos, and hexavalent chrome. Specific products are selected to provide service performance comparable to or improved over that of the original coating materials. Coating colors are chosen to help maintain a safe working environment.

5.5.3 DESIGN BASES

The design of the turbine building provides the required features as outlined in Criteria 2, 3, 4, 5, Appendix A of 10 CFR Part 50.

5.5.3.1 Bases for Design Loads

Although only a portion of the turbine building houses Class I equipment and components, the entire structural system is designed for a Seismic Class I loading.

The design loads imposed on the structure are as follows:

a. Dead loads

b. Live loads

c. Thermal loads

d. Earthquake loads

e. Wind and tornado loads

f. Crane loads

5.5.3.1.1 Dead Loads

These loads consist of the weight of all structural materials, including all partitions, hangers, trays, pads, and pedestals. Equipment dead loads are those specified on the drawings supplied by the manufacturers of the various equipment installed within the building.

5.5.3.1.2 Live Loads

These loads consist of design floor loads, tank liquid weights, piping loads, and equipment live loads specified on the drawings supplied by the manufacturers of the various equipment installed within the building.

A snow load of 60 psf is applied to the exposed roof over the area housing Class I equipment or components, and 40 psf for all other exposed roof area.

5.5.3.1.3 Thermal Loads

Expansion and contraction in structural members due to changes in temperature are considered. Provisions for normal expansion and contraction are made by the use of slotted connections as required.

5.5.3.1.4 Earthquake Loads

Seismic analysis is as defined in Sections 5.2.2.1.5 and 5.8.

5.5.3.1.5 Wind and Tornado Loads

Wind loads for the turbine building are determined on the basis of the ASCE Paper 3269, "Wind Forces on Structures," using the highest wind velocity at the site for a 100 year recurrence period. The ASCE paper is used mainly to determine the shape factors. Based upon the site location and the structure classification, the design wind and velocity is taken to be 115 mph with gusts up to 140 mph.

The turbine building is analyzed for tornado loads on the following basis:

a. Differential bursting pressure between the interior and exterior of the structure is assumed to be 3 psi pressure occurring in three seconds (1 psi/second), followed by a calm for two seconds and a re pressurization.

b. Lateral loads on the structure are based on a tornado funnel which is conservatively assumed to have a peripheral tangential velocity of 300 mph and a translational velocity of 60 mph. These velocities are added together, resulting in a design basis tornado wind velocity of 360 mph. The applicable portions of the wind design methods described in the ASCE paper are used, particularly for the shape factors. The provisions in the paper for gust factors and variation of wind velocity with respect to height are not applied. The wind velocity is assumed to be uniformly distributed over the height of the structure.

c. A tornado driven missile as defined in Section 5.2.5.1.2.

With the exception of the missile impact area, the allowable stresses to resist the effects of tornadoes are 90 percent of the yield strength of the reinforcing steel and 85 percent of the ultimate strength of the concrete.

A discussion of the probability of tornado occurrence is presented in Section 2.3.

5.5.3.1.6 Crane Loads

These loads include the dead and live loads of the turbine building crane.

5.5.3.2 Design Load Combinations

To ensure the structural integrity of the turbine building, the allowable stresses specified in the 1963 AISC "Manual of Steel Construction," and the ACI-318-63 "Building Code Requirements for Reinforced Concrete," are used. These allowable stresses are increased by 33-1/3 percent for the 115 mph base wind loads and the operating basis earthquake loads. For the tornado wind and the design base earthquake, the allowable stresses are 90 percent of the yield strength of the reinforcing, and 85 percent of the ultimate strength of concrete.

The following load combinations are considered:

a. $D + L + C_D + C_L$

b. $D + L + C_D + W_w$

c. $D + L + C_D + W_T$

d. $D + C_D + L + E$

e. $D + C_D + L + E'$

Where

D = dead loads

L = live loads

C_D = crane dead loads

C_L = crane live loads

W_w= wind loads (115 mph base)

W_t = tornado loads (360 mph base)

E = operating basis earthquake

E' = design basis earthquake

The design loads and stresses for all concrete walls and columns are within the allowable values for each wall or column. Design stresses are not applicable for the columns inasmuch as the interaction diagrams are used to proportion reinforcing and concrete required to support a given load with its corresponding eccentricity. Exterior walls having vertical and lateral loads are designed for bending and axial loadings and the resulting combined stresses are kept within the code allowables. There are numerous walls, slabs, columns, and beams within the building, each element of which was designed for the pertinent loading. Design stresses for the various components are recorded in the design calculations. The allowable loading, or combined loading, depends on the reinforcing which was added and varies considerably depending upon the applied conditions.

The following loading combinations and reinforcing ratios (of moment/axial force and shear) shown in Table 5.5-1 governed the design.

5.5.3.3 Structural Analysis

The main frame of the turbine building and the turbine generator pedestal are designed for the load combinations stated in Section 5.5.3.2 using working stress methods.

5.5.3.3.1 Seismic Analysis

Analysis of the turbine building for the effects of an earthquake is performed in accordance with Section 5.8.

These earthquake loads are superimposed on the other structural loads to obtain the loading combinations as stated in Section 5.5.3.2.

5.5.3.3.2 Wind and Tornado Analysis

The design wind loads on the turbine building are a function of the kinetic energy per volume of the moving air mass. The product of one-half of the air density and the square of the resultant design velocity results in a pressure corresponding to the design wind.

Determination of the design wind pressure on the structure is in accordance with the ASCE Paper 3269, “Wind Forces on Structures.”

The pressure corresponding to the standard air at 0.07651 pcf at 15°C and 760 mm of mercury in terms of the velocity at the appropriate height zone is given by:

$$q = 0.002558\ V^2$$

Similarly, the design pressure, including the effect of the shape coefficient (C_d), is given by:

$$p = q \times C_d = 0.002558V^2C_d$$

Using these equations and the wind velocities given in Section 5.5.3.1.5, the wind forces are calculated for the main frame and are then applied to the structure.

Class I systems and equipment are protected from a tornado missile by structural walls and slabs.

TABLE 5.5-1 MAXIMUM ACTUAL STRESSES - TURBINE BUILDING

Design Component	Loading Combination	Moment/Axial Force As required ≤ 1.0 As provided	Shear Actual ≤ 1.0 Allowable
Footing slab	C	0.92	0.98
Substructure walls	C	1.00	0.84
Operating floor	C	0.94	0.80
Superstructure walls	D	0.81	0.79
Roof	Tornado Missile Protection Requirements		

FIGURE 5.5–1 JOINT DETAIL AT TURBINE PEDESTAL

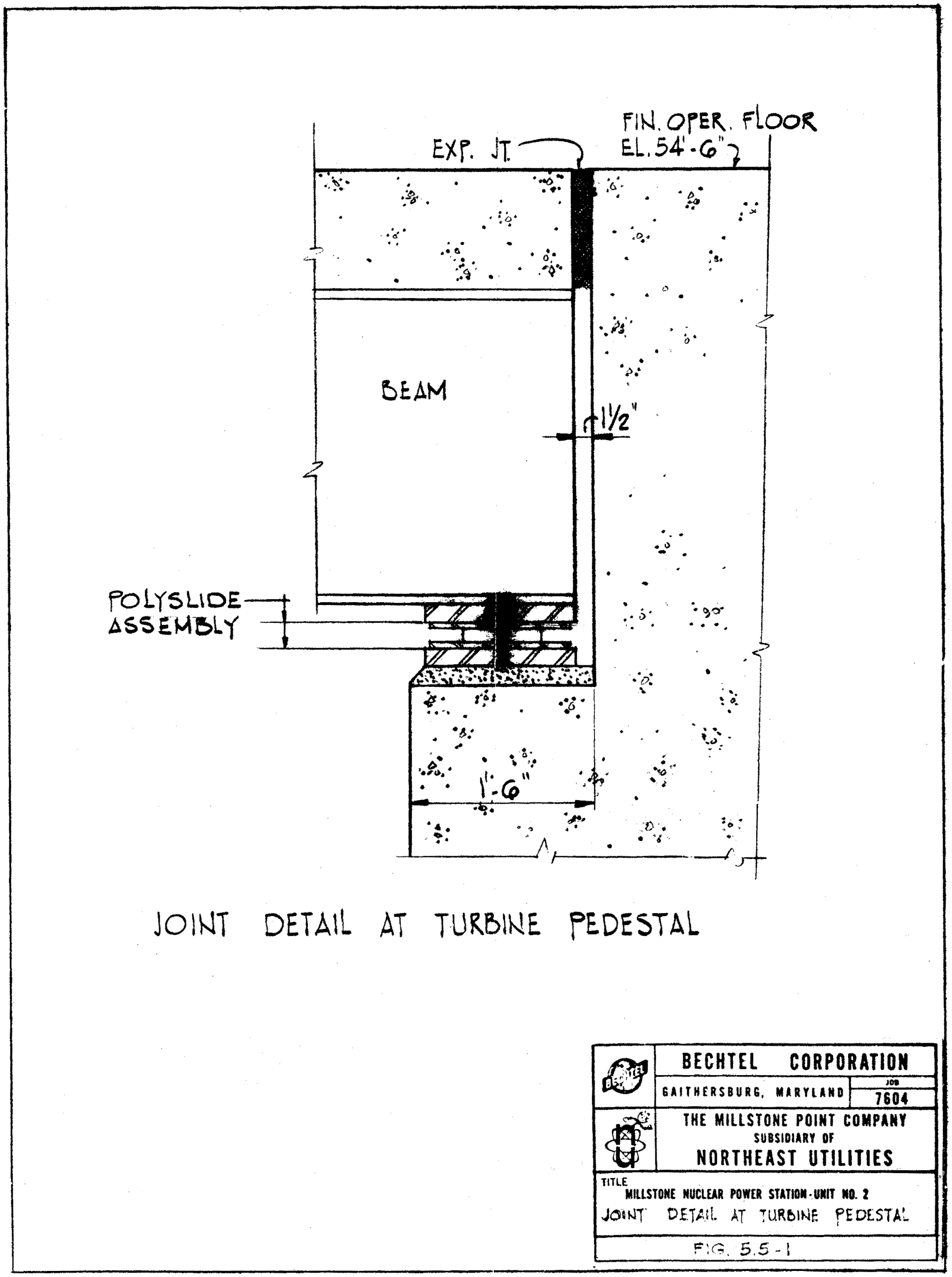
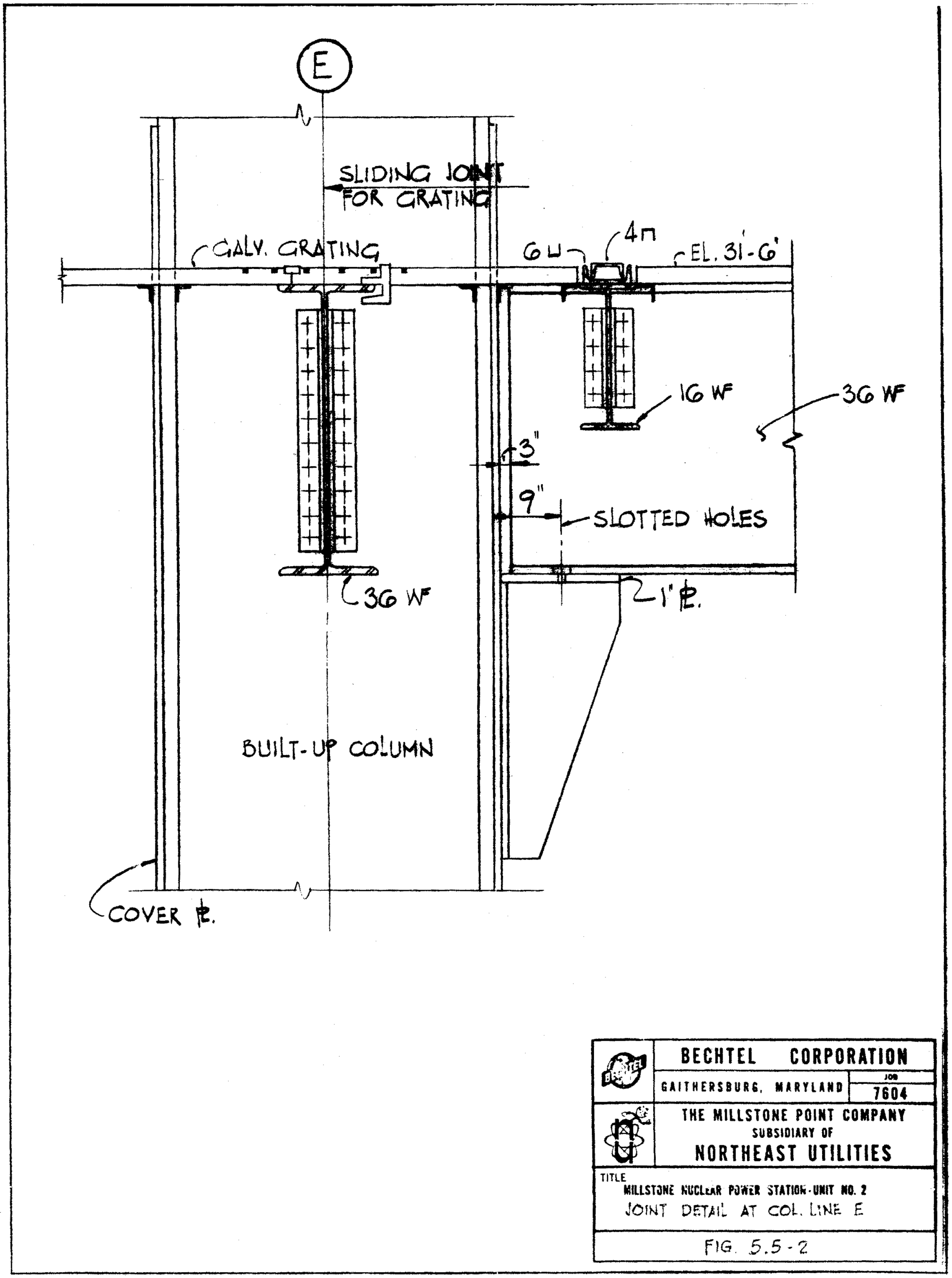


FIGURE 5.5–2 JOINT DETAIL AT COL. LINE E



5.6 INTAKE STRUCTURE

5.6.1 GENERAL DESCRIPTION

The intake structure, located west of the main plant, is a reinforced concrete structure founded on bedrock. It houses four circulating water pumps which supply water from Niantic Bay to the condensers positioned under the turbine-generator. Also located in the structure are three service cooling water pumps for the closed cooling water system. Access to these pumps is provided through hatches with removable covers.

The design of the intake structure incorporates several features which will ensure the safe, continuous operation of the circulating water system. These items are:

a. Trash racks and traveling screens which protect the pumps and condensers from foreign bodies present in the water supply.

b. A cutoff wall which extends 10 feet below the minimum water level to prevent the ecologically rich surface water from entering the system.

c. Passages provided as exit routes for fish which enter below the cutoff wall.

d. Vertical guides in the sides of each intake channel to receive stop-logs so that individual channels may be drained.

e. A monorail and trolley provided to service the interior of the intake structure.

An adjacent Class II building, which houses the chlorination equipment, is isolated from the intake structure by a joint filled with compressible material.

General layouts of the intake structure and circulating water system are shown on Figures 5.6–1 and 5.6–2, respectively.

5.6.2 CONSTRUCTION MATERIALS

The following materials are used in the construction of the intake structure:

a. Structural and miscellaneous steel

Rolled shapes, plates and bars	ASTM A-36
High strength bolts	ASTM A-325 or A-490

b. Reinforcing steel ASTM A-615 Grade 60

c. Concrete, 28 day strength (psi) 4000

d. Interior coatings (Original Construction)

Surfaces below elevation 14–0	Woolsey antifouling paint

Carbon steel	Epoxy

e. Interior maintenance coatings

Coating materials used in the intake structure represent current day technology and comply with current day environmental requirements regarding volatile organic compound (VOC) content and hazardous material constituents such as lead, asbestos, and hexavalent chrome. Specific products are selected to provide service performance comparable to or improved over that of the original coating materials. Coating colors are chosen to help maintain a safe working environment.

5.6.3 DESIGN BASES

The design of the intake structure provides the required features as outlined in Criteria 1, 2, 3, 4, 5, 44, 46, Appendix A of 10 CFR Part 50.

5.6.3.1 Bases for Design Loads

The intake structure is designed for all credible conditions of loadings including loads from normal operation and those due to adverse environmental conditions. The following loadings are considered.

a. Dead loads

b. Live loads

c. Earthquake loads

d. Lateral pressure loads

e. Wind and tornado loads

f. Hurricane wave loads

5.6.3.1.1 Dead Loads

These loads consist of the weight of all structural materials including partitions, hangers, trays, and pads. Equipment dead loads are those specified on the drawings supplied by the manufacturers of the various types of equipment installed within the building.

5.6.3.1.2 Live Loads

These loads consist of the design floor loads, tank liquid weights, piping loads, and equipment live loads specified on the drawings supplied by the manufacturers of the various types of equipment installed within the building.

A snow load of 60 psf is applied to the exposed roof.

5.6.3.1.3 Earthquake Loads

These loads are as defined in Section 5.8.

5.6.3.1.4 Lateral Pressure Loads

The lateral pressure loads include the active and the passive soil pressures where applicable. The buoyant and lateral forces of the displaced water are also considered.

5.6.3.1.5 Wind and Tornado Loads

Wind loads on the intake structure are determined on the basis of the ASCE Paper 3269, "Wind Forces on Structures," using the highest wind velocity at the site for a 100 year recurrence period. The ASCE Paper is used mainly to determine the shape factors. Based upon the site location and the structure classification, the design wind velocity is taken to be 115 mph with gusts up to 140 mph.

The intake structure is analyzed for tornado loads on the following basis:

a. Differential bursting pressure between the interior and exterior of the structure is assumed to be 3 psi pressure occurring in three seconds (1 psi/second), followed by a calm for two seconds and a re pressurization.

b. Lateral loads on the intake structure are based on a tornado funnel which is conservatively assumed to have a peripheral tangential velocity of 300 mph and a translational velocity of 60 mph. These velocities are added together, resulting in a design basis tornado wind velocity of 360 mph. The applicable portions of the wind design methods described in the ASCE Paper 3269 are used, particularly for the shape factors. The provisions in the paper for gust factors and variation of wind velocity with respect to height are not applied. The wind velocity is assumed to be uniformly distributed over the height of the structure.

c. A tornado-driven missile as defined in Section 5.2.5.1.2.

With the exception of the missile impact area, the allowable stresses to resist the effects of tornadoes are 90 percent of the yield strength of the reinforcing steel and 85 percent of the ultimate strength of the concrete.

A discussion of the probability of tornado occurrence is presented in Section 2.3.

5.6.3.1.6 Hurricane Wave Loads

These loads, resulting from a hurricane wave striking the front of the intake structure, are considered.

5.6.3.2 Design Load Combinations

The structural integrity of the intake structure is ensured by using the allowable stresses as specified in the 1963 AISC Manual of Steel Construction, and the ACI-318-63, “Building Code Requirements for Reinforced Concrete,” for the various loading combinations. A 33-1/3 percent increase in the allowable stresses is permitted for the combined stresses involving wind, earthquake, or tornado.

The following load combinations are used in the design:

a. $D + L$

b. $D + L + W_t$

c. $D + L + E'$

d. $D + L + W + H$

where:

D = dead loads

L = live loads

W_t = tornado loads

W = wind loads

E' = design basis earthquake

H = hurricane wave loads

The design loads and stresses for all concrete walls and columns are within the allowable values for each wall or column. Design stresses are not applicable for the columns inasmuch as the interaction diagrams are used to proportion reinforcing and concrete required to support a given load with its corresponding eccentricity. Exterior walls having vertical and lateral loads are designed for bending and axial loadings and the resulting combined stresses are kept within the code allowables. There are numerous walls, slabs, columns, and beams within the buildings, each element of which was designed for the pertinent loading. Design stresses for the various components are recorded in the design calculations. The allowable loading, or combined loading, depends on the reinforcing which was added and varies considerably depending upon the applied conditions.

The maximum combined stress ratio in the building frame is 0.98, resulting from an axial stress of 6.75 ksi and a bending stress of 25.1 ksi. This stress occurs under loading combination c above.

5.6.3.3 Structural Analysis

The intake structure is designed and analyzed using the working stress method.

5.6.3.3.1 Seismic Analysis

The analysis of the intake structure subjected to seismic loads is performed in accordance with the method described in Section 5.8.

These earthquake loads are superimposed on the other structural loads to obtain the loading combination as stated in Section 5.6.3.2.

5.6.3.3.2 Wind and Tornado Analysis

The design wind loads on the intake structure are a function of the kinetic energy per volume of the moving air mass. The product of one-half of the air density and the square of the resultant design velocity results in a pressure corresponding to the design wind.

Determination of the design wind pressure on the intake structure is in accordance with the ASCE Paper 3269, "Wind Forces on Structures."

The pressure corresponding to the standard air at 0.07651 pcf at 15°C and 760 mm of mercury in terms of the velocity at the appropriate height zone is given by:

$$q = 0.002558\ V^2$$

Similarly, the design pressure, including the effect of the shape coefficient (Cd), is given by:

$$p = q \times C_d = 0.002558 V^2\ C_d$$

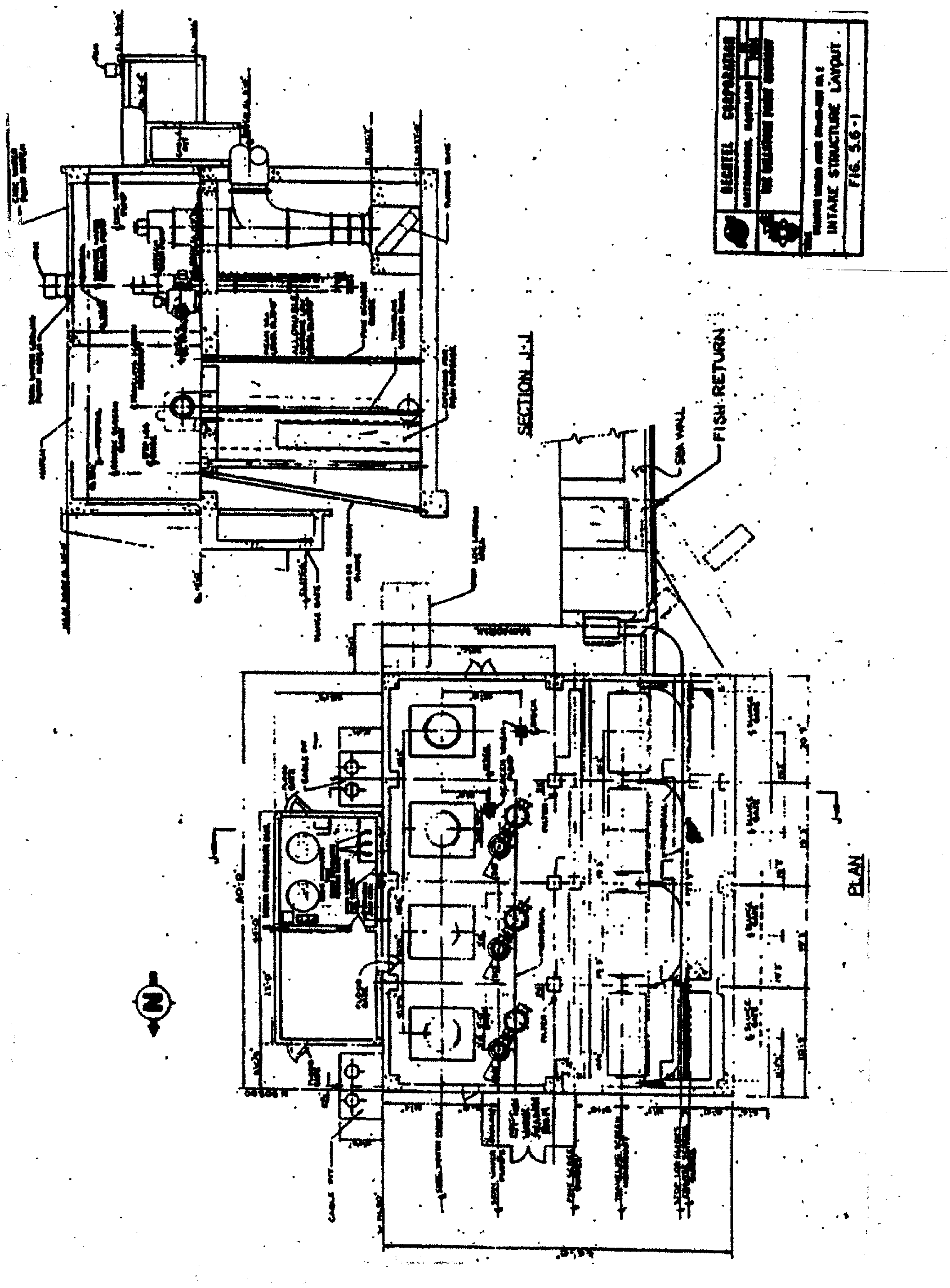
Using the equations given and the wind velocities noted in Section 5.6.3.1.6, the wind forces are calculated and then applied to the structure.

For the tornado loading condition, the hatches over the circulating water pumps and the traveling screens serve as blowout panels to relieve the pressure differential. Additional vents are also placed in the west wall and roof.

5.6.3.3.3 Hurricane Wave Analysis

The maximum hurricane wave is determined by the use of the U. S. Coastal Engineering Research Center Paper, "Shore Protection Planning and Design," 1966. The static and dynamic forces from this wave are applied to the front of the intake structure.

FIGURE 5.6–1 INTAKE STRUCTURE LAYOUT



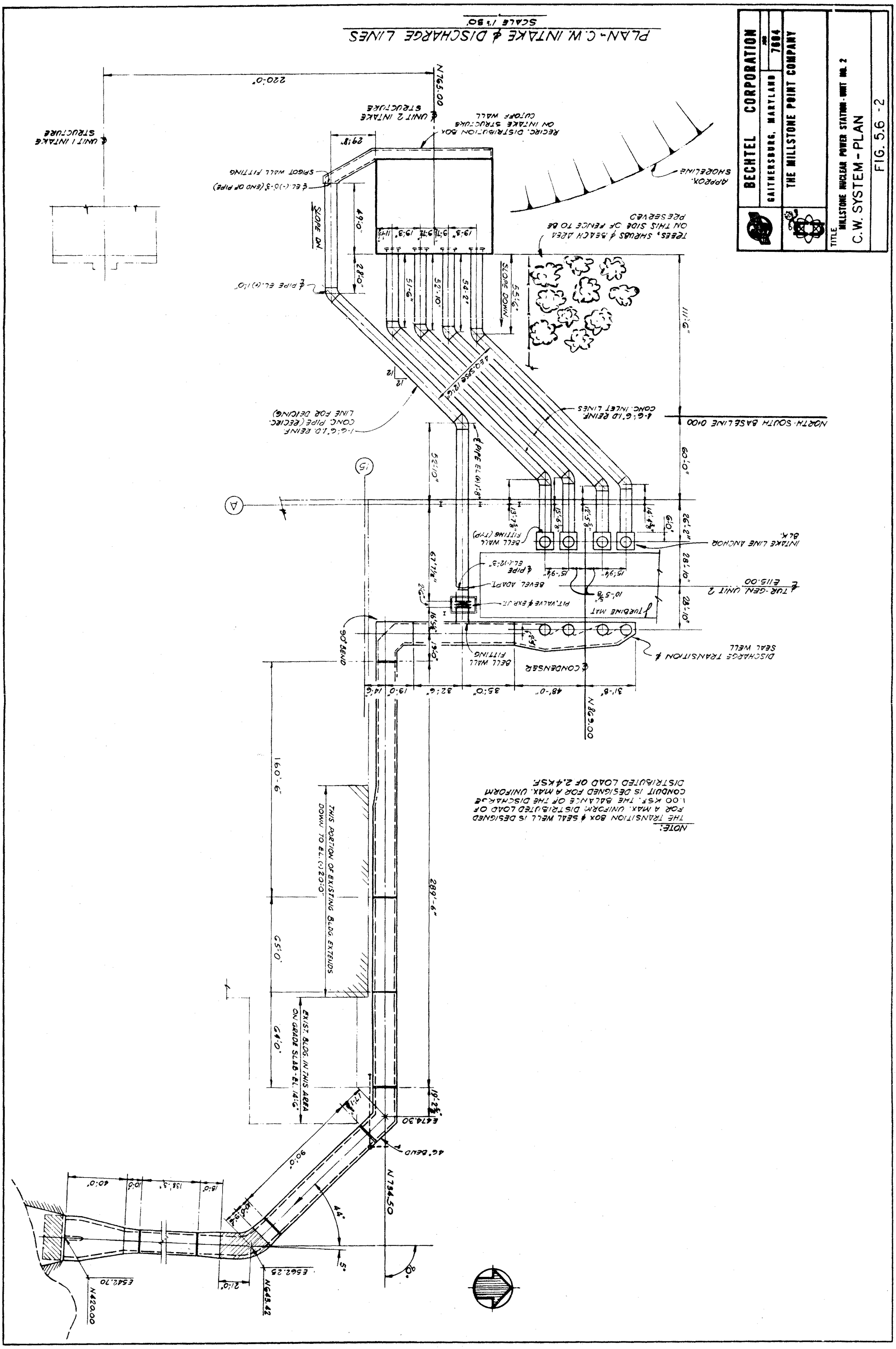


FIGURE 5.6–2 C. W. SYSTEM - PLAN

5.7 EXTERNAL CLASS I TANKS

5.7.1 GENERAL DESCRIPTION

The following tanks in the yard are classified as Seismic Class I structures:

a. Refueling water storage tank

b. Condensate storage tank

The condensate storage tank is located to the northwest, and the refueling water storage tank is located to the northeast of the containment, respectively, as shown in Figure 1.2–2. These tanks are supported on concrete foundations which rest on compacted structural backfill. The backfill is compacted to 95 percent of the maximum density as obtained by the Modified Proctor test in accordance with ASTM D-1557. The structural concrete foundations are proportioned such that the applied contact pressure, from dead loads and live loads in each design loading combination which result in uniform long-term settlement, does not exceed 3000 psf. The underlying supporting material for the condensate storage tank and refueling water storage tank is undisturbed glacial till and compacted glacial till, respectively. Allowable soil bearing pressures of 12000 psf for undisturbed glacial till and 5000 psf for compacted glacial till are utilized in the foundation design. These allowable values may be increased by one-third for wind and seismic conditions.

5.7.2 CONSTRUCTION MATERIALS

Structural concrete for the foundations conforms to the requirements of Section 5.9.3.1 and has a design strength of 3,000 psi at 28 days. Reinforcing steel conforms to Specification ASTM A-615, Grade 60.

The refueling water storage tank is fabricated from stainless steel conforming to ASTM A-240, Type 304. Design and fabrication are in accordance with Section III of the ASME Code.

The condensate storage tank was originally designed in accordance with the American Water Works Association Standard, AWWA D100. It is fabricated from carbon steel which conforms to Specification ASTM A-285, Grade C. The applicable portions of API 650, “Welded Steel Tanks for Oil Storage,” API 620, “Recommended Rules for Design and Construction of Large, Welded, Low Pressure Storage Tanks,” ACI 318-89, “Building Code Requirements for Reinforced Concrete,” ACI 349-80, “Code Requirements for Nuclear Safety Related Concrete Structures,” and 1980 ASME Section III, Division I, Subsection NE for Class MC were used to design the structural modifications necessary to support the addition of a nitrogen blanketing system.

5.7.3 DESIGN BASES

The design of the external Class I tanks provides the required features as outlined in Criteria 1, 37, Appendix A of 10 CFR Part 50.

5.7.3.1 Bases for Design Loads

The external Class I tanks are designed for all credible combinations of loading including loads under normal operation and loads due to adverse environmental conditions. The following loadings are considered:

a. Dead loads

b. Live loads

c. Earthquake loads

d. Wind and tornado loads

5.7.3.1.1 Dead Loads

These loads consist of the weight of all structural materials.

5.7.3.1.2 Live Loads

Live loads consist of tank liquid weights and a snow load of 60 psf applied to the roofs for the condensate storage tank, the live load included the internal operating pressure band of +1.0 psig to -0.34 psig which resulted from the addition of a nitrogen blanketing system.

5.7.3.1.3 Earthquake Loads

These loads are defined in Section 5.8.

5.7.3.1.4 Wind and Tornado Loads

Wind loads for the external Class I tanks are determined on the basis of the ASCE Paper 3269, "Wind Forces on Structures," based upon a design wind velocity of 115 mph with gusts up to 140 mph.

The condensate storage tank is analyzed for tornado loads on the following basis:

a. Differential bursting pressure between the interior and exterior of the structure is assumed to be 3 psi pressure occurring in three seconds (1 psi/sec), followed by a calm for two seconds and a re pressurization.

b. Lateral loads on the structure are based on a tornado funnel which is conservatively assumed to have a peripheral tangential velocity of 300 mph and a translational velocity of 60 mph. These velocities are added together, resulting in a design basis tornado wind velocity of 360 mph. The applicable portions of the wind design methods described in the ASCE paper are used, particularly for the shape factors. The provisions in the paper for gust factors and variation of wind velocity with respect to height are not applied. The wind velocity is assumed to be uniformly distributed over the height of the structure.

c. Provision is made for protection of the condensate storage tank against tornado driven missiles as defined in Sections 5.2.5.1.2 and 10.4.5.3 so as to provide sufficient water for a safe shutdown.

5.7.3.2 Design Load Combinations

The load combinations considered in the design of these tanks are listed below:

a. $D + L$

b. $D + L + W_w$

c. $D + L + W_t$ (applicable to CST only)

d. $D + L + E$

e. $D + L + E'$

Where

D = dead loads

L = live loads

W_w = wind loads (115 mph base)

W_t = tornado loads (360 mph base)

E = operating basis earthquake

E' = design basis earthquake

Stresses from load combination a, b, c, and d do not exceed the allowable permitted by the ASME Code. Standard Review Plan criteria were used for the condensate storage tank modifications which resulted from the addition of a nitrogen blanketing system. Stresses from load combination e do not exceed the allowable stresses of 90 percent of the steel yield strength and 85 percent of the concrete ultimate strength.

5.8 SEISMIC DESIGN

5.8.1 INPUT CRITERIA

The response spectrum technique is used to analyze Class I structures, systems and equipment when they are subjected to seismic motion. The response spectrum technique assumes a constant damping factor for each mode of the model of the major structural elements. The input ground motion is expressed in terms of a smooth design spectrum curve associated with the damping factor. Small equipment, as well as piping and cables located within the structure, are neglected in the model due to their relatively insignificant masses.

For the design of the piping systems and equipment, design spectrum curves at each of their supports are generated from a synthetic time history ground motion.

5.8.1.1 Design Response Spectra

The operating basis earthquake (OBE) used in the design of this plant is based on a ground motion having a maximum horizontal ground acceleration of 0.09 g and a vertical ground acceleration of 0.06 g, acting simultaneously. For the safe shutdown earthquake (SSE), a maximum horizontal ground acceleration of 0.17 g and a vertical ground acceleration of 0.11 g are used. The design response spectrum curves for structures supported on rock are shown on Figures 5.8–1 and 5.8–2, and the design response spectrum curves for structures supported on compacted structural backfill are shown on Figures 5.8–3 and 5.8–4.

A synthetic time history whose response spectrum curve corresponds to the design response spectrum curve is used to generate the response spectrum curves at different elevations within the structure. These are used in analyzing Class I equipment and piping at the respective locations. Comparisons of the response spectra derived from the time history and site seismic design response spectra for the damping values of 0.5, 1.0, 2.0, and 5.0 in percent of critical damping are shown in attached Figures 5.8–5, 5.8–6, 5.8–7 and 5.8–8.

The system period intervals at which the spectra values are calculated are as follows:

Frequency Range (cps)	**Frequency Increment (cps)**
0.2 to 1	0.05
1 to 10	0.1
10 to 33	1.0

The frequency increment shown above is always less than 10 percent apart between two consecutive frequencies, except that for the first frequency range of 0.2 to 1.0 cps. To verify that the 0.05 is a sufficiently small increment for this frequency range, the 2 and 5 percent damping response spectra are computed at the smaller increment of 0.0125 cps, which is one-quarter of the original one. Figure 5.8–9 shows these spectra in comparison with those based on 0.05 cps frequency increment, and with the design spectra, indicating that the 0.05 cps increment is indeed sufficient for engineering purposes. Comparison of the response spectrum curve generated by the synthetic time history and the design response spectrum with two percent of critical damping is shown on Figure 5.8–5. A composite comparison of the design response spectrum curve with the N69W components of the 1952 Taft and the N-S components of the 1940 El Centro recorded earthquakes, normalized to the same ground acceleration of 0.09 g with two percent of critical damping, is shown on Figure 5.8–10.

5.8.1.2 Synthetic Time History

The synthetic time history is generated as follows:

The response of a set of linear single degree-of-freedom systems to seismic action is governed by the equation:

$$\ddot{X} + 2\beta\omega_n\dot{X} + \omega_n^2 X = -\ddot{u}(t) \qquad \text{Eq. (1)}$$

where

$\ddot{X}$ = relative acceleration

$\dot{X}$ = relative velocity

X = relative displacement

β = percent of critical damping

ω_n = natural frequency

ü(t) = forcing acceleration time history as a function of time t

The acceleration spectra is defined by:

$$S_a = (X + u)_{max} = [\omega_d \int_o^t u\ (\tau)e - {}^{\beta w_n(t-\tau)}\ \sin\omega_d(t-\tau)d\tau]_{max} \qquad \text{Eq. (2)}$$

where $\omega_d = \omega_n(1-\beta^2)^{1/2}$

T = time variable

If S_a - is specified as an arbitrary function of ß and ω, it is desirable to determine the ü(t) that will produce the acceleration spectra.

Assume that the acceleration has the form:

$$\ddot{u}(t) = \sum_{i=0}^{M} \ddot{u}_i \cos(\omega_i t + \phi_i) \qquad \text{Eq. (3)}$$

where M = number of Fourier Series terms

The absolute magnitude of the response of a single degree-of-freedom system to a sinusoidal acceleration is represented by:

$$\left|\frac{X}{\ddot{u}_i}\right|_{max} = \frac{1/\omega_n^2}{\left\{\left[1 - \frac{\left(\omega_i^2\right.}{\omega_n^2}\right]^2 + \left[2\beta\frac{\omega_i}{\omega_n}\right]^2\right\}^{1/2}} \qquad \text{Eq. (4)}$$

Assuming that:

$$\ddot{X} = -X\omega_n^2 \cos \omega_n t \qquad \text{Eq. (5)}$$

The following is obtained:

$$S_{\ddot{a}} = \left(\frac{\ddot{x}}{\ddot{u}} + 1\right)\ddot{u} = \sum_{i=0}^{M}\left\{1 + \frac{1}{\left\{\left(1 - \frac{\omega_i^2}{\omega_n}\right)^2 + \left(2\beta\frac{\omega_i}{\omega_n}\right)^2\right\}^{1/2}}\right\}\ddot{u}_i \qquad \text{Eq. (6)}$$

Equation (6) will be satisfied by solving $\ddot{u}_i$ for the unknown coefficients. It is assumed that the maxima will occur at the same time. A matrix equation is set for frequencies:

$$\omega_{i+1} = 1.05\omega_i \qquad i = 1, 2, ...143 \qquad \text{Eq. (7)}$$

where $\omega_1 = 0.1$ cps

The damping values are as stated in Table 5.8-1.

Once a solution vector has been determined from:

$$\ddot{u} = A^{-1}S_a \qquad \text{Eq. (8)}$$

and substituted into Equation (3), the maximum acceleration spectra is formed by numerically integrating Equation (1). The maximum acceleration spectra, Sa_i, is then compared to the originally assumed spectra, $S\ddot{a}_o$. Weighting functions are developed from the relationship:

$$\varepsilon_i = f(\omega_i) = \frac{S\ddot{a}_o}{S\ddot{a}_i} \qquad \text{Eq. (9)}$$

Thus, from the nth iteration the following is obtained:

$$\ddot{u}_{n+1,\dot{c}} = \ddot{u}_{n,i}\ \varepsilon_{n,i} \qquad \text{Eq. (10)}$$

An acceleration time history has been developed using this procedure. It has been determined that for all frequencies considered, the bounds of Sa predicted by Equation (2) exceeded a 10 percent variation at certain frequencies.

5.8.2 SOIL-STRUCTURES INTERACTION

5.8.2.1 Soil-Foundation Interaction

The outlines of the foundations for Millstone Unit 2 structures are shown on Figures 5.8–11 to 5.8–12. These structures are supported by different materials.

The structures which are supported on bedrock are:

a. Containment

b. Enclosure building

c. Auxiliary building (except as noted below)

d. Intake structure

e. Turbine building (except as noted below)

The following structures are supported on compacted structural backfill:

a. Warehouse portion of the auxiliary building.

b. Auxiliary feedwater pump foundations located in the auxiliary bay of the turbine building.

Soil-foundation interaction is considered by introducing equivalent springs and viscous dashpots for the supporting mediums while the foundations are assumed to be rigid. The horizontal translational and rocking effects on structures are represented by equivalent spring stiffness. The horizontal translational and rocking modes have been checked for structures founded on rock (i.e., with dynamic soil modulus greater than 500 x 10^3 ksf). The rigidity of the rock is so much greater than the structures it supports that rocking does not occur. The deflection patterns are such that there is no slope at the bases of the structures. Hence, for structures supported on rock, fixed-base assumption is used in the mathematic models formulated for the structural analysis.

For structures resting on compacted structural backfill, the equivalent spring stiffnesses for soil are evaluated using the formulas developed by Richart, Hall and Woods (Reference 5.2-61).

Isolation joints are provided between the foundations of the main structures. The differential movement of adjacent structures due to seismic motion is evaluated, and the size of the isolation joint is based on the anticipated horizontal movement of the foundations during the operating basis earthquake and the safe shutdown earthquake. The isolation joint is filled with a compressible material to minimize the influence of the foundations of the main structures on each other.

5.8.2.2 Dynamic Soil Pressure on Structures

The horizontal earth pressures for the walls are evaluated for both the static and the dynamic conditions. The rigidity of the walls and the backfill that is placed after the walls are constructed and framed at the top do not allow sufficient movement for the development of the active earth pressure case. Therefore, the at-rest condition is developed.

An equivalent fluid pressure is derived for the soil subject to seismic motion. The earth pressures are determined based on the characteristics of the materials to be used for backfill from the site, grading, and pit excavation. The backfill used is sand and silty sand. The equivalent fluid unit weight above the water table is 55 lb/cubic feet. Below the water table equivalent fluid unit weight is 95 lb/cubic feet which includes both the water pressure and the lateral at-rest earth pressure. Pressure distribution is assumed to be hydrostatic.

The dynamic earth pressures are considered for this plant. The analysis is based on work by Newmark, Ishii, Terzaghi, and the U.S. Army Corps of Engineers. These references provide the pressure coefficients which depend upon the magnitude of the acceleration factor of the earthquake. Although there is uncertainty concerning the behavior of backfill during earthquakes, the dynamic earth pressures can be approximated by the methods outlined in these references. The horizontal earthquake acceleration is combined with the static earth pressure acting on the wall. The values of the dynamic pressures are dependent on the types of backfills. These references show that, for typical sandy or silty sand backfill materials, the dynamic earth pressures are equivalent to the static earth pressures plus the static earth pressures times 2 a, where “a” is the ratio between acceleration produced by an earthquake shock and gravitational acceleration. Based on this information, the dynamic earth pressure is found to be equal to 1.34 times the static earth pressure for the safe shutdown earthquake.

The maximum soil reaction includes stresses due to the structural weight, the maximum overturning moment from the lateral analysis and the maximum inertia force from the vertical component of earthquake. The absolute sum of stresses due to the above effects will not exceed the allowable bearing pressure for the soil. The overturning effect of the structure in a seismic event is considered negligible.

5.8.2.3 Underground Structures

Seismic analyses are performed on the following underground structures.

a. One 36 inch carbon steel off-gas pipe from the auxiliary building to the Millstone stack.

b. Two 24 inch cast iron headers from the Service Water System are routed from the intake structure to the auxiliary building.

c. One 10 inch carbon steel condensate water pipe from the condensate storage tank to the condenser.

d. Two electrical ducts encased in reinforced concrete from the intake structure to the turbine building.

The Class I underground ducts subjected to earthquake motion are analyzed biaxially, i.e., along the duct run and perpendicular to the duct run. Assuming that the ducts displace with the adjacent soil, the relative movements of the ducts to their supports will be determined. With these displacements, soil duct support models are formulated to determine the induced stresses in the ducts.

a. Due to wave propagation

b. At the supports due to differential movements of buildings and soil

c. At bends

5.8.3 SEISMIC STRUCTURAL ANALYSIS

5.8.3.1 Methods of Analysis

For seismic analysis of Class I structures, the response spectrum technique, using the design response spectrum curves, is employed. For Class I equipment analysis, response spectrum curves at the equipment bases are generated by the time history technique.

The procedure used to account for the number of earthquake cycles during one seismic event includes consideration of the number of significant motion peaks expected to occur during the event. The number of significant motion peaks during one seismic event would be expected to be equivalent in severity to no more than 40 full load cycles about a mean value of zero and with an amplitude equal to the maximum response produced during the entire event. Based upon this consideration and the assumption that seismic events equivalent to 5 Operating Basis Earthquakes will occur during the life of the plant, Class I systems, components and equipment are designed for a total of 200 full load cycles.

The seismic analyses performed for Class I structures are based on elastic and linear behavior of all components involved, and as such, do not include any gradual or accidental deterioration of the structure. The blowdown forces associated with a concurrent loss-of-coolant accident (LOCA) are computed separately and combined with the seismic loads.

There are removable concrete slabs located in the containment building and auxiliary building. These slabs are placed over low pressure radwaste equipment, such as filters and demineralizers, and weigh approximately 4,000 pounds. The slabs will not receive a seismic acceleration in the upward direction sufficient to cause the slab to become a missile.

Removable blocks are self-locking and contain staggered horizontal and vertical joints. These block panels are designed to remain in place by use of retainers during a safe shutdown earthquake.

5.8.3.2 Procedure for Analysis

5.8.3.2.1 Structural Responses

5.8.3.2.1.1 Response Spectrum Method

The seismic loads on the containment are determined from a dynamic analysis of the structure. The dynamic analysis is made on a mathematical model consisting of lumped masses and weightless elastic columns acting as spring restraints. It is performed in the following two parts:

a. Determination of the natural frequencies of the structure and its mode shapes.

b. Determination of the model responses of the modes to the earthquake motions by the response spectrum method.

The natural frequencies and mode shapes are computed from the equations of motion of the lumped masses established by a stiffness or displacement method. They are solved by the interaction techniques through the use of a computer program. The form of the equation is:

$$[K][\Delta]_n = \omega^2[M][\Delta]_n$$

where:

[K] = matrix of stiffness coefficients including the combined effects of shear, flexure, rotation and horizontal translation.

[M] = matrix of concentrated masses

[Δ] = matrix of mode shape

The computation results in several values of w_n and mode shapes $(\Delta)_n$ for n = 1, 2, 3---n, where n is the number of degrees of freedom (i.e., lumped masses) assumed in the idealized structure.

The response of each mode of vibration to the safe shutdown earthquake is then computed by the response spectrum technique, as follows:

a. The base shear contribution of the nth mode

$$V_n = W_n \delta_{an}(W_n \beta_n)$$

where

W_n = effective weight of the structure in the nth mode

$$W_n = \frac{(\Sigma_x \Delta_{xn} W_x)^2}{\Sigma_x \Delta_{xn}^2 W_x}$$

where the subscript, x, refers to the levels throughout the height of the structure, and

Δ_{xn} = mode shape for the mode under consideration

δ_{an} (ω_n, β_n) = spectral acceleration of a single degree-of-freedom system with a damping coefficient of β_n' obtained from the response spectrum.

ω_n = angular frequency of the nth mode.

b. The horizontal load distribution for the nth mode is then computed as:

$$Fx = \frac{V_n(\Delta_{xn} W_x)}{\Sigma_x \Delta_{xn} W_x}$$

Then, using the modal inertia force Fx, the shears V_x at each point for each mode are calculated. The moments for each mode are obtained by integrating the shear diagram of the structure from the top down if a cantilever model is used. For coupled systems, with interconnecting members between cantilever members, the modal inertia forces are applied at the mass points, and analysis is made to obtained shears and moments at each point. The design shears and moments for the structure are the absolute sums of modal shears and moments. The overturning moment of the structure is the moment obtained at the base of the model.

For structure founded on structural backfill, a rotational, a vertical and a horizontal spring are used to represent the soil properties. Formulas for computing the equivalent spring stiffnesses for the case of rectangular base mat are based on Richart, Hall and Woods. Reference for the above article is given in Section 5.8.2.1.

The smoothed ground response spectrum curves used as input for the response spectrum method are derived from the report of J.A. Blume and Associates. The report was presented in Appendix F of the Design and Analysis Report (AEC Docket Number 50 - 245) which is a supplement to the Millstone Unit 1 FSAR. The spectrum curves were derived based on a careful examination of available historical records in the vicinity of the site and the underlying soil condition. The spectrum curves provide maximum responses over the frequency range from 2 cps to 10 cps in which the natural frequencies of the structures fall. Therefore, modal period variation in the mathematical models for Class I structures due to variations in material properties would not result in any significant increase in the resultant seismic loads.

5.8.3.2.1.2 Time History Method

The floor response spectrum curves are generated using time history modal analysis. Consider a viscosity damped, multi-degrees of freedom system subjected to the base acceleration $\ddot{u}(t)$; the equation of motion is given by:

$$[M]\ddot{x} + [\beta]\dot{x} + [K]x = -[M]\ddot{u}(t)\Delta^{T} \qquad \text{Eq. (11)}$$

where

Δ^{T} is the unit vector

This equation can be uncoupled to a set of independent equations analogous to the equation for a single degree of freedom system. The multi-degrees of freedom system can then be defined simply in terms of its mode shapes, frequencies and mass distribution as follows:

$$M_1 \ddot{x}_i + 2\beta_1 \omega_1 M_1 \dot{x}_1 + \omega_1^2 M_1 x_1 = -[M]\ \ddot{u}(t)\Delta_1^T$$

$$M_n \ddot{x}_n + 2\beta_n \omega_n M_n \dot{x}_n + \omega_n^2 M_n x_n = -[M]\ \ddot{u}(t)\Delta_n^T$$

Any of these equations can be rearranged as:

$$\ddot{x}_n + 2\beta_n W_n \dot{x}_n + \omega_n^2 \chi_n = \frac{[M]\Delta^{T_n}\ddot{u}(t)}{M_n} \qquad \text{Eq. (12)}$$

This set of equations can then be integrated numerically and independently. The spectrum values at any mass point can be obtained by direct application of Equation 12 to the digitized earthquake record at equal time intervals of 0.01 second.

The following tabulation shows a comparison of maximum seismic accelerations at selected critical locations in Class I structures as computed by the response spectrum and time history methods. The results based on the methods of “sum of absolute values” and “square root of sum of squares” by response spectrum technique are both shown for comparison. From the table as shown below, it can be concluded that the results obtained by response spectrum and time history methods are consistent with each other and that the Class I structures are conservatively designed since the responses by the "sum of absolute values" were used.

STRUCTURE	MASS POINT	ACCELERATION (g's)		
		Response Spectrum: ABS	**Response Spectrum: SRSS**	**TME HISTORY**
Containment	7	0.265	0.170	0.192
Structure	14	0.425	0.306	0.393
Containment	4(N-S)	0.337	0.289	0.275
Internals	6(N-S)	0.457	0.356	0.329
Auxiliary	5(N-S)	0.274	0.250	0.296
Building	5(E-W)	0.285	0.254	0.325

5.8.3.2.2 Combination of Vertical and Horizontal Responses

The vertical ground design spectrum curves are derived as two-thirds of the horizontal values. This two-third value is considered to be conservative based on the strong motion records from both the United States and foreign countries.

Analyses for both the horizontal and vertical directions are performed using the ground design spectrum curves. The forces, moments and resulting stresses are combined directly, assuming a simultaneous occurrence of the vertical and horizontal motions. Vertical structural elements are considered vertically rigid. Horizontal structural elements of the Class I structures were further investigated for vertical responses and were found to be rigid.

The vertical ground response spectrum curve is used for equipment design. The equipment is attached to the rigid portions of the structure which have high natural frequencies. The ground motion would not be appreciably altered. The forces, moments and stresses on the equipment from both the vertical and horizontal motions are considered as acting simultaneously.

5.8.3.2.3 Torsional Effect Considerations

The torsional effect induced by the unsymmetric nature of the building was compensated for by considering a static torsional moment acting at the elevation under consideration. The magnitude of this moment is equal to the sum of the individual products of the inertia force and the eccentricity between the center of rigidity and center of gravity on and above that elevation.

To justify the above procedure, a torsional analysis was made on auxiliary building which is the least symmetric structure in this plant. A natural frequency of 10.4 CPS was obtained for the torsional mode compared to the natural frequencies of 4.1 CPS and 7.5 CPS obtained for the translational mode in two major axes. The torsional natural frequencies for the other buildings are expected to be higher than the translational natural frequencies. The justification of combining the uncoupled results is presented in Appendix C of the topical report BC-TOP-4, Seismic Analysis of Structures and Equipment for Nuclear Power Plants, Rev. 1, dated September, 1972, Bechtel Corporation.

5.8.3.2.4 Natural Frequencies and Response Loads

The natural frequencies, loads in the form of mode shapes and total response loads, and the response spectra at critical plant equipment elevations for the following structures are presented in the forms of graphs. (See Figures 5.8–14 through 5.8–61.)

1. Containment and Intervals
2. Auxiliary Building
3. Turbine Building
4. Warehouse
5. Intake Structure

5.8.3.3 Damping Values

Material damping values used in the seismic analyses of structures and systems are shown in Table 5.8-1.

For structures made of a single material, the damping values are selected from values listed in Table 5.8-1 for the material under consideration. For structures made up of composite materials, the damping values can be considered as a function of both the mass and the particular mode shape value to calculate the composite damping values. The following Mass Mode Weighting method is used:

$$\beta_c = \frac{\Sigma_{i=1}^{i=n} M_i|\phi_i|\beta_i}{\Sigma_{i=1}^{i=n} M_i|\phi_i|}$$

β_c composite damping

β_i damping associated with mass point

$|\phi_i|$ absolute value of the mode shape at mass point

M_i mass at mass point

The only structure composed of major subsystems that are made of different materials is the warehouse area of the auxiliary building. Results based on the energy method and Mass Mode Weighting method were compared and negligible difference was obtained in this case.

For a structure whose motion is primarily composed of translational (flexural) displacement and foundation rotation (rocking), the mode shape must be broken down into its translational and rotational components, denoted as ϕ_γ and ϕ_f, respectively. Since the rotation is due to the fact that the structure is supported on a flexible foundation, the foundation damping, denoted as β_γ will influence the total damping value. Denoting the damping of the structures material by β_f, the composite damping can be computed by the following equation:

$$\beta_c = \frac{\beta_\gamma\phi_\gamma + \beta_f\phi_f}{\phi_\gamma + \phi_f}$$

Since the only Class I structure supported on a flexible foundation is the warehouse portion of the auxiliary building, the above procedure is not used. A constant damping value of 2% is used for all the modes and is a conservative approach.

5.8.4 SEISMIC SYSTEM ANALYSIS

To determine the seismic response of equipment, a time history analysis is performed on the structural model, using an earthquake as the input ground motion. This analysis generates the floor acceleration time- histories at the various mass points at which the equipment is located. The equipment response spectrum curve is then generated for each of the floor acceleration time-histories at various damping values and is used in the design of the equipment.

The equipment response spectrum curves are broadened by a smooth curve extended 10 percent each way at the peak response associated with the natural frequencies of the structure. This measure reflects the expected variations in the natural frequencies of the structure due to variations in structural material properties.

To determine the piping and instrumentation responses to an earthquake, Class I seismic piping systems are analyzed dynamically by means of a three dimensional model using two-thirds horizontal ground response spectra for the vertical spectra. The valves are included in the model by means of lumped masses and eccentric moments arms to account for the torsional effects of valves in the seismic piping analysis. The locations of seismic supports and restraints for the piping system are determined so that the piping system will not be in resonance with the supporting structures. The induced seismic effects of Class II piping on Class I piping systems are also considered by including the Class II piping in the model. For each piping system, a horizontal response spectrum analysis was performed for the north-south and for the east-west directions. Modal responses are combined using the square-root-of-the-sum-of-the-squares method (except for RCS piping and components discussed in Appendix 4.A). The results of each analysis are combined with the results of excitation in the vertical direction. The design internal force or moment, or displacement is the larger number obtained from either of these analyses. The possible combined vertical and horizontal amplified response loads for the design of piping and instrumentation include the effects of the responses of building, floors, supports, equipment, and components.

The piping system is analyzed for the relative seismic displacements between piping supports, i.e., floors and components, at different elevations within a building and between buildings. Stresses in the piping system due to the most unfavorable directions of movements of supports are combined with thermal, seismic and operating stresses and used for piping design.

The locations of seismic supports and restraints for seismic Category I piping, piping system components, and equipment, including placement of snubbers and dampers are determined so that the resulting seismic stresses when combined with operating stresses will not exceed the allowable stresses given by governing codes.

A field surveillance is conducted to assure that the supports, restraints, etc. have been installed in the designated locations. Any change in location due to interference or other factors must be approved by engineers. For 2 inch and smaller Category I piping, a field installation manual is provided so that field engineers can properly design and locate pipe supports and restraints. Upon completion the design is reviewed by the engineers.

For seismic Category I buried piping, the pipe was assumed fixed at the end entering a structure and extending infinitely into the soil. The horizontal and vertical movements at the entry point, resulting from the seismic analysis of the structure, was then taken as end displacement in computing the stresses.

For seismic Category I piping outside the containment structure, extending from one structure to another, the differential movements at support points of the two structures were assumed to be out of phase. The resulting stresses when combined with thermal stresses are within allowable stresses.

5.8.5 SEISMIC EQUIPMENT ANALYSIS

For all purchased Class I equipment, the vendors are required to submit seismic calculations made in compliance with the equipment specification to demonstrate the capability of the equipment to satisfy the functional requirements under specified seismic conditions. Equipment is not released to operations without engineering approval of the calculations.

The supports for all Class I equipment are designed for the induced seismic forces. There are no significant gaps between the equipment and their supports, and, hence, they are not considered in the seismic analysis of the equipment.

The equipment hatch and personnel lock of the containment are seismic Class I equipment and are designed for the following accelerations (OBE):

Lock	Accelerations: Vertical (g)	Accelerations: Horizontal (g)
Equipment Lock	0.06	0.20
Personnel Lock	0.06	0.27

The acceleration values are multiplied by the normal operating weight of the hatch lock, or parts of the hatch lock, to obtain the horizontal and vertical components of the earthquake forces. Both horizontal and vertical earthquake components are considered acting simultaneously with normal operating loads, without exceeding code allowable stresses at a temperature of 120°F of the materials.

The earthquake forces due to the safe shutdown earthquake are obtained by multiplying the aforementioned accelerations by 1.95. The equipment hatch and personnel lock are designed to withstand the simultaneous action of design basis earthquake components and the accident loads, as stated in Section 5.2.2.3.4, at a temperature of 289°F, without exceeding material yield stresses and without loss of function.

For certain Class I systems and equipment, where analytical models and normal theory do not produce results of a significant confidence level, dynamic testing of prototypes or similar equipment is substituted to ensure functional integrity. Test data conform to one of the following:

a. Performance data of equipment which, under the specified conditions, have been subjected to equal or greater dynamic loads than those to be experienced under the specified seismic conditions.

b. Test data from previously testing comparable equipment which, under similar conditions, have been subjected to equal or greater dynamic loads than those specified.

c. Actual testing of equipment in accordance with one of the following methods:

    1. The equipment is subjected to an artificial time history response at the elevation of interest.

    2. The equipment is subjected to a sinusoidal excitation, sweeping through the desired range of significant frequencies, using input acceleration amplitudes for the forcing function which simulates the specified seismic conditions.

    3. The equipment is subjected to a transient sinusoidal motion synthesized by a pulse exciting a group of octave filters such that the response of the shaking table and the duration of loading simulates the artificial response spectrum curve at the elevation of interest.

The certified test data and results are required to be submitted for engineering approval.

All Class II components and equipment are sufficiently separated from Class I components and equipment so that the Class II components and equipment will not damage the Class I components and equipment under seismic conditions.

5.8.5.1 Static Tests

Supports for lightly loaded safety related components rely on friction to resist vertical and seismic force. These components are not subject to thermal cycling or mechanical vibration.

Reliance on friction as the sole means of restraining vertical and seismic forces has been verified by testing performed on safety related accumulator tanks.

Reliance on friction is considered appropriate, provided that the frictional forces include a margin of safety consistent with the appropriate design criteria for the structure.

5.8.5.2 STERI Evaluations

The STERI process, Seismic Technical Evaluation of Replacement Items, EPRI TR-104871 (Reference 5.8-1) may be used to demonstrate that seismically rugged or insensitive replacement items exhibit seismic performance equivalent to original items. STERI evaluations document that the seismic qualification status of the original items and host equipment is maintained by the replacement items.

5.8.5.3 GIP NARE Evaluations

As an alternative to the methods described in preceding paragraphs, the Seismic Qualification Utilities Group Generic Implementation Procedure Revision 3, “GIP-3” (Reference 5.8-2), as modified and supplemented by the U. S. Nuclear Regulatory Commission Supplemental Safety Evaluation Reports SSER Number 2 (Reference 5.8-3) and SSER Number 3 (Reference 5.8-4), may be used as an alternative to existing methods for the seismic design and verification of modified, new and replacement equipment classified as Seismic Class I (NARE).

Only those portions of GIP-3 which apply to the seismic design and verification of mechanical and electrical equipment, electrical relays, tanks and heat exchangers, and cable and conduit raceway systems shall be used. The other portions of the GIP are not applicable since they contain administrative, licensing, and documentation information which is applicable only to the USI A-46 program. Plant procedures provide detailed GIP-3 implementation guidance.

The GIP method should not be used on equipment systems for which seismic qualifications have been imposed or committed to IEEE 344-1975 as listed below.

- Reactor Coolant Pump Speed Sensing System (RCPSSS)

- Feedwater Regulating System and Feedwater Pump Speed Control System, specifically the SPEC 200 control equipment
- Auxiliary Feedwater Automatic Initiation System and Component Parts, except logic power supplies and in-containment mounted sensors
- Auxiliary Steam Line Break Detection/Isolation System (ASDI)
- Alternating Current Instrumentation and Control Components, specifically the inverter and static switches

5.8.6 SEISMIC INSTRUMENTATION PROGRAM

5.8.6.1 Conformance with NRC Requirements

Seismic instrumentation for Millstone Point Unit 2 is provided on the basis of the existing NRC requirements specified in Regulatory Guide 1.12, Revision 1, (Instrumentation for Earthquakes), Appendix A of 10 CFR 100 and upon the best available information on the ability of seismic instrumentation to predict plant responses to seismic motion.

5.8.6.2 Description of Program

The following instrumentation is used to measure plant response to earthquake motion:

a. Five triaxial time history accelerographs.

b. Four peak accelerographs.

c. One triaxial response spectrum recorder.

Number	**Accelerograph Location**
1	Elevation (-)24 feet 0 inches containment base slab at 215 degrees outside of the containment.
2	Elevation 75 feet 0 inches containment structure at 215 degrees outside of the containment.
3	Elevation 14 feet 6 inches warehouse area of the auxiliary building.
4	Elevation 18 feet 0 inches intake structure south wall.
5	Elevation 14 feet 6 inches ground level on pad 139 degrees southeast of condenser storage tank.

Accelerographs Numbers 1 and 2 represent two key locations in the model used in the containment seismic analysis. They are located outside the containment structure such that they are accessible for periodic servicing. Accelerograph Number 3 is located on the warehouse base slab founded on compacted backfill. Accelerograph Number 4 is located in the intake structure which has more than half of its structure below grade. Accelerograph Number 5 represents the free field accelerograph.

The five triaxial time history accelerographs are combined within the basic accelerograph system. It features central recording on magnetic tape cassettes with remote transducer unit and a separate triggering unit. Both the transducer and the triggering unit which normally remain dormant are connected to a recording and playback system in the control room. Upon a seismic event, the system is activated within 0.1 second. Once the trigger is activated by a seismic event, the operators are alerted by visual and audible alarms. Signals from the transducer unit are fed into a multi-channel tape recording unit and recorded on a time/history basis with a common time signal. The recording system, once triggered, will continue to operate for at least five seconds beyond the last detection of a seismic signal of triggering intensity. At the end of the seismic event, the recorded tapes are transferred to a strip chart through a playback system. The entire event, from seismic trigger to visual accelerograph can be accomplished within a few minutes after an earthquake.

The transducer unit contains three accelerographs mounted in a triaxial orthogonal array 90 degrees apart. Sensitivity of the accelerograph is 0.001g to 1g with a natural frequency of at least 33 Hz.

The triggering unit which is located in the same area as the Number 1 accelerograph will activate all triaxial time history accelerograph and the recording system. The triggering unit generates a triggering signal by either an omni-directional horizontal or vertical component of a seismic acceleration. Trigger sensitivity is adjustable from 0.005g to 0.02g for the vertical trigger and the horizontal trigger has an adjustable gap from 0.005 inches to 0.06 inches. The trigger unit is engineered to discriminate against false starts from other operating inputs such as from traffic, elevators, people, and rotating equipment. The triggering unit is initially set to trigger at 0.01g vertical acceleration and 0.02 inch gap for the horizontal displacement. At this level no damage can be done in the plant and no spurious triggering of the recording system is expected. Triggering levels are determined on the basis of plant operating experience to ensure proper operation of this system.

Number	Recorder Location
1	Elevation (-)24 feet 0 inches containment base slab. (Outside Containment).
2	Elevation (-)0 feet 7 inches Steam generator Number 1 support.
3	Elevation 14 feet 6 inches Pressurizer support.
4	Elevation 38 feet 6 inches Safety injection tank support.

The peak accelerographs are all located on the major Class I equipment which has designed response spectrum readily available for comparison with the recorded acceleration level in the event of an earthquake.

The peak accelerographs are the Teledyne Geotech model PRA-103. These are mechanical devices which detect and record peak amplitudes of acceleration caused by seismic disturbances.

Number	Spectral Recorder Location
1	Elevation (-)24 feet 0 inches containment base slab. (Outside Containment)

This location corresponds to the pertinent input vibratory motion assumed in the containment seismic analysis.

The triaxial response recorder is an Engdahl Model PSR 1200 Peak Shock Recorder. This is a mechanical device which records the peak acceleration experienced at each of twelve frequencies for three mutually orthogonal directions. This device contains metal reeds which vibrate when excited at their natural frequencies. The maximum excursion of each reed is scribed onto a plate by a stylus mounted at the reed end. This plate is removed after a seismic occurrence and the length of the scribed line is measured and converted to its corresponding peak acceleration. The twelve reeds are resonant at the following frequencies:

Reed Number	Frequency (Hertz)
1	2.02
2	2.54
3	3.20
4	4.02
5	4.92
6	6.02
7	8.08
8	10.2
9	12.7
10	16.2
11	20.6
12	26.1

The above values should be considered nominal values since the resonant frequency of these reeds may vary from calibration to calibration.

5.8.6.3 Action Following an Earthquake

Activation of the seismic trigger on the accelerograph system will be annunciated in the control room.

The operator will verify the operational status of the plant by means of Control Room instrumentation and will initiate a visual inspection of the plant.

If the operator identifies abnormal or emergency conditions, he carries out appropriate procedures.

Following an earthquake, the operator will retrieve the recordings made by the peak recording spectral accelerograph for comparison of this data with the OBE for the plant site. The recordings from the strong motion accelerographs will also be analyzed to determine accurately the actual seismic acceleration spectrum experienced. The results of this analysis will be considered accurate, will override any preliminary indication by the peak shock spectral recorder, and by comparison with the OBE will determine whether the plant should continue to operate, be shutdown, or resume operations, as required by Appendix A of 10 CFR 100.

Following an earthquake of sufficient magnitude to shut the plant down, an extensive program will then be performed to evaluate the adequacy of all safety related structures, systems and equipment. The data on the earthquake's frequency and amplitude recorded by the strong motion accelerographs will be translated into computer codes best available at that time. For structure and system analyses using response spectrum techniques, the designed spectrum will be compared with that developed by the recorded time history. If the measured responses are less than the values used in the design for the SSE, the structure and system are considered adequate for future operations. Otherwise, the structure and system will be analyzed to check their adequacies. For system analysis such as the NSSS system using the time history technique, the recorded time history will be used as a direct input. Time histories at different elevations will be generated using the structure model. The fundamental frequency of the containment will be verified by the two accelerographs installed on the containment structure. Results will be evaluated to check the adequacy of the system.

5.8.7 REFERENCES

5.8-1 Seismic Technical Evaluation of Replacement Items, Ref: EPRI TR-104871

5.8-2 Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment, Revision 3, Seismic Qualification Utilities Group, May 16, 1997.

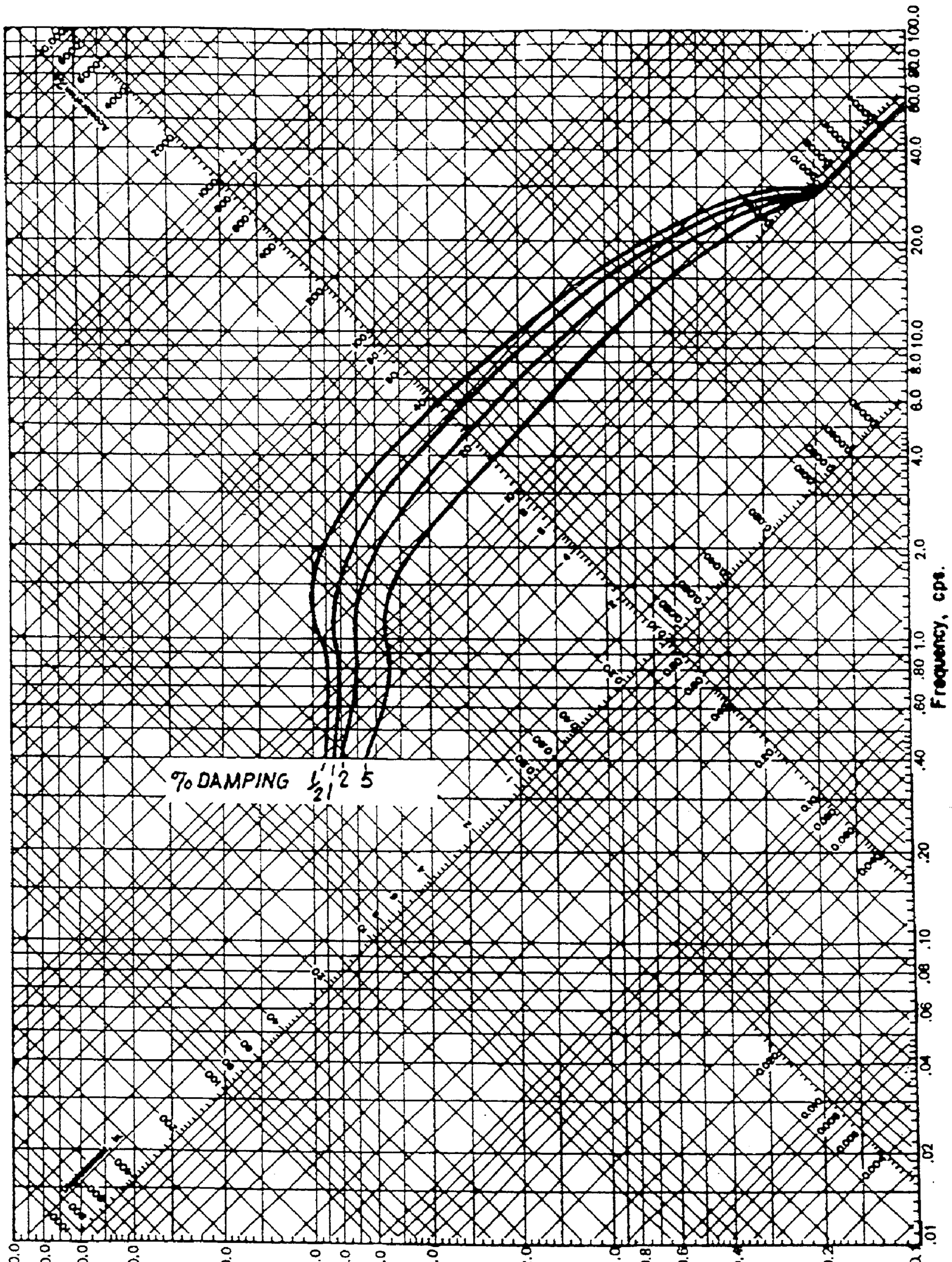
5.8-3 U.S. NRC Supplemental Safety Evaluation Report Number 2 (SSER Number 2) on SQUG Generic Implementation Procedure, Revision 2, as corrected on February 14, 1992 (GIP-2). May 22, 1992.

5.8-4 U.S. NRC Supplemental Safety Evaluation Report Number 3 (SSER Number 3) on the Review of Revision 3 to the Generic Implementation Procedure for Seismic Verification of Nuclear Power Plant Equipment Updated May 16, 1997 (TAC Number M93624), December 4, 1997.

TABLE 5.8-1 MATERIAL DAMPING VALUES

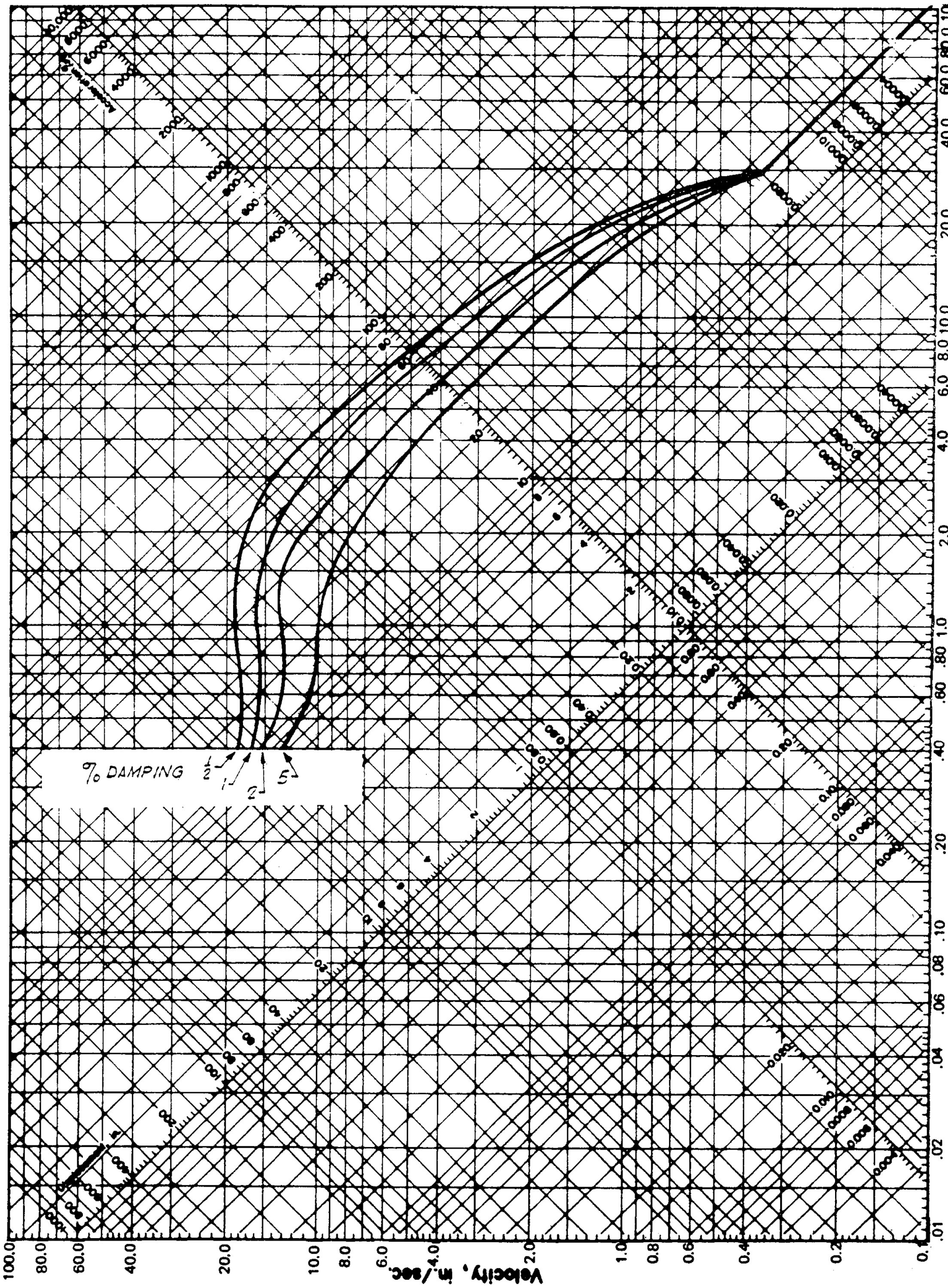
	Critical Damping	
	OBE (0.09 g ground acceleration)	**DBE (0.17 g ground acceleration)**
Welded steel plate assemblies	1	1
Welded steel framed structures	2	2
Bolted or riveted steel framed structures	2.5	2.5
Reinforced concrete equipment Supports	2	3
Reinforced concrete frames and buildings	3	5
Prestressed concrete structures	2	5
Steel piping	0.5	0.5
Soil (foundation)	2	5

FIGURE 5.8–1 RECOMMENDED DAMPED RESPONSE SPECTRA 9%G ACCELERATION, OPERATING BASIS EARTHQUAKE



RECOMMENDED DAMPED RESPONSE SPECTRA 9% g ACCELERATION
OPERATING BASIS EARTHQUAKE
FIGURE 5.8 -1

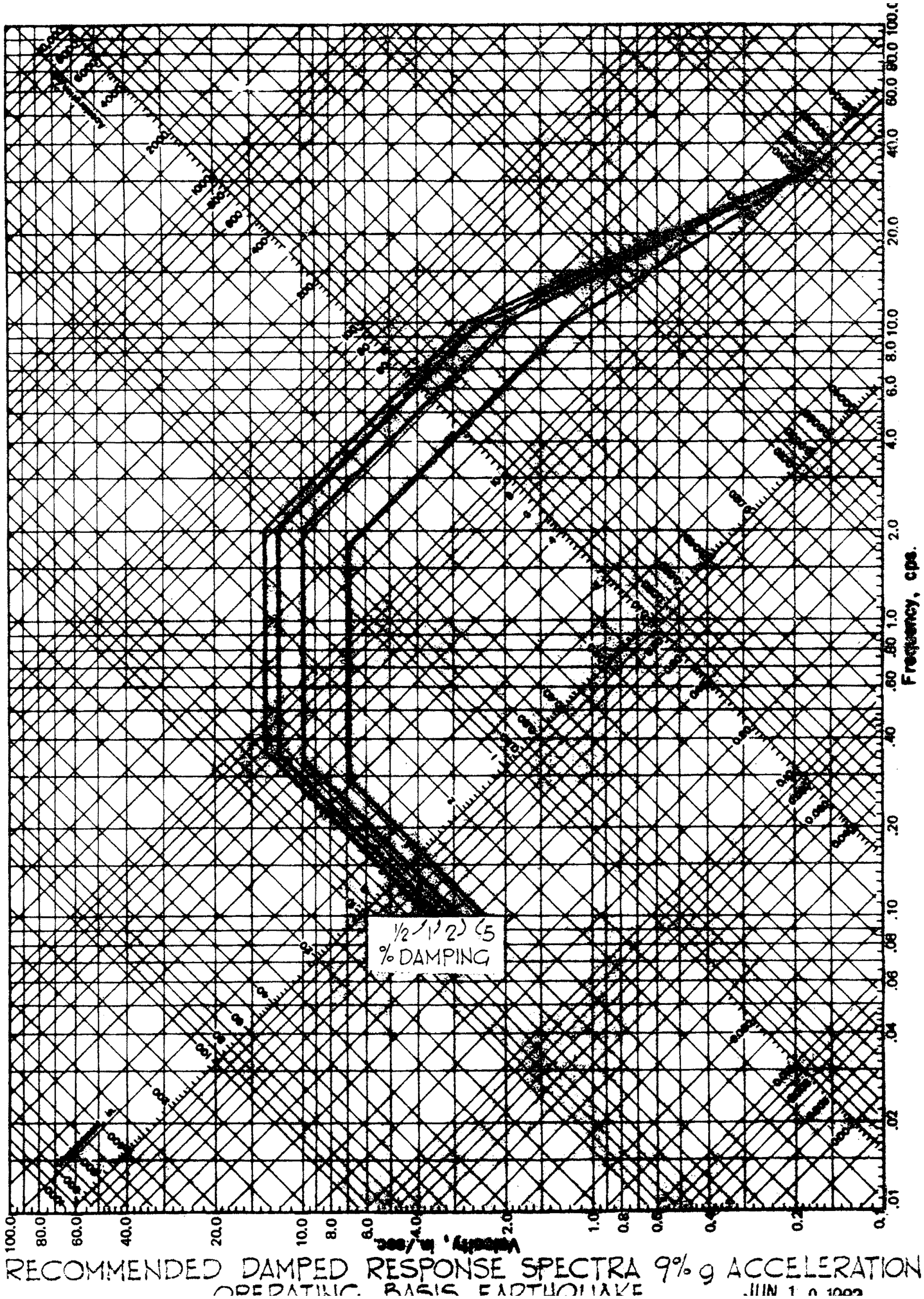
FIGURE 5.8–2 RECOMMENDED DAMPED RESPONSE SPECTRA 17%G ACCELERATION, DESIGN BASIS EARTHQUAKE



RECOMMENDED DAMPED RESPONSE SPECTRA 17% g ACCELERATION
DESIGN BASIS EARTHQUAKE

FIGURE 5.8-2

FIGURE 5.8–3 RECOMMENDED DAMPED RESPONSE SPECTRA 9%G ACCELERATION, OPERATING BASIS EARTHQUAKE (SOIL SURFACE)



RECOMMENDED DAMPED RESPONSE SPECTRA 9% g ACCELERATION OPERATING BASIS EARTHQUAKE

FIGURE 5.8–4 RECOMMENDED DAMPED RESPONSE SPECTRA, 17%G ACCELERATION, DESIGN BASIS EARTHQUAKE (SOIL SURFACE)

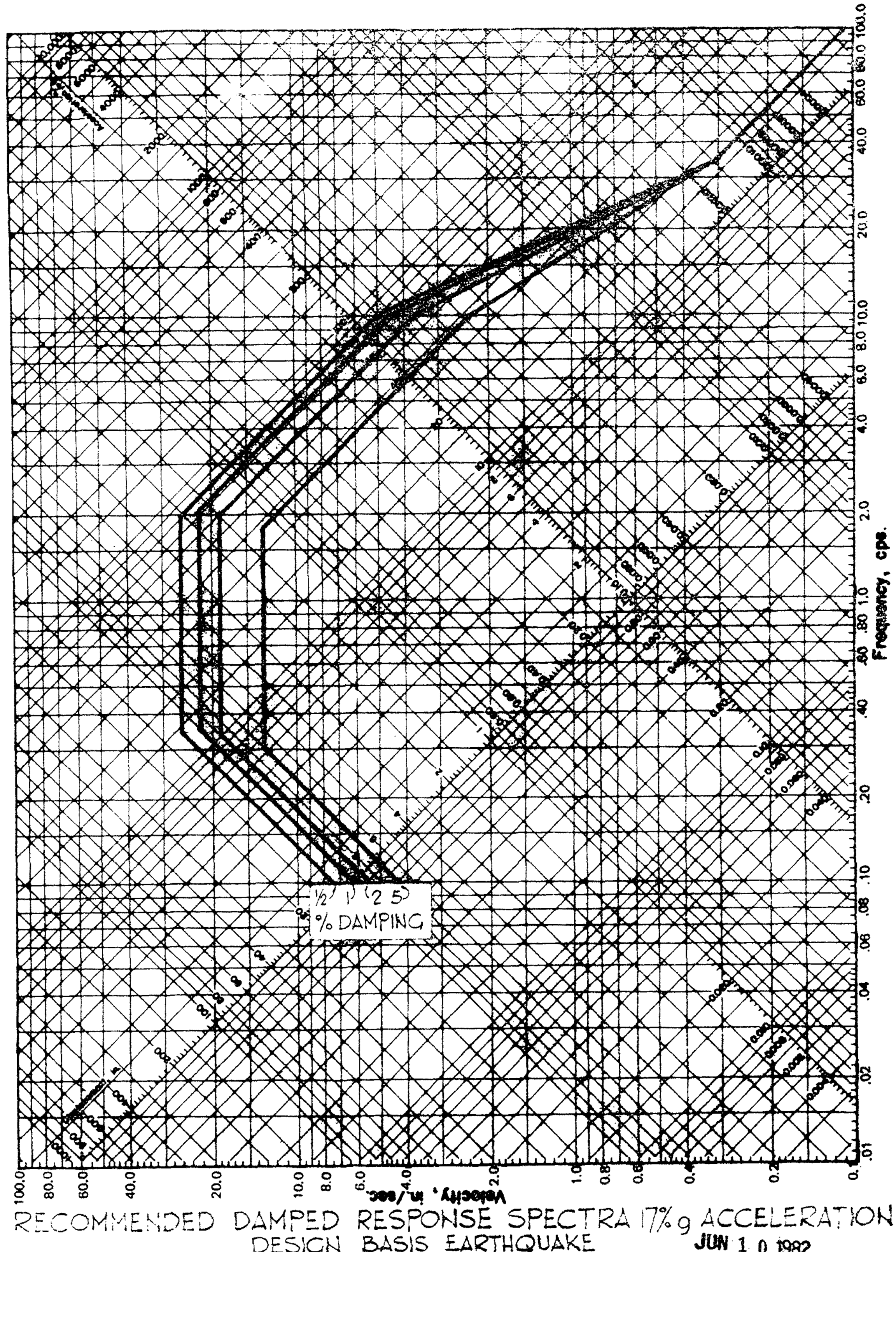


FIGURE 5.8–5 COMPARISON OF SMOOTH RESPONSE SPECTRA VS. RESPONSE SPECTRA FROM THE ANAMET TIME HISTORY DESIGN EARTHQUAKE (CRITICAL DAMPING = 0.5%)

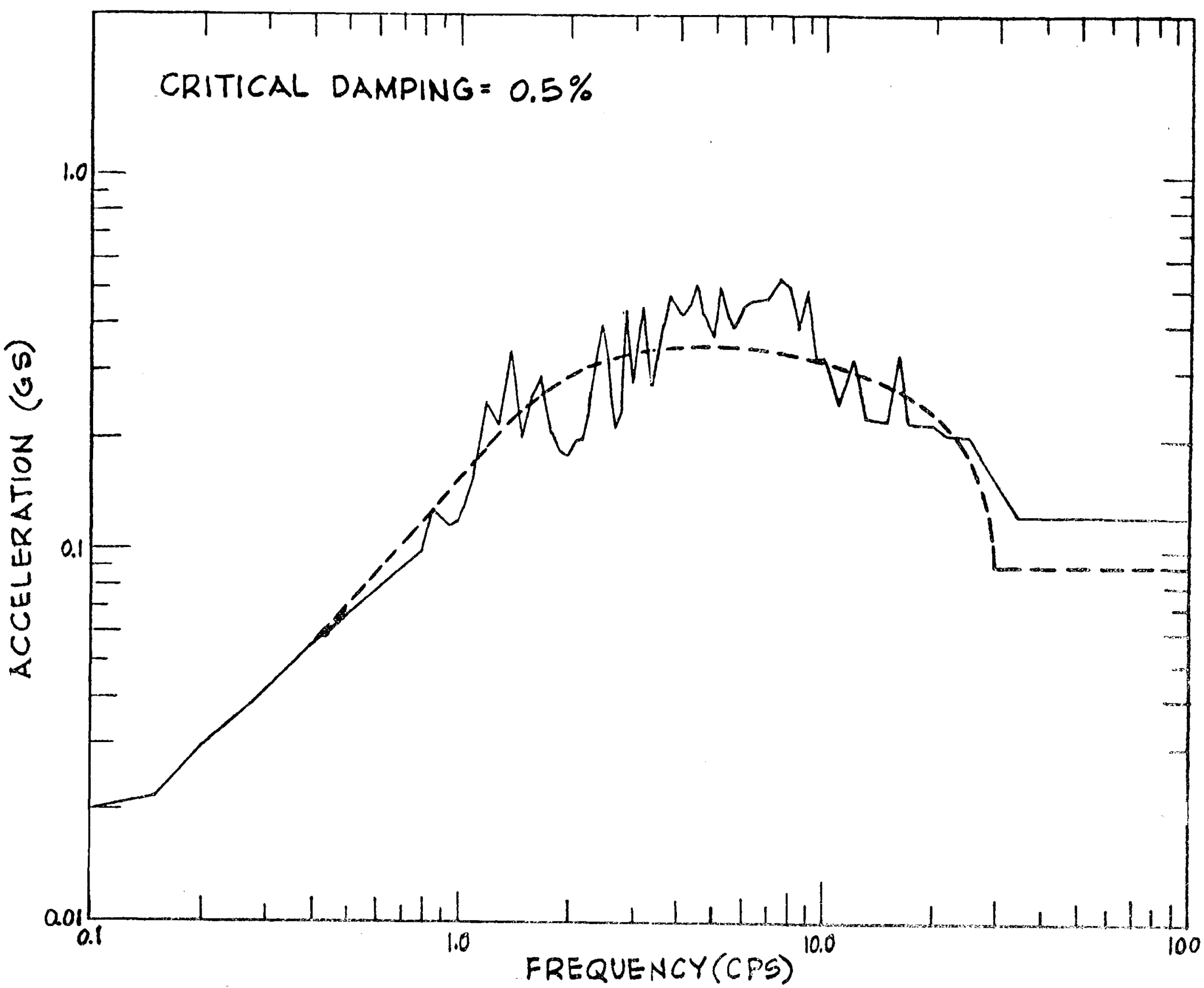


FIGURE 5.8–6 COMPARISON OF SMOOTH RESPONSE SPECTRA VS. RESPONSE SPECTRA FROM THE ANAMET TIME HISTORY DESIGN EARTHQUAKE (CRITICAL DAMPING = 1%)

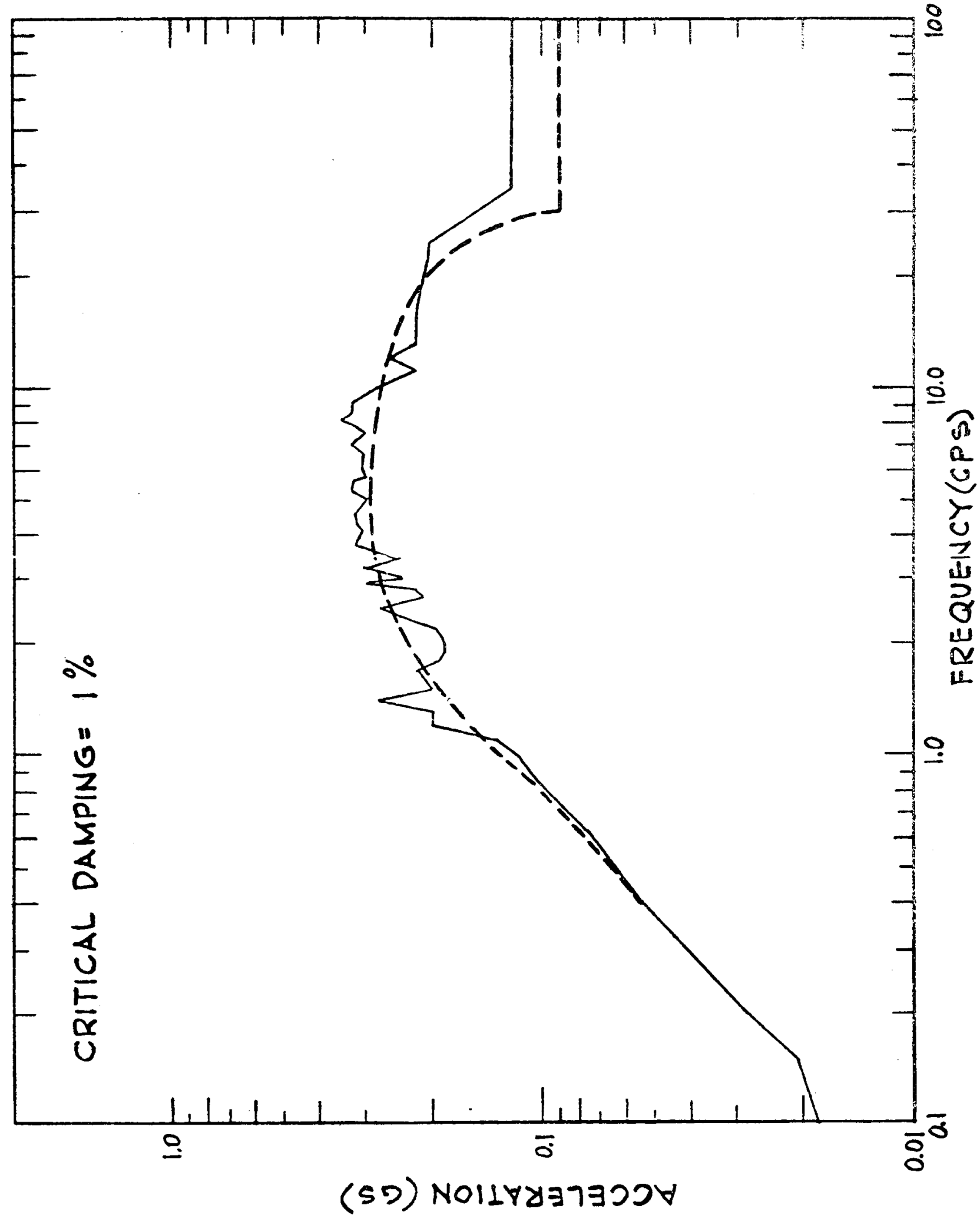


FIGURE 5.8–7 COMPARISON OF SMOOTH RESPONSE SPECTRA VS. RESPONSE SPECTRA FROM THE ANAMET TIME HISTORY DESIGN EARTHQUAKE (CRITICAL DAMPING = 2%)

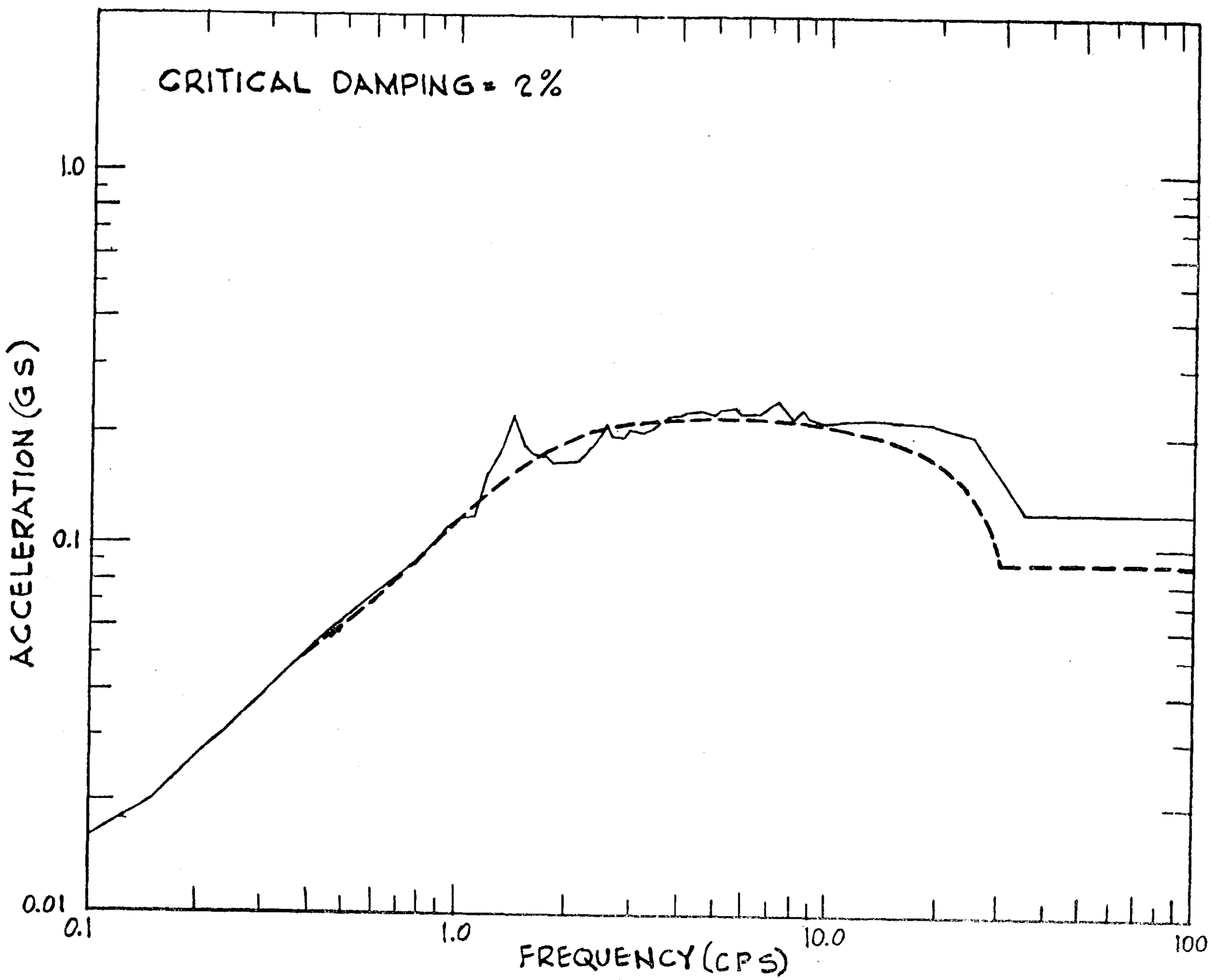


FIGURE 5.8–8 COMPARISON OF SMOOTH RESPONSE SPECTRA VS. RESPONSE SPECTRA FROM THE ANAMET TIME HISTORY DESIGN EARTHQUAKE (CRITICAL DAMPING = 5%)

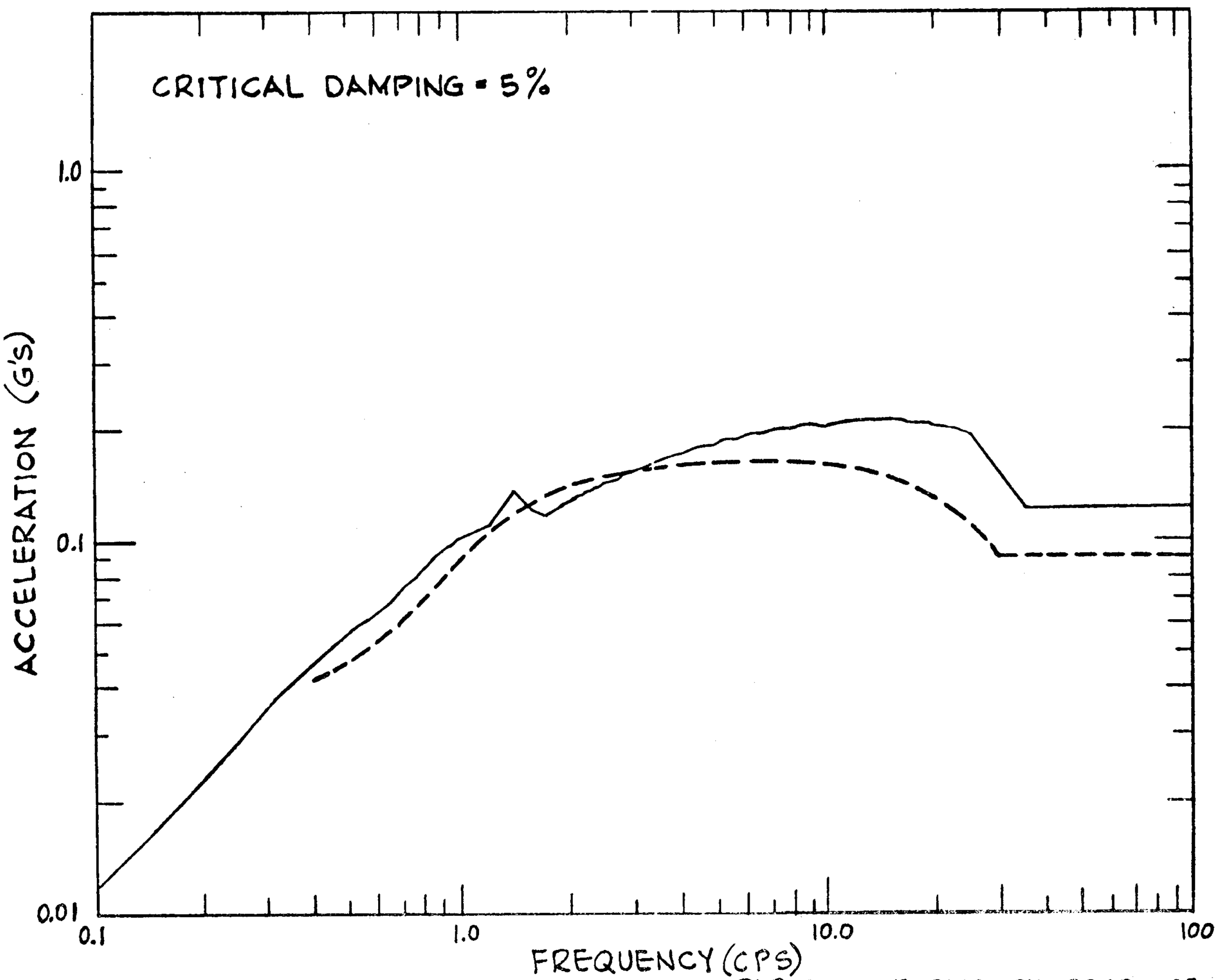


FIGURE 5.8–9 RESPONSE SPECTRA FROM THE TIME HISTORY DESIGN EARTHQUAKE WITH VARIOUS FREQUENCY INTERVALS

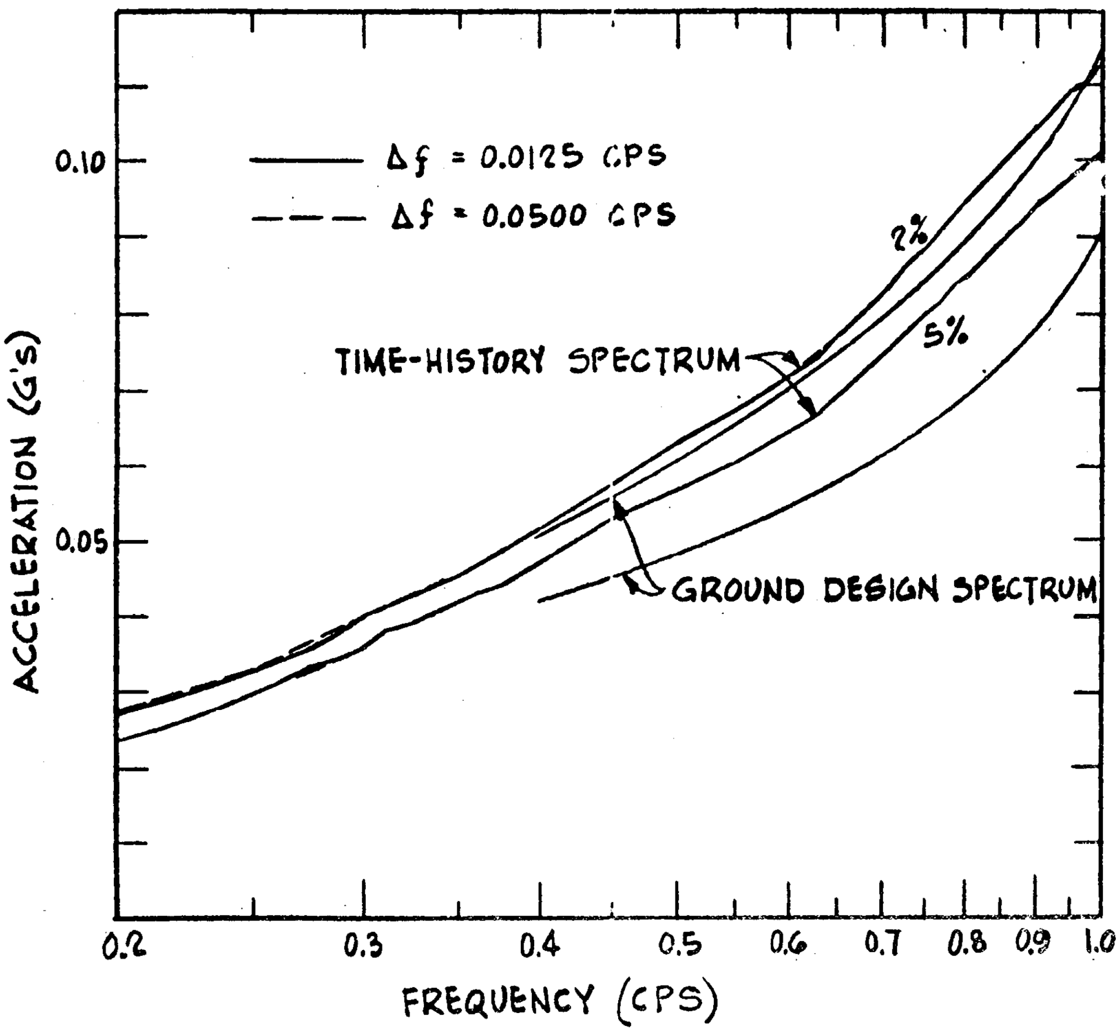
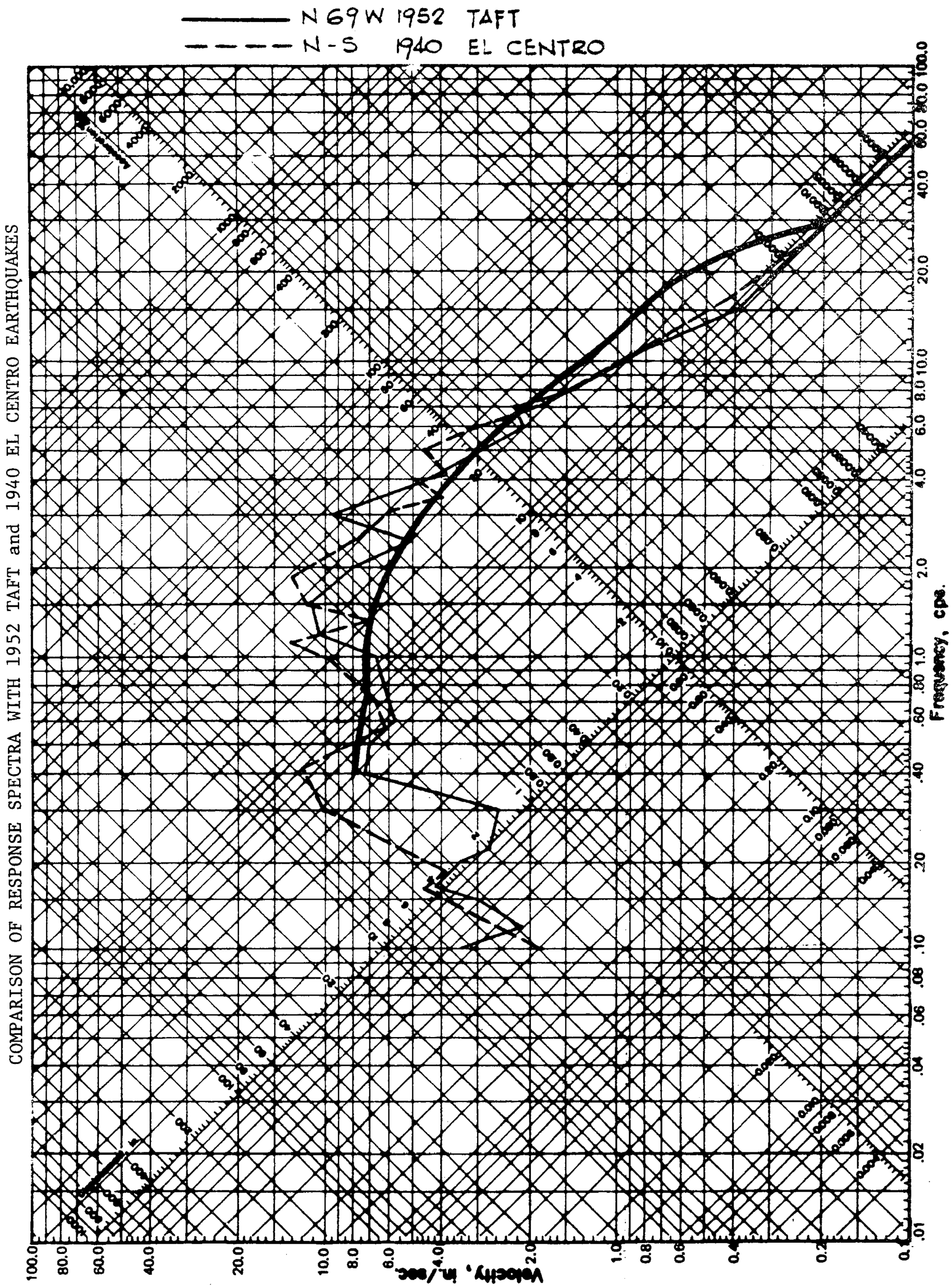


FIGURE 5.8–10 COMPARISON OF RESPONSE SPECTRA WITH 1952 TAFT AND 1940 EL CENTRO EARTHQUAKE



Withhold under 10 CFR 2.390 (d) (1)

FIGURE 5.8–11 FOUNDATION OUTLINE

[redacted]

Withhold under 10 CFR 2.390 (d) (1)

FIGURE 5.8–12 SECTIONS A-A & B-B

[redacted]

FIGURE 5.8–13 AUXILIARY BAY OF TURBINE BUILDING

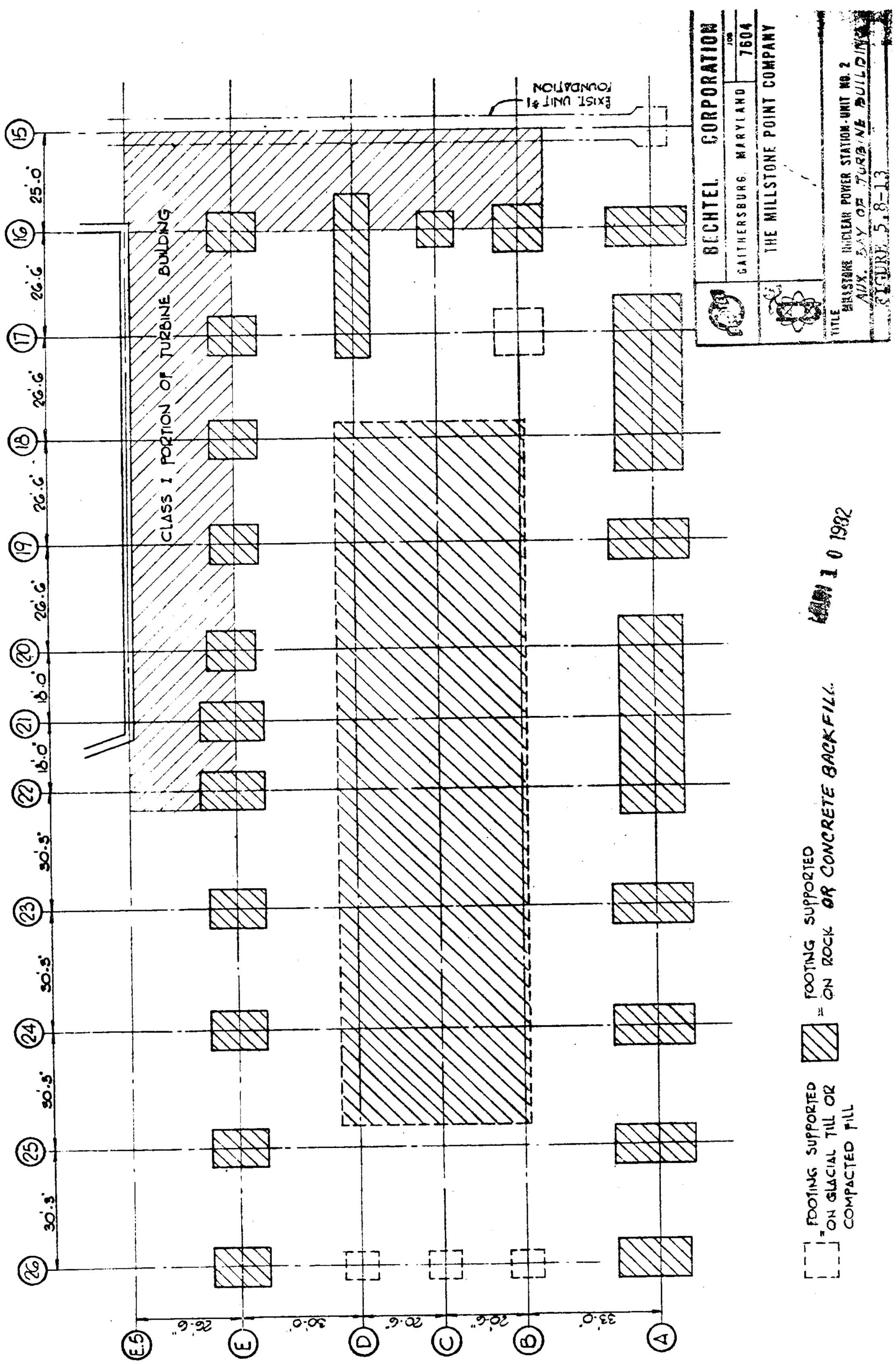


FIGURE 5.8–14 CONTAINMENT MASS MODEL

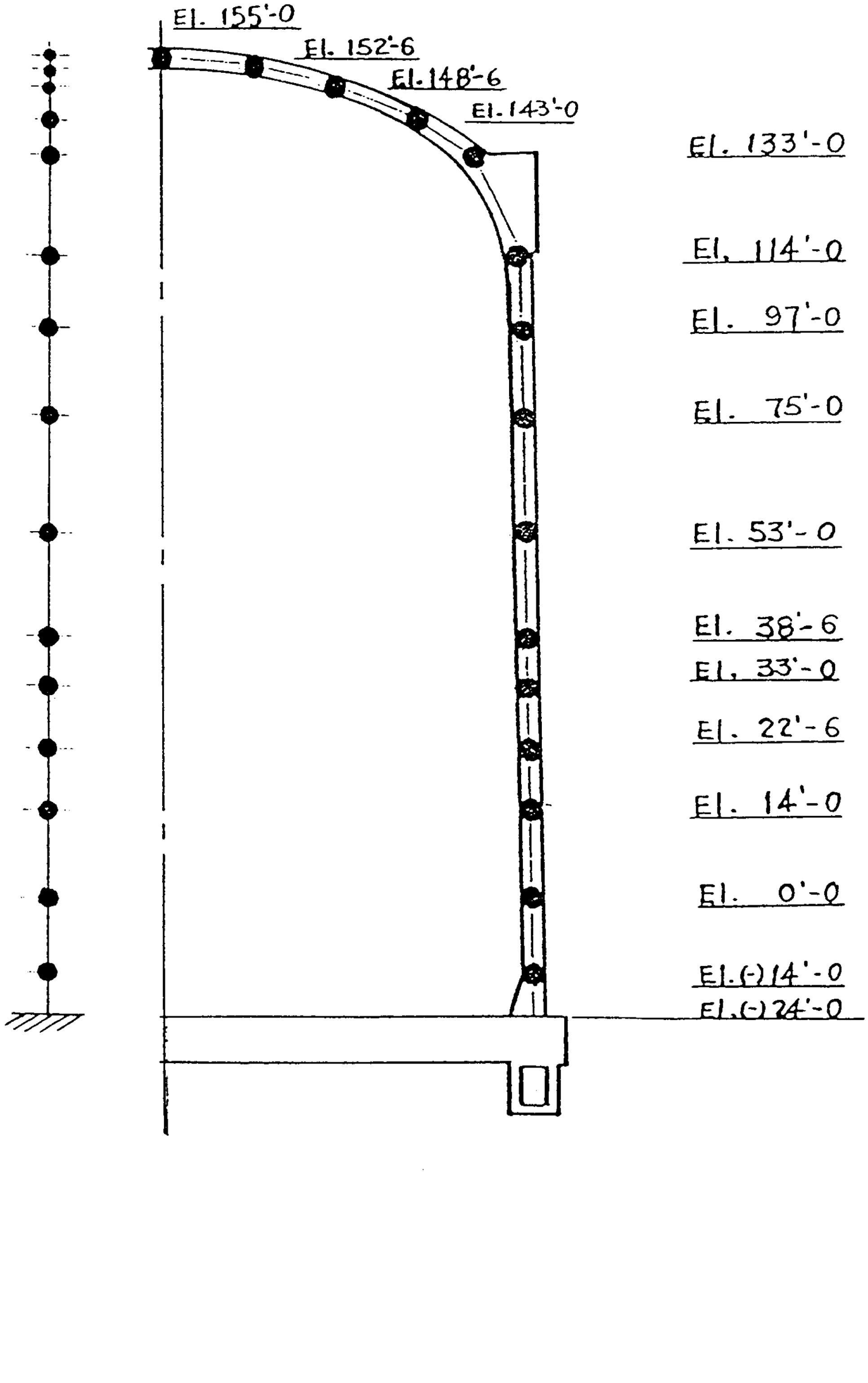


FIGURE 5.8–15 MODE SHAPES FOR THE CONTAINMENT BUILDING

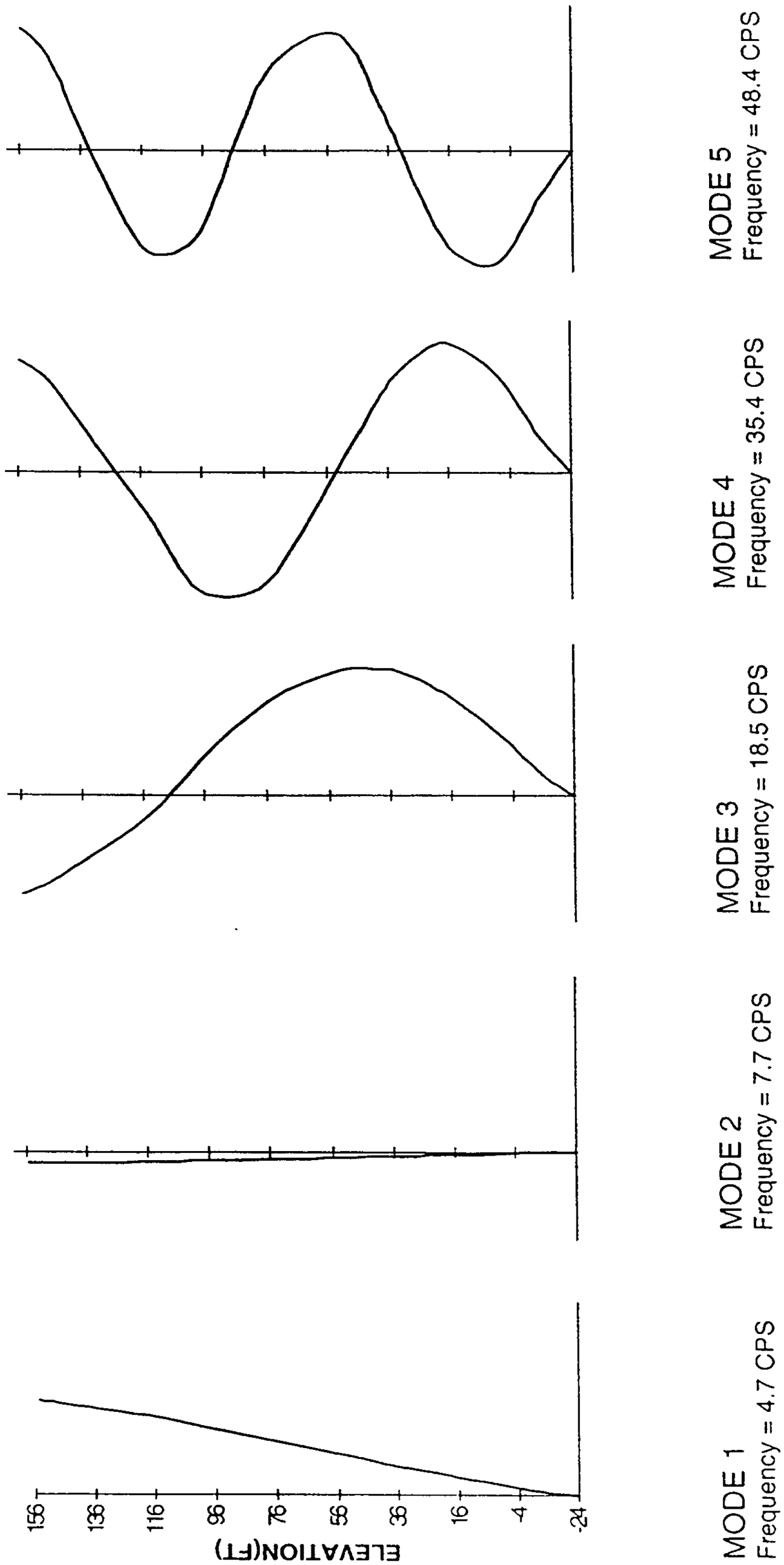


FIGURE 5.8–16 CONTAINMENT BUILDING DESIGN EARTHQUAKE 9% GROUND ACCELERATION

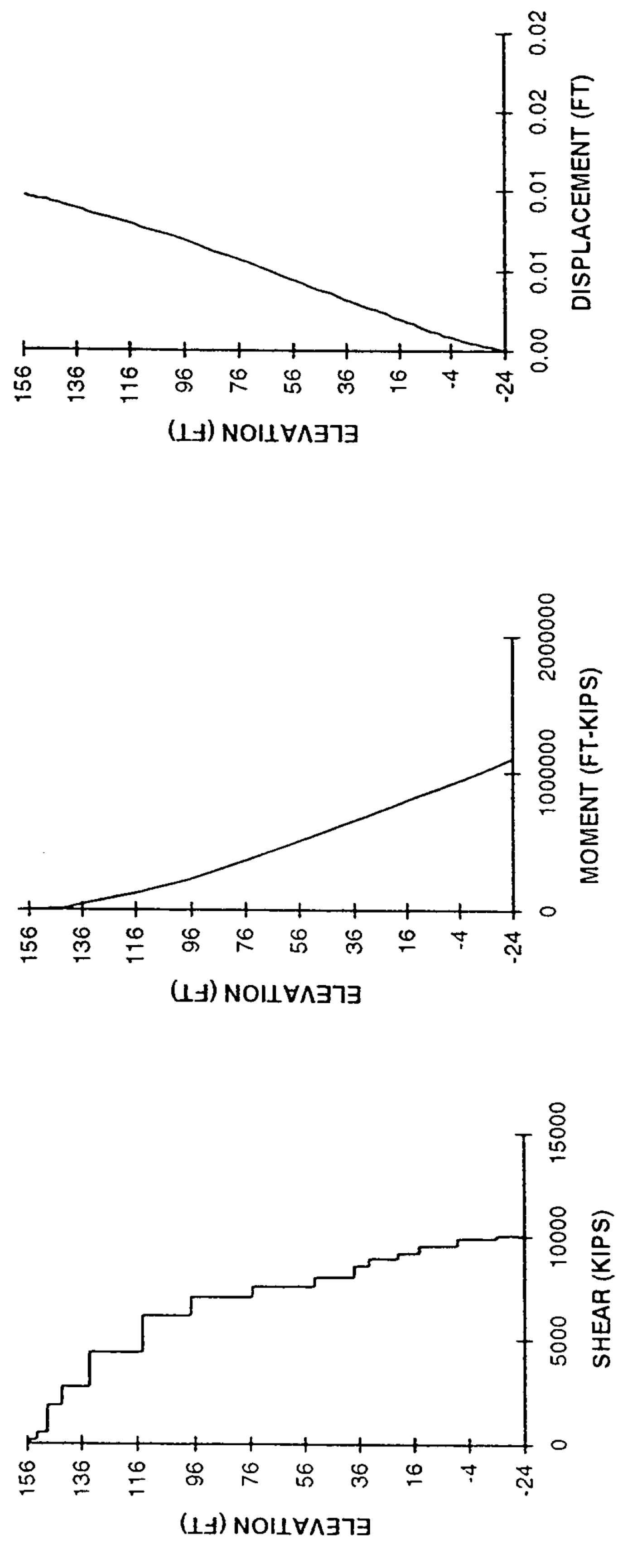


FIGURE 5.8–17 CONTAINMENT BUILDING DESIGN EARTHQUAKE 17% GROUND ACCELERATION

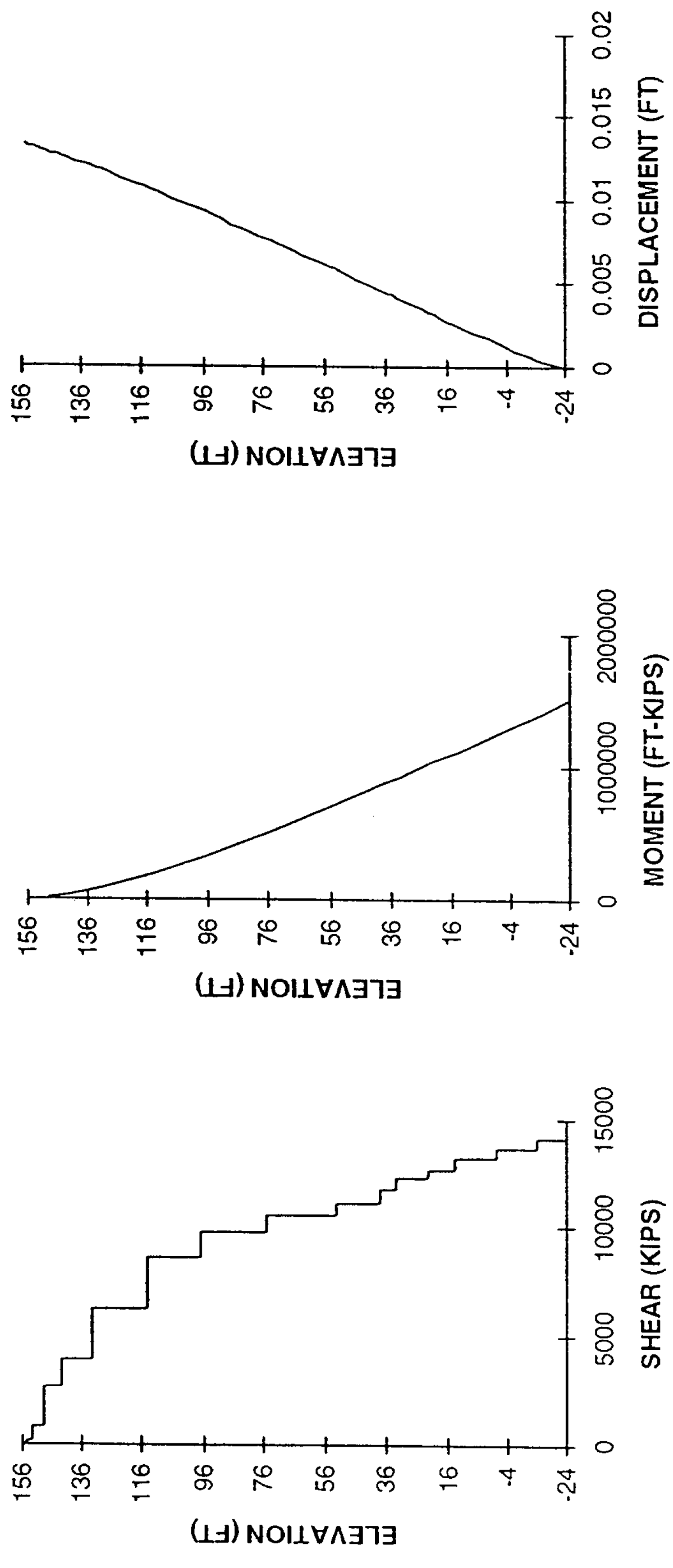


FIGURE 5.8–18 CONTAINMENT STRUCTURE ELEVATION 53 FEET 0 INCHES

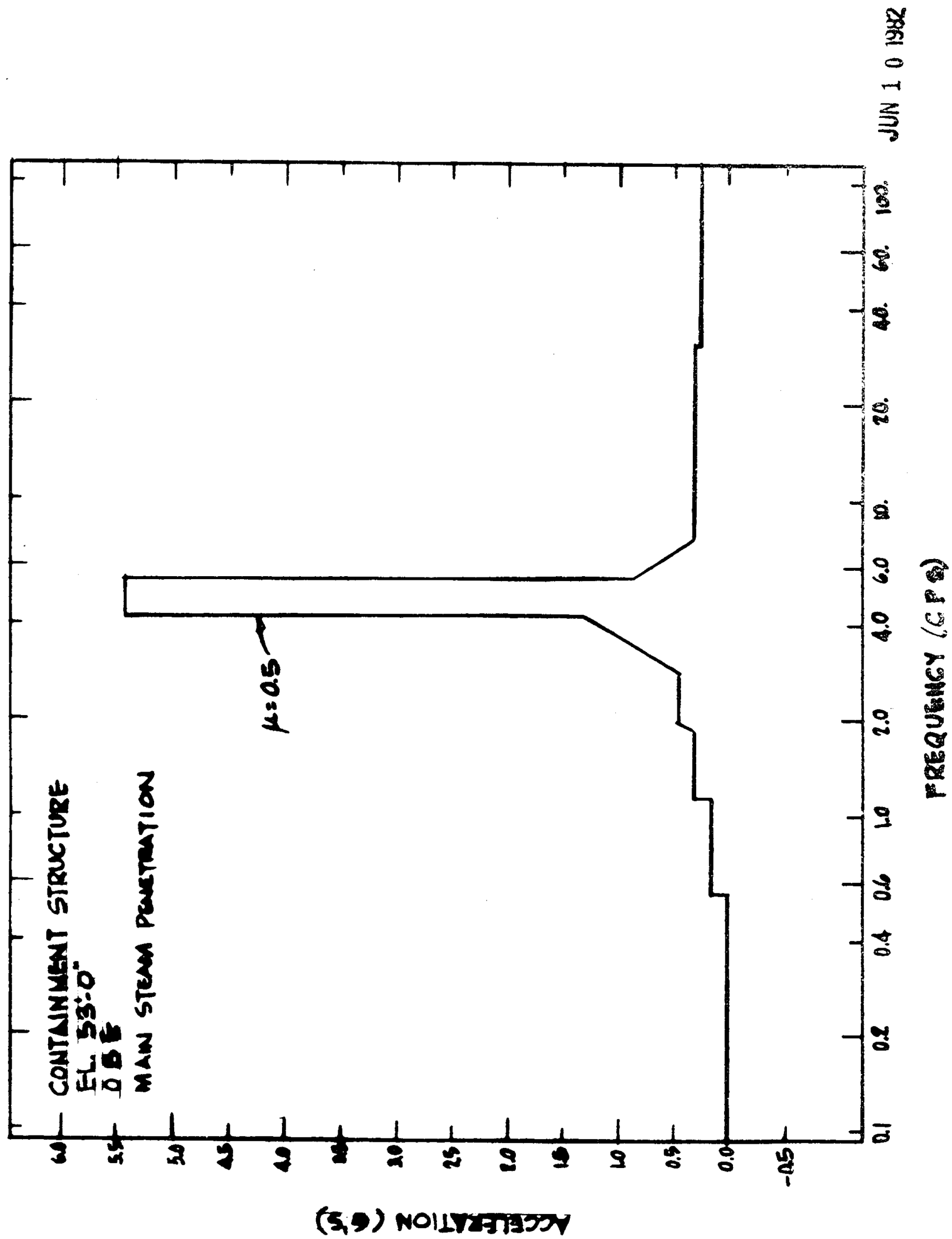


FIGURE 5.8–19 CONTAINMENT STRUCTURE ELEVATION 97 FEET 0 INCHES

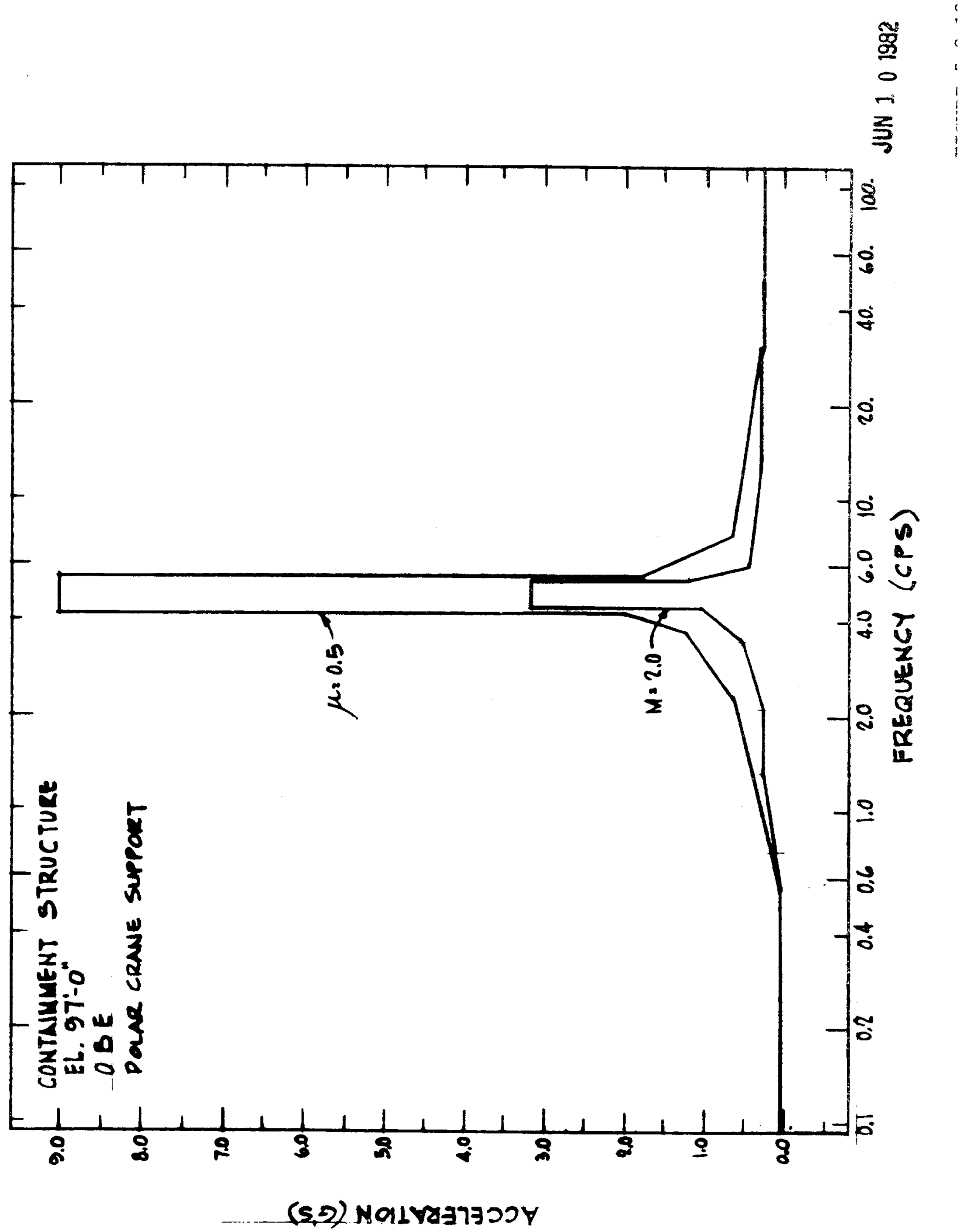


FIGURE 5.8–20 CONTAINMENT STRUCTURE ELEVATION 152 FEET 6 INCHES

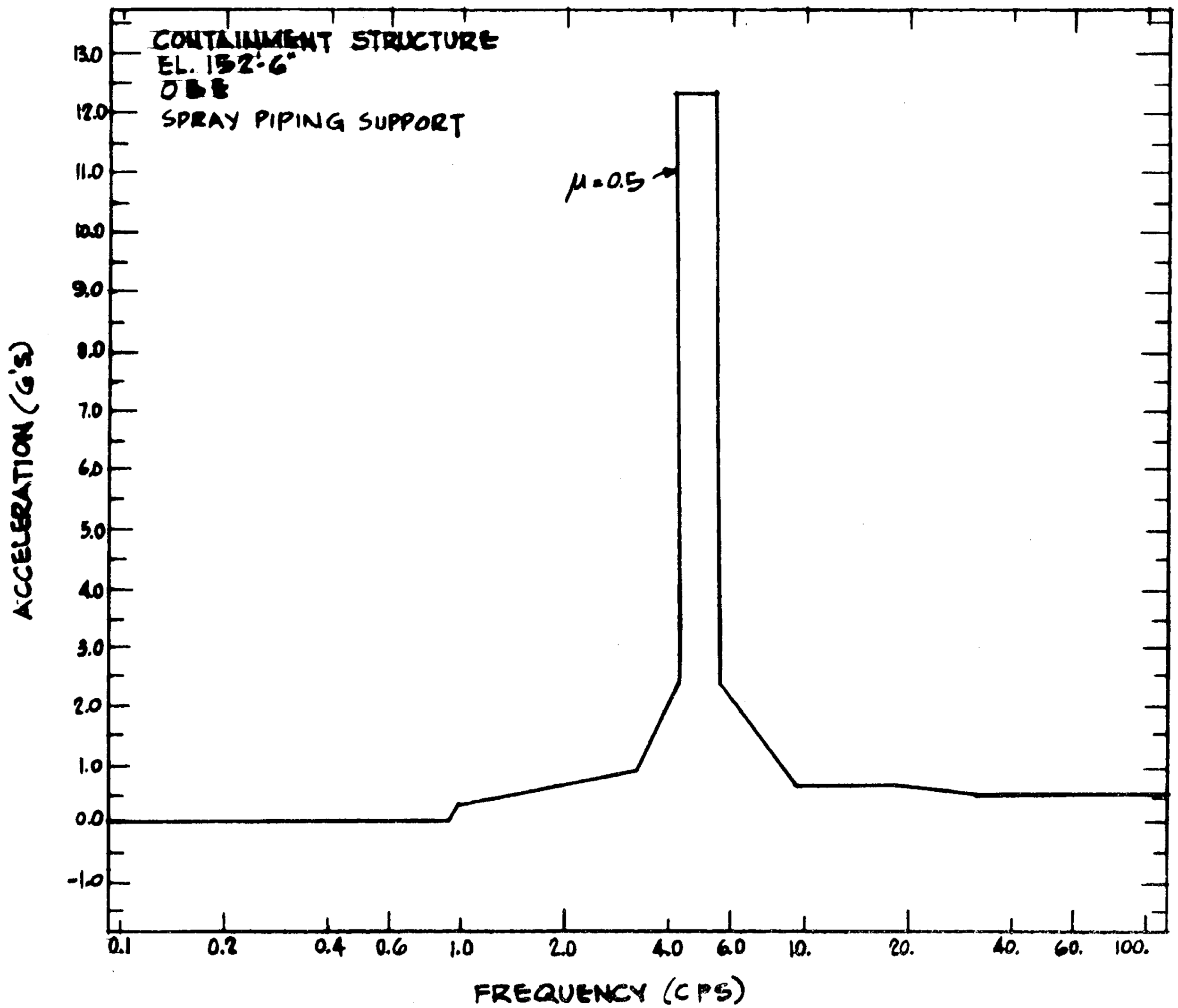


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FIGURE 5.8–21 CONTAINMENT INTERNALS MODEL

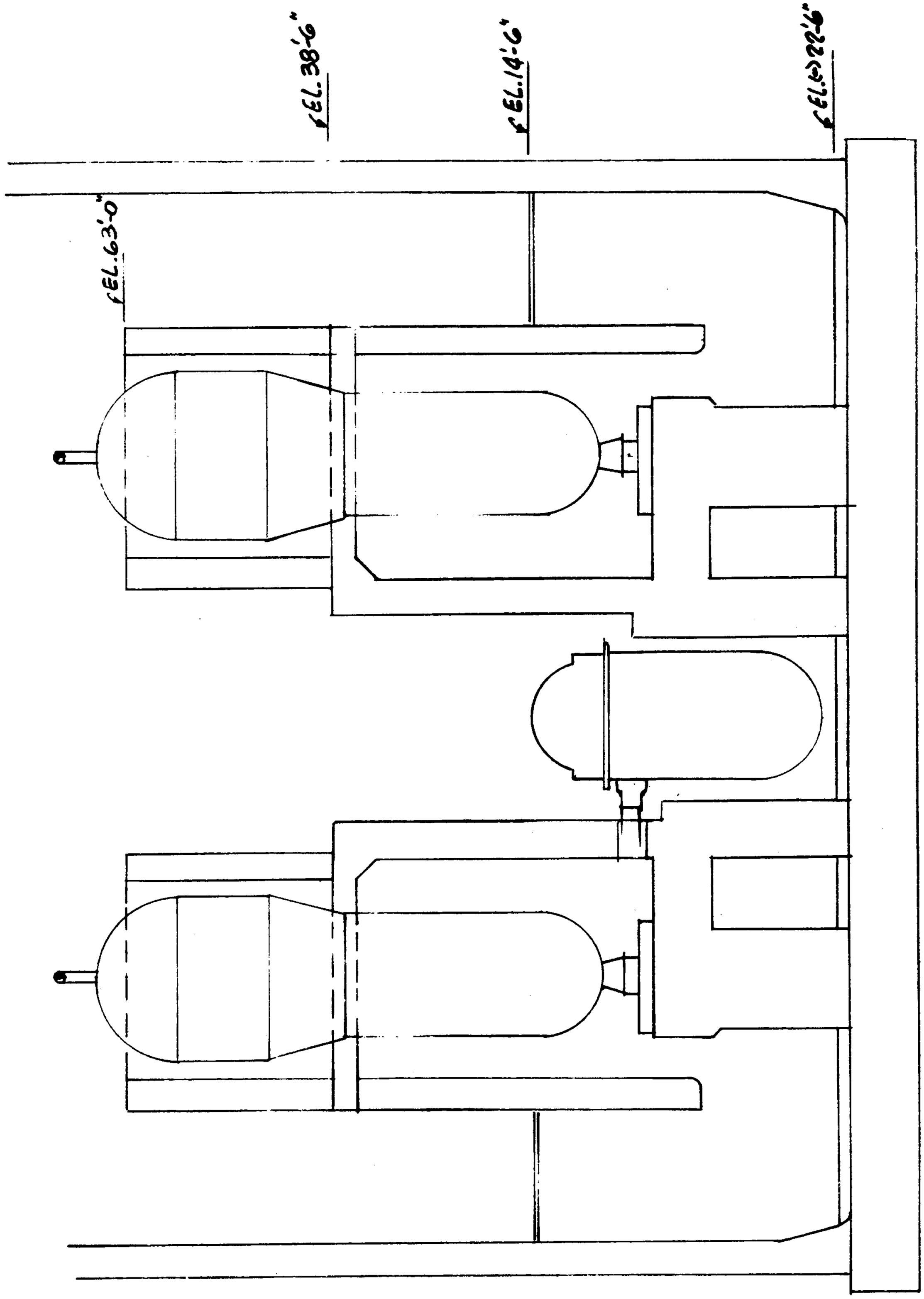


FIGURE 5.8–22 MODE SHAPES & FREQUENCIES CONTAINMENT INTERNALS - NORTH-SOUTH

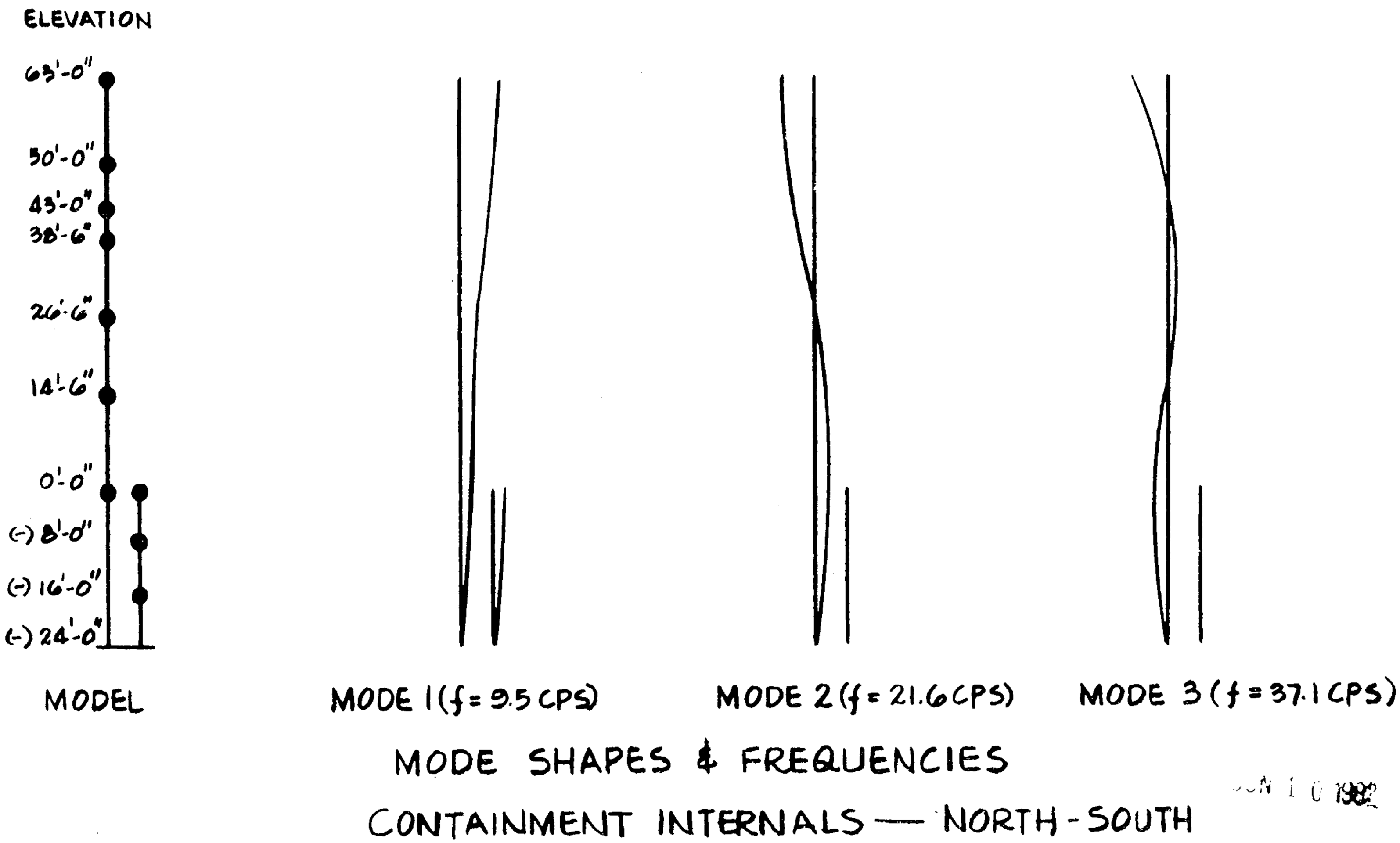


FIGURE 5.8–23 MODE SHAPES & FREQUENCIES CONTAINMENT INTERNALS - NORTH-SOUTH OBE (DBE)

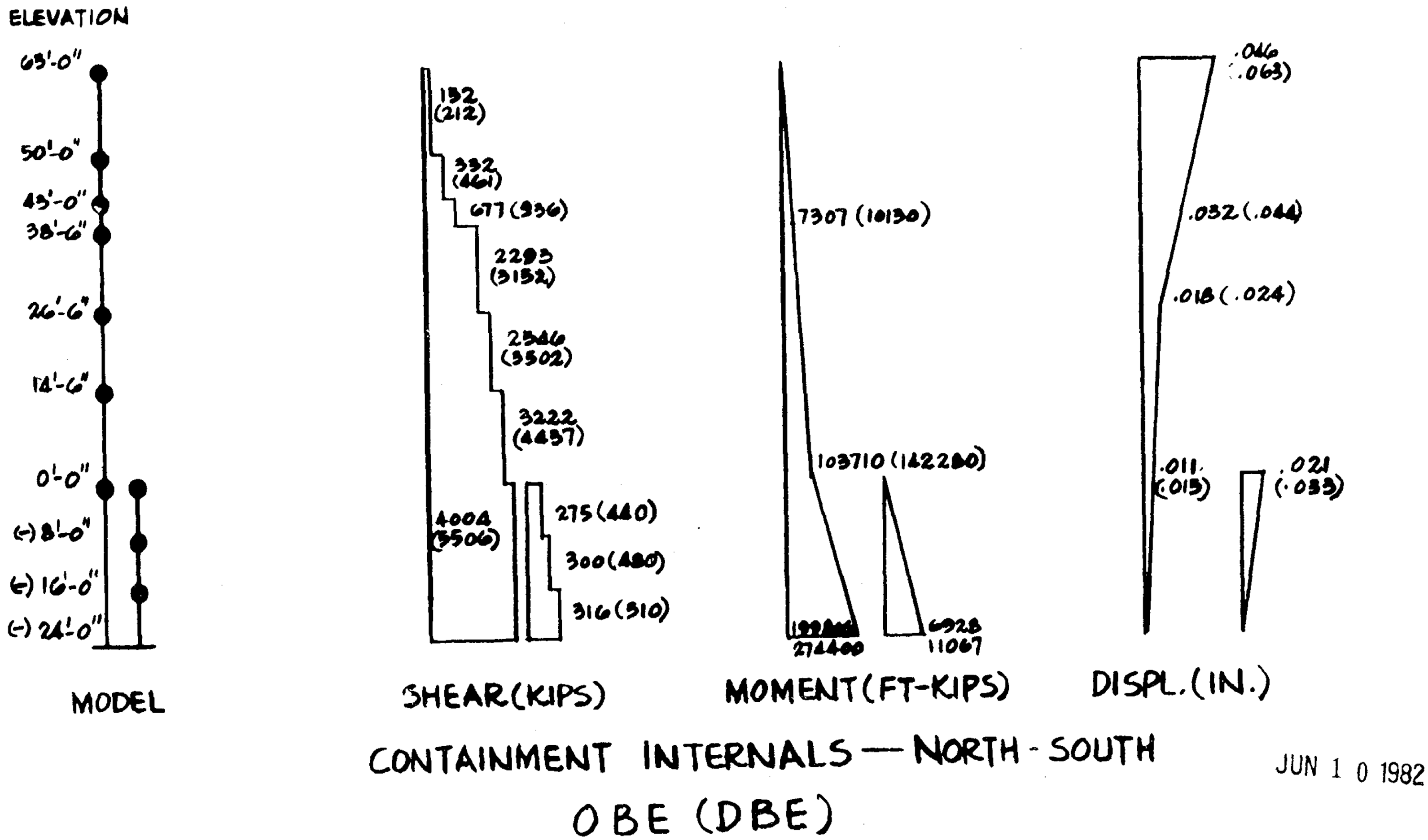


FIGURE 5.8-23

FIGURE 5.8–24 MODE SHAPES & FREQUENCIES CONTAINMENT INTERNALS - EAST-WEST

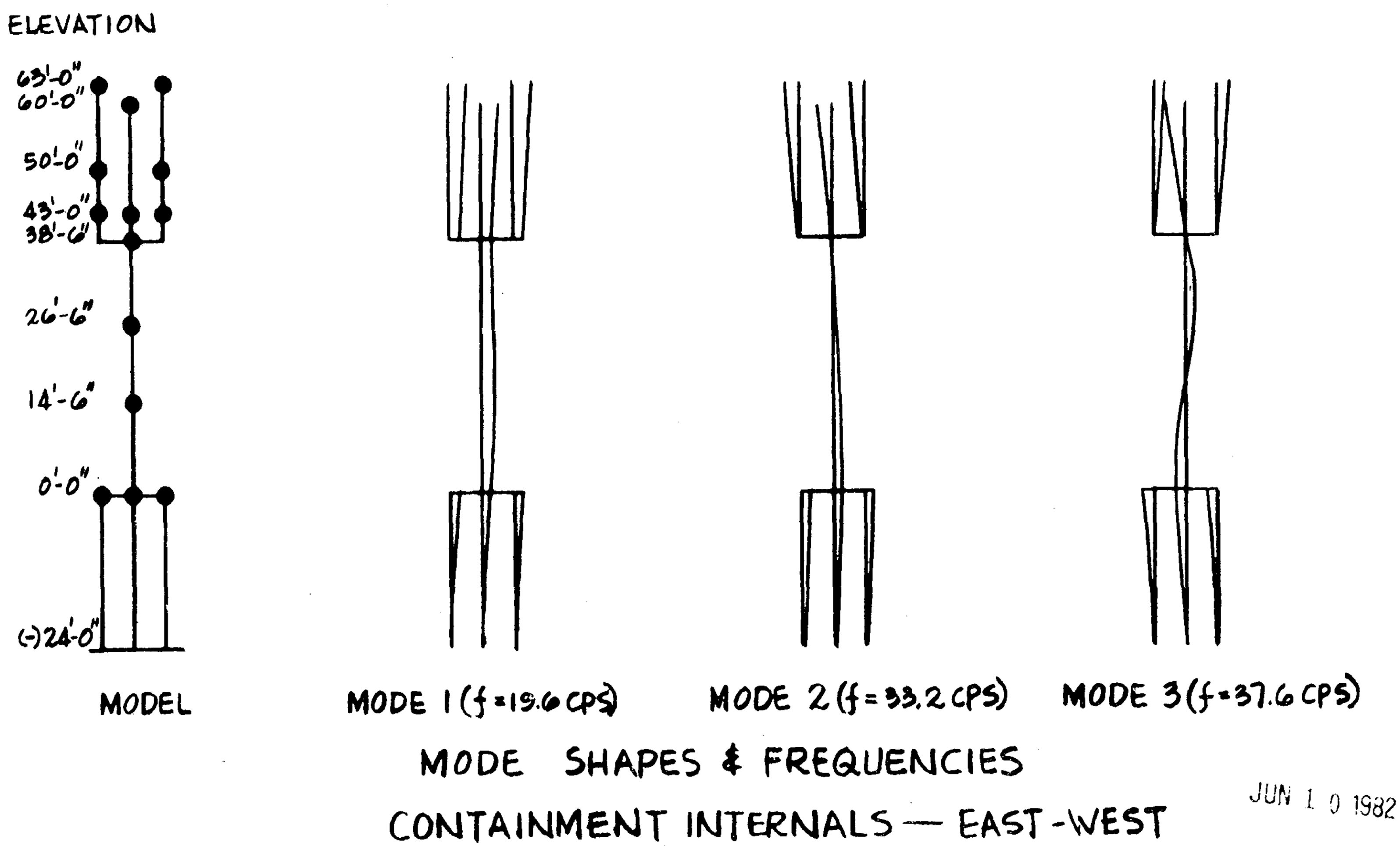


FIGURE 5.8–25 CONTAINMENT INTERNALS - EAST-WEST OBE (DBE)

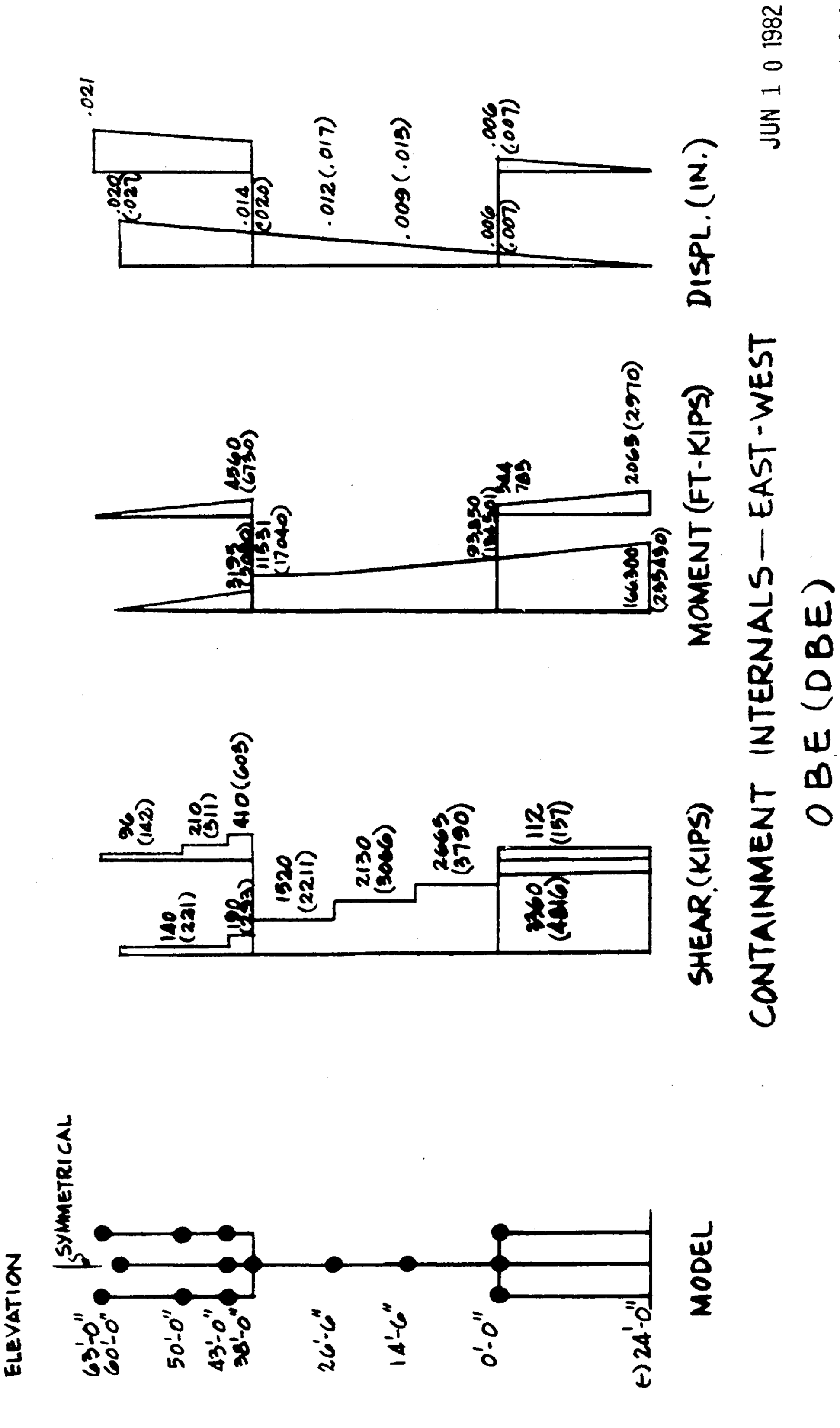


FIGURE 5.8–26 CONTAINMENT INTERNALS ELEVATION 0 FEET 0 INCHES - OBE (NORTH-SOUTH) REACTOR VESSEL SUPPORT

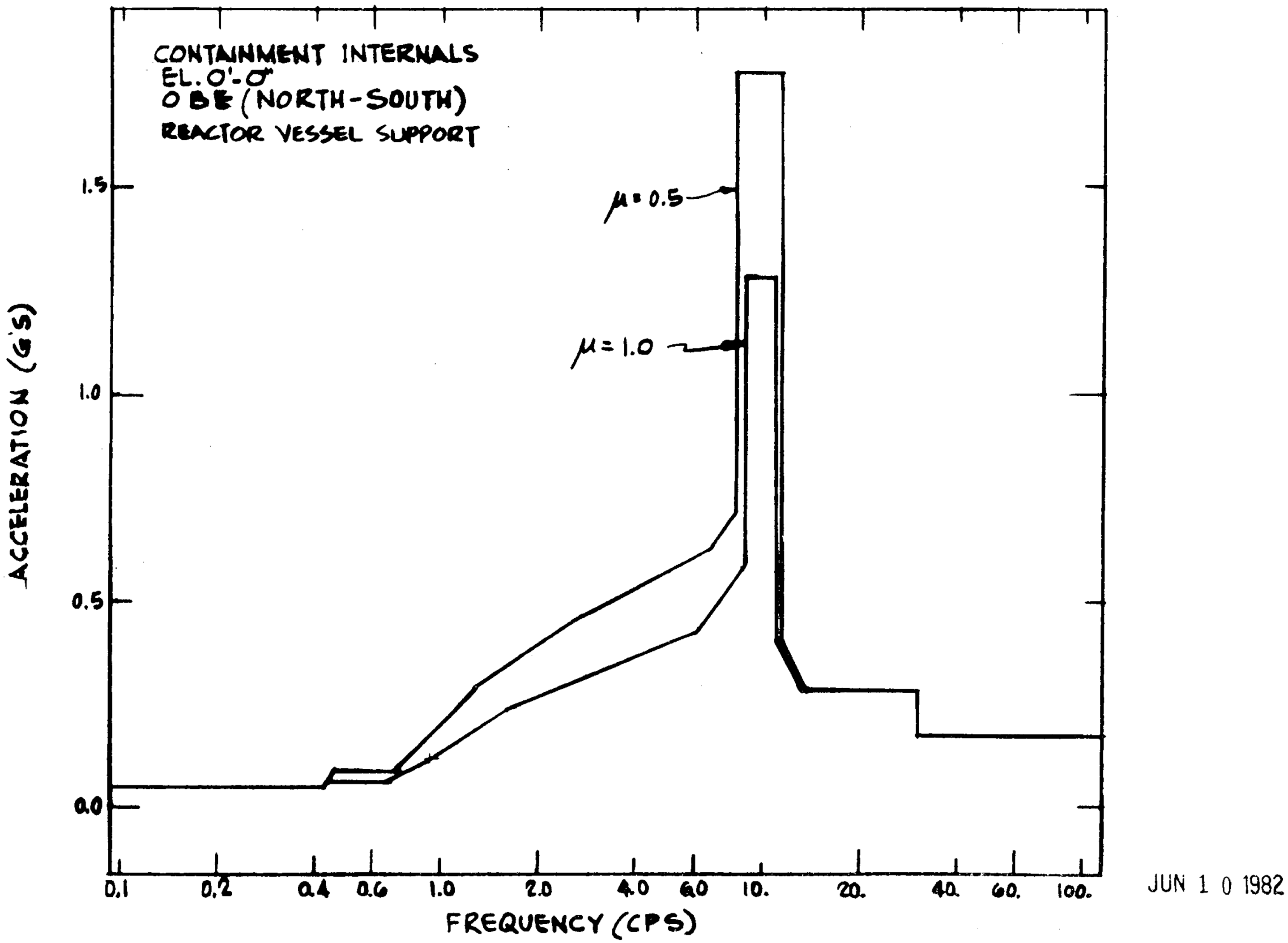


FIGURE 5.8–27 CONTAINMENT INTERNALS ELEVATION 0 FEET 0 INCHES - OBE (EAST-WEST) REACTOR VESSEL SUPPORT

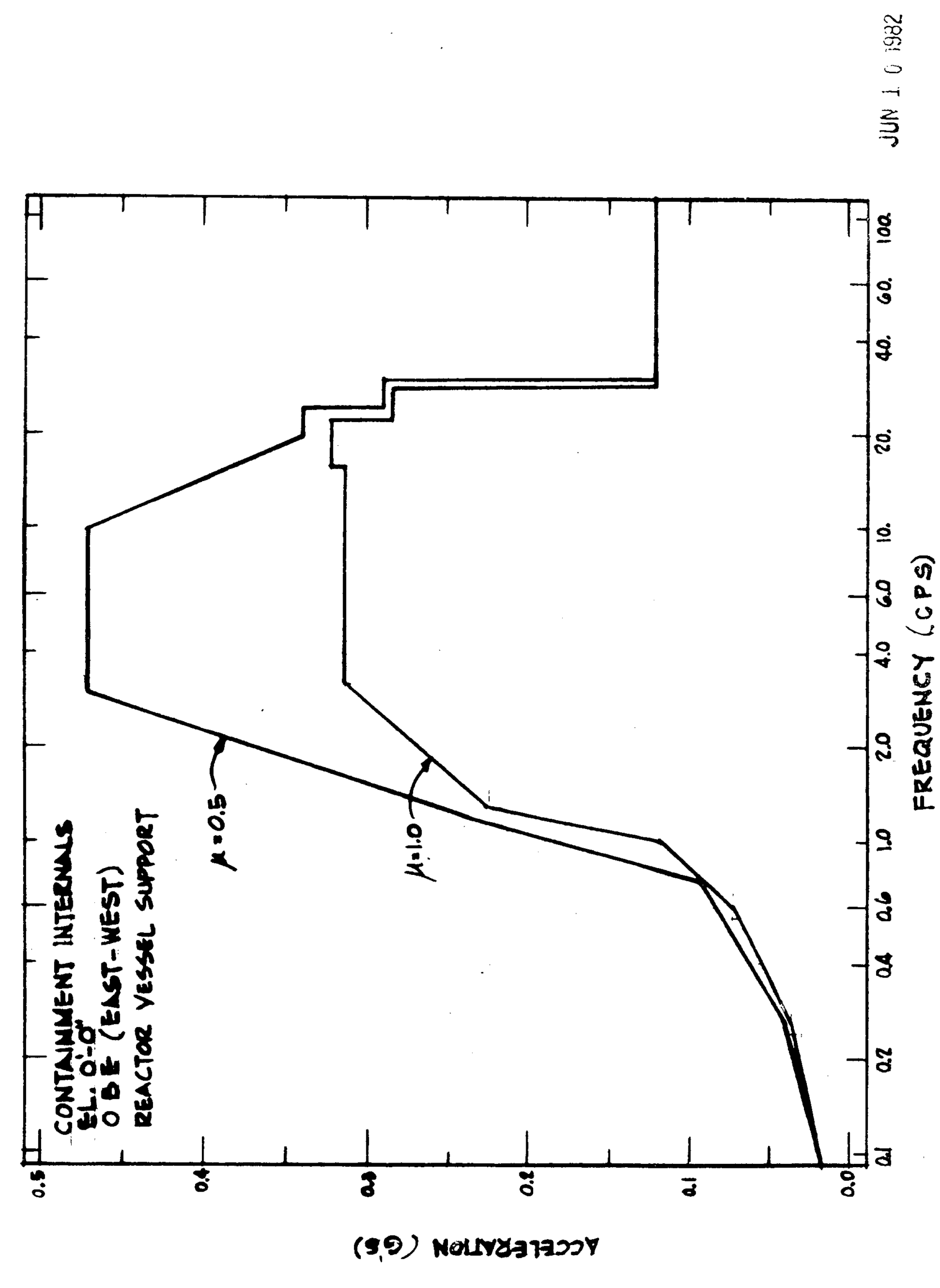
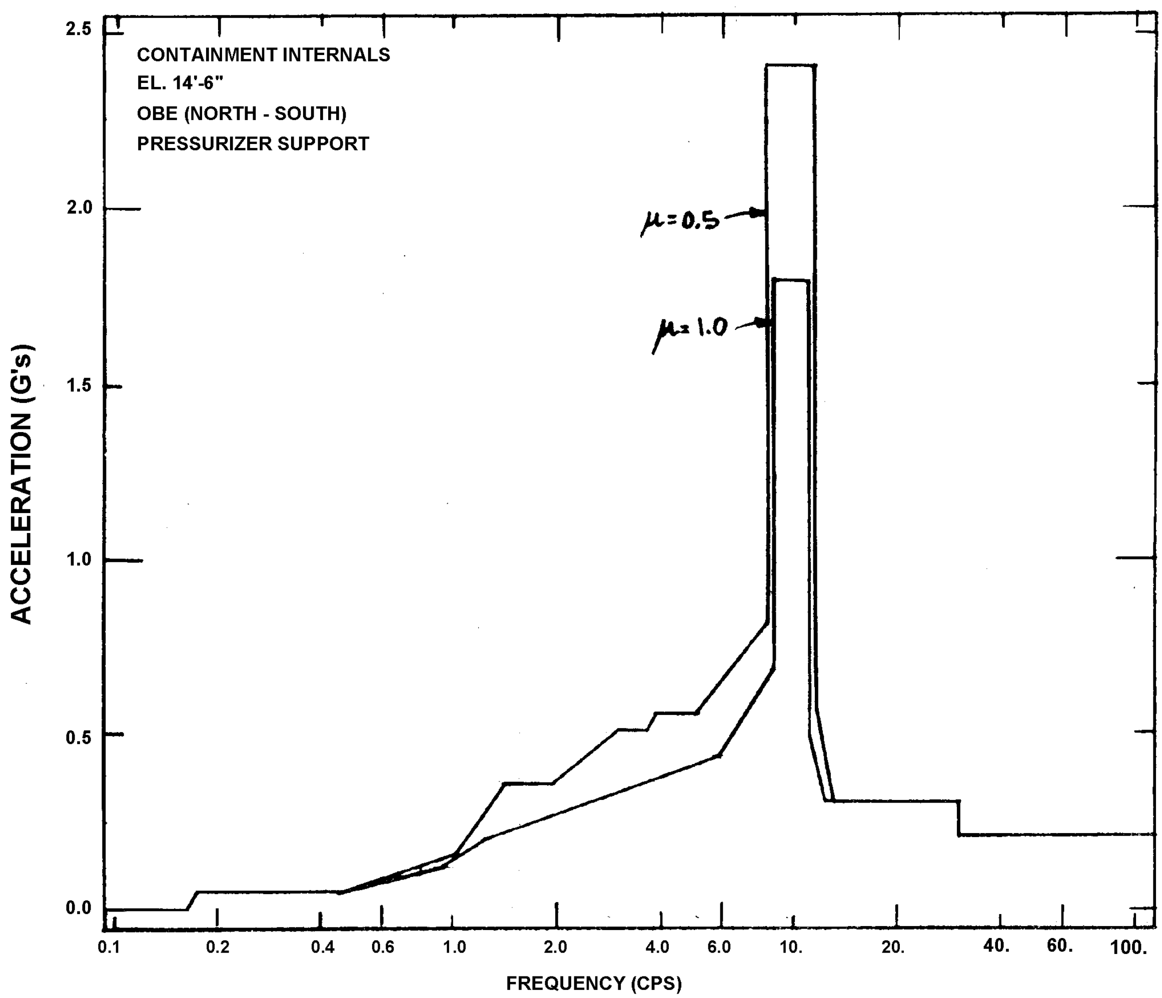
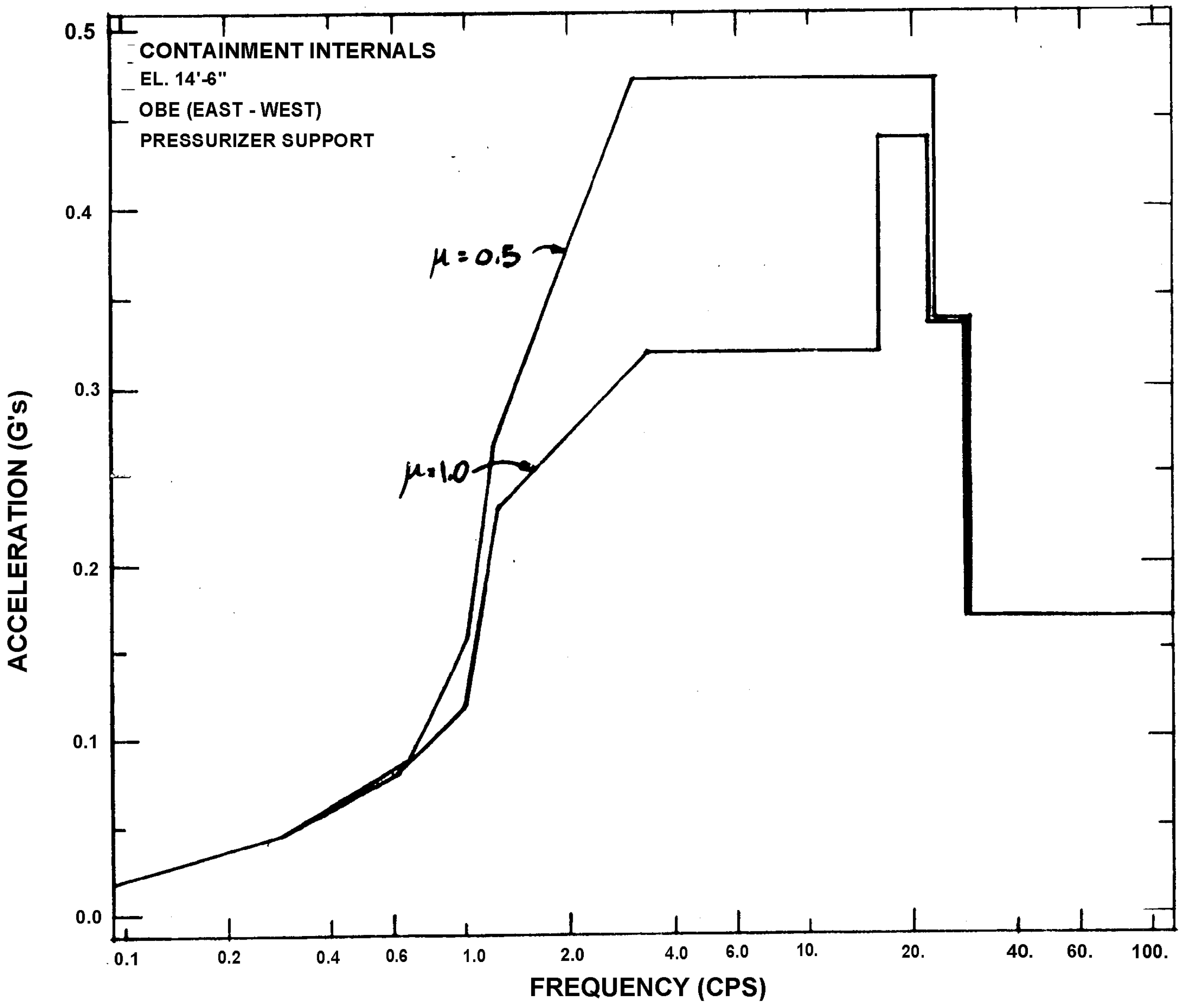


FIGURE 5.8–28 CONTAINMENT INTERNALS ELEVATION 14 FEET 6 INCHES - OBE (NORTH-SOUTH) PRESSURIZER SUPPORT



Note that this ARS is retained in here for historical purpose only. The ARS for the modified pressurizer cubicle are contained in Specification SP-M2-ME-368.

FIGURE 5.8–29 CONTAINMENT INTERNALS ELEVATION 14 FEET 6 INCHES - OBE (EAST-WEST) PRESSURIZER SUPPORT



Note that this ARS is retained in here for historical purpose only. The ARS for the modified pressurizer cubicle are contained in Specification SP-M2-ME-368.

FIGURE 5.8–30 CONTAINMENT INTERNALS ELEVATION 38 FEET 6 INCHES - OBE (NORTH-SOUTH) SAFETY INJECTION TANK SUPPORT

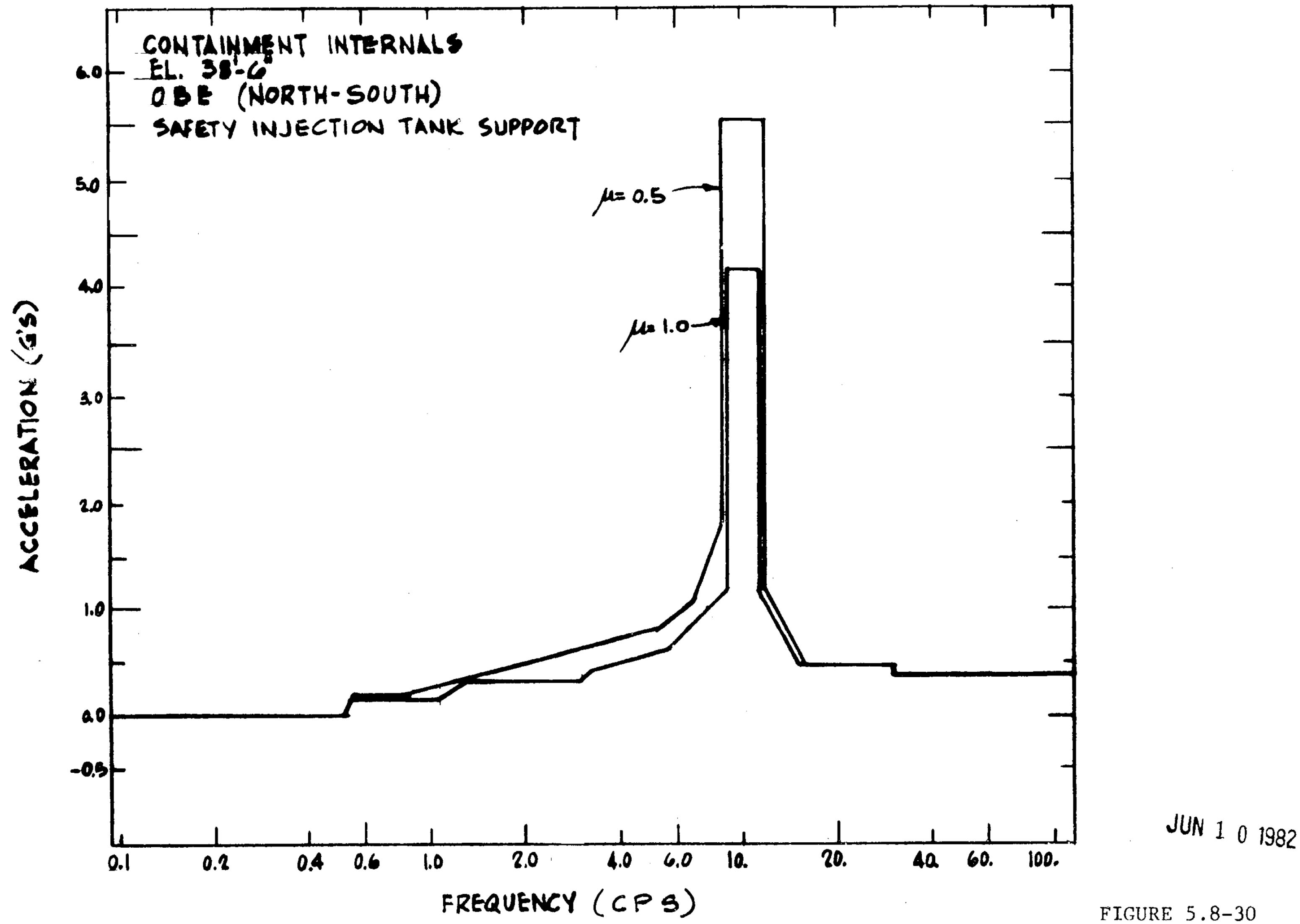


FIGURE 5.8-30

FIGURE 5.8–31 CONTAINMENT INTERNALS ELEVATION 38 FEET 6 INCHES - OBE (EAST-WEST) SAFETY INJECTION TANK SUPPORT

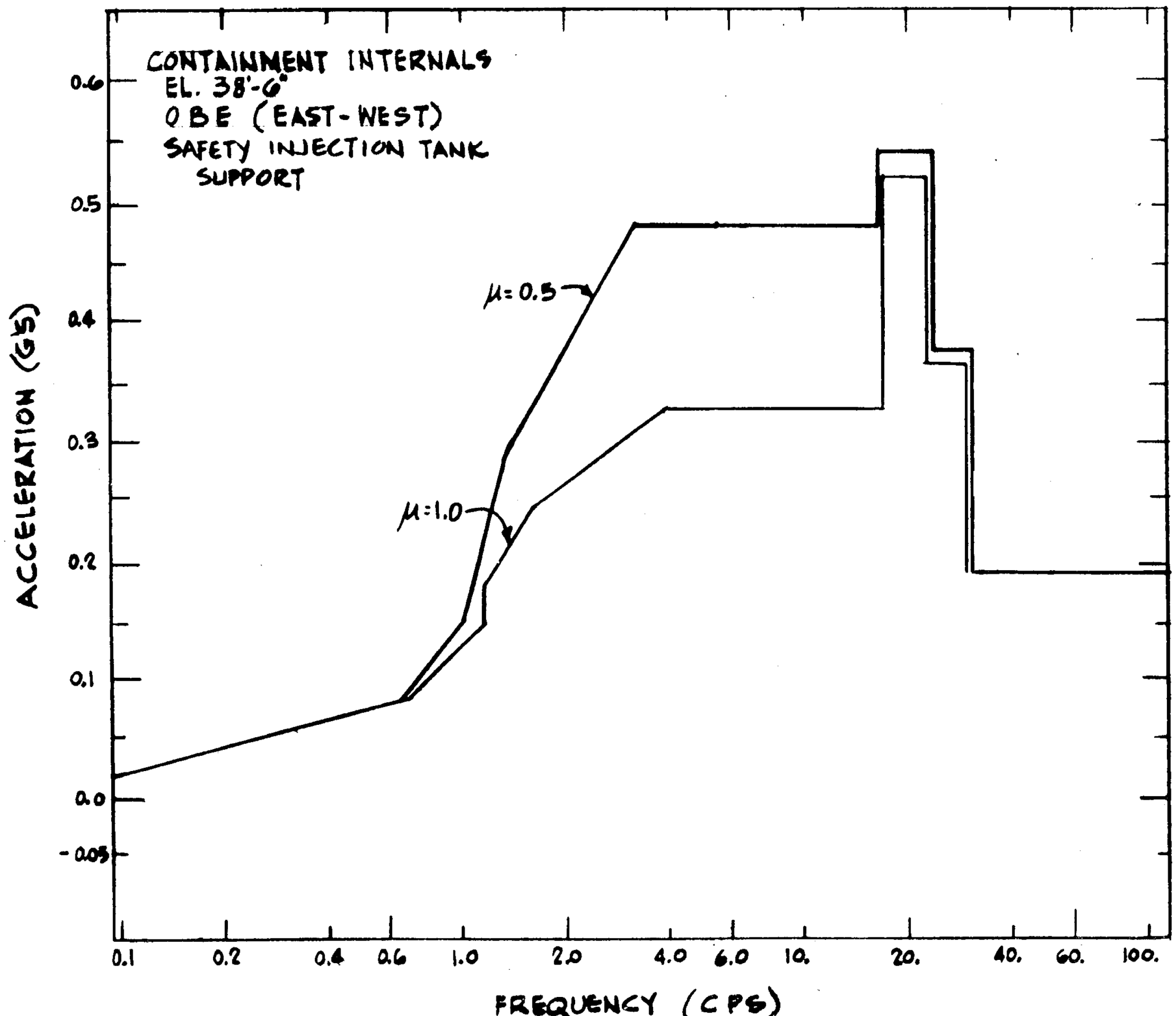


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FIGURE 5.8-31

FIGURE 5.8–32 CONTAINMENT INTERNALS ELEVATION 43 FEET 0 INCHES - OBE (NORTH-SOUTH) STEAM GENERATOR UPPER SUPPORT (SNUBBERS)

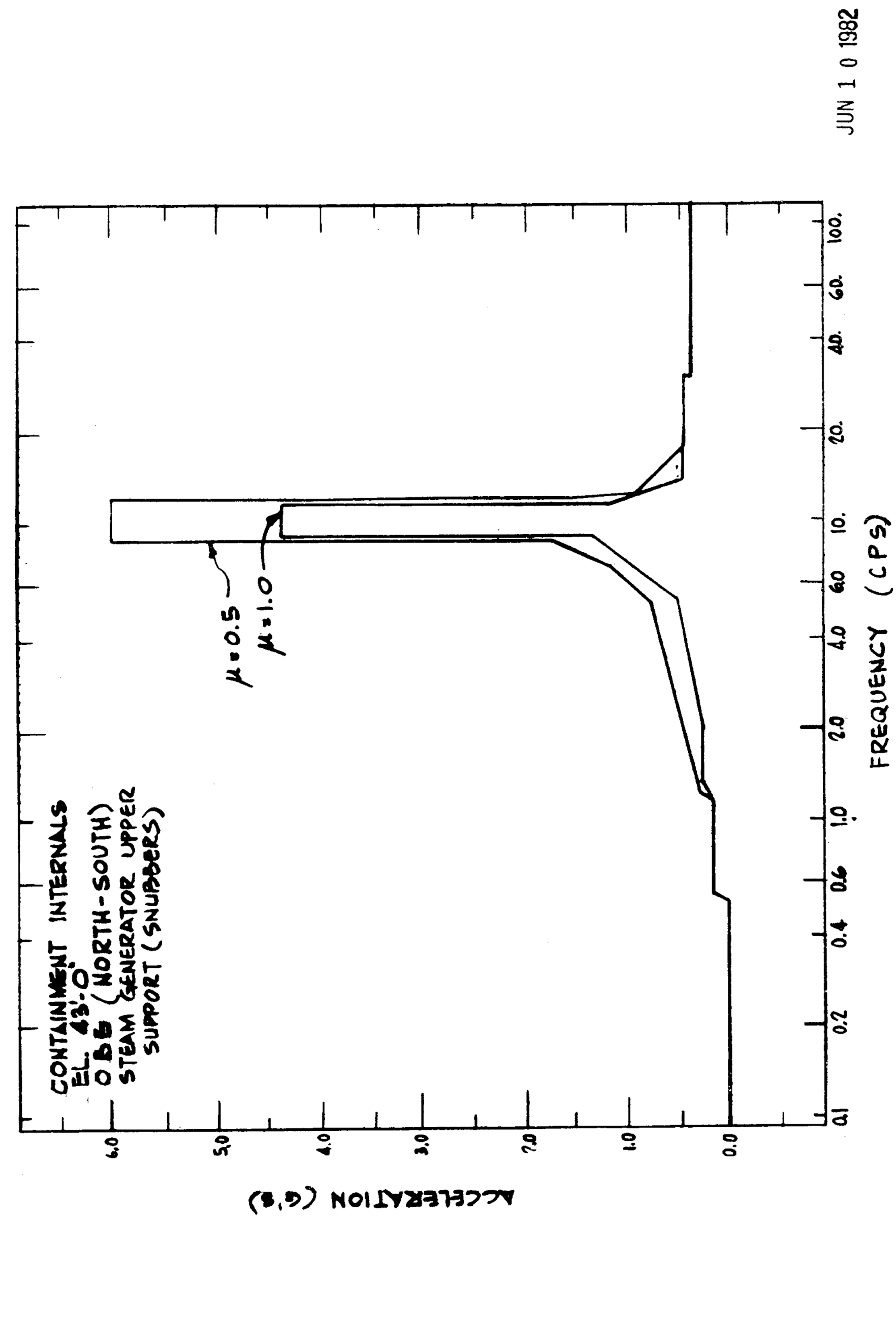

FIGURE 5.8–33 CONTAINMENT INTERNALS ELEVATION 43 FEET 0 INCHES - OBE (EAST-WEST) STEAM GENERATOR UPPER SUPPORT (SNUBBERS)

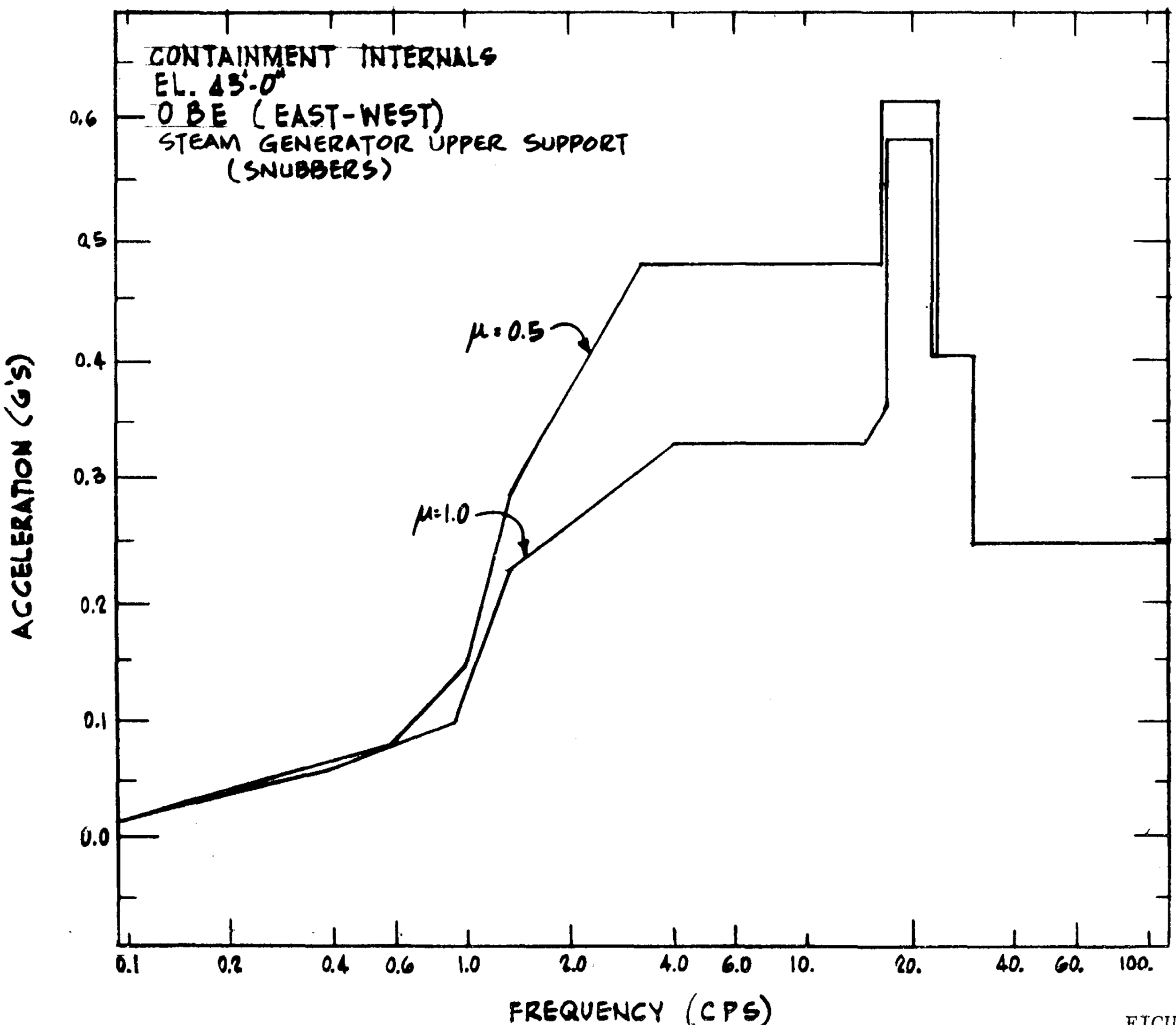

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FIGURE 5.8–34 CONTAINMENT INTERNALS ELEVATION 50 FEET 0 INCHES - OBE (NORTH-SOUTH) STEAM GENERATOR UPPER SUPPORT (SHEAR KEYS)

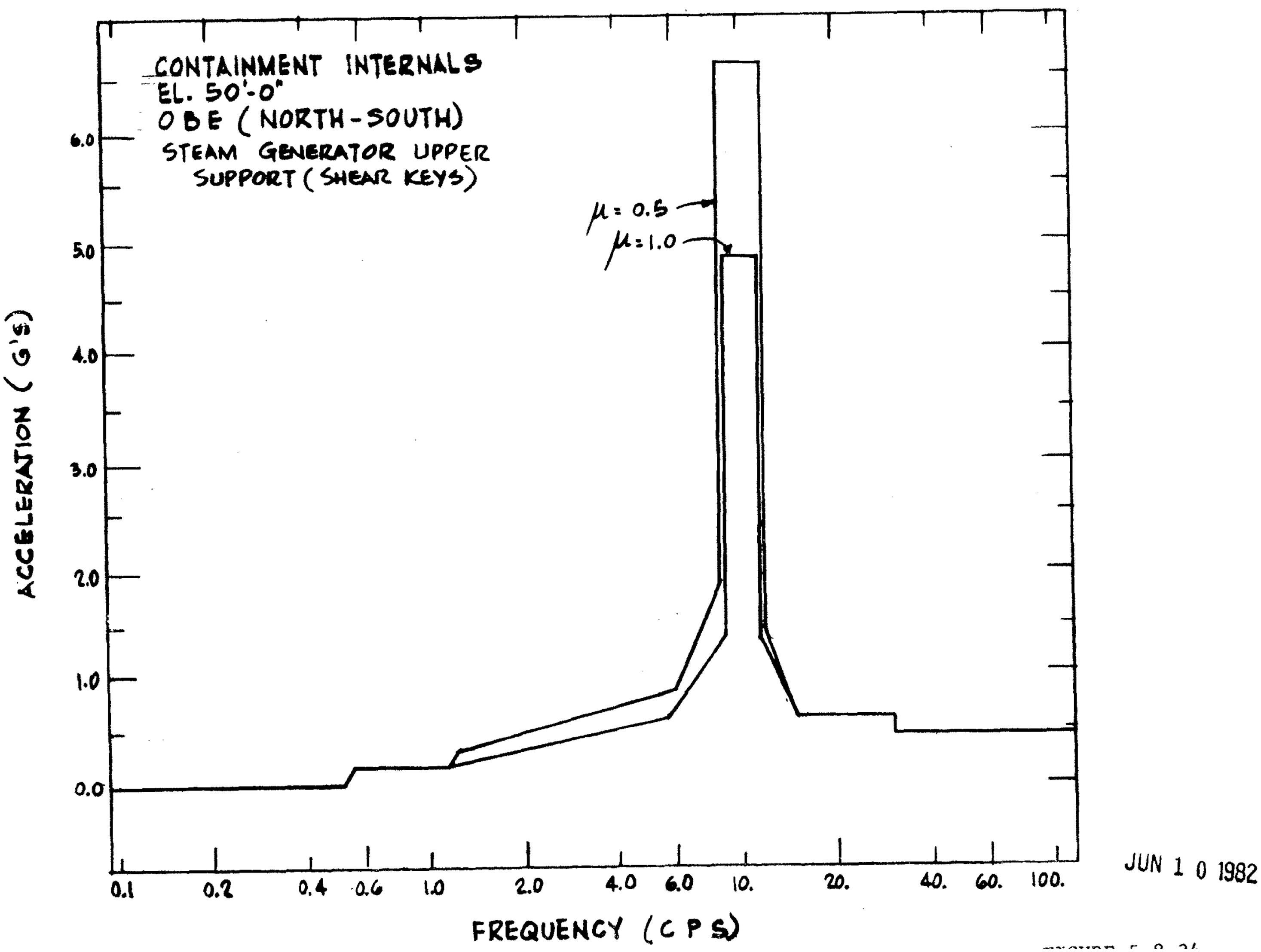

FIGURE 5.8–35 CONTAINMENT INTERNALS ELEVATION 50 FEET 0 INCHES - OBE (EAST-WEST) STEAM GENERATOR UPPER SUPPORT (SHEAR KEYS)

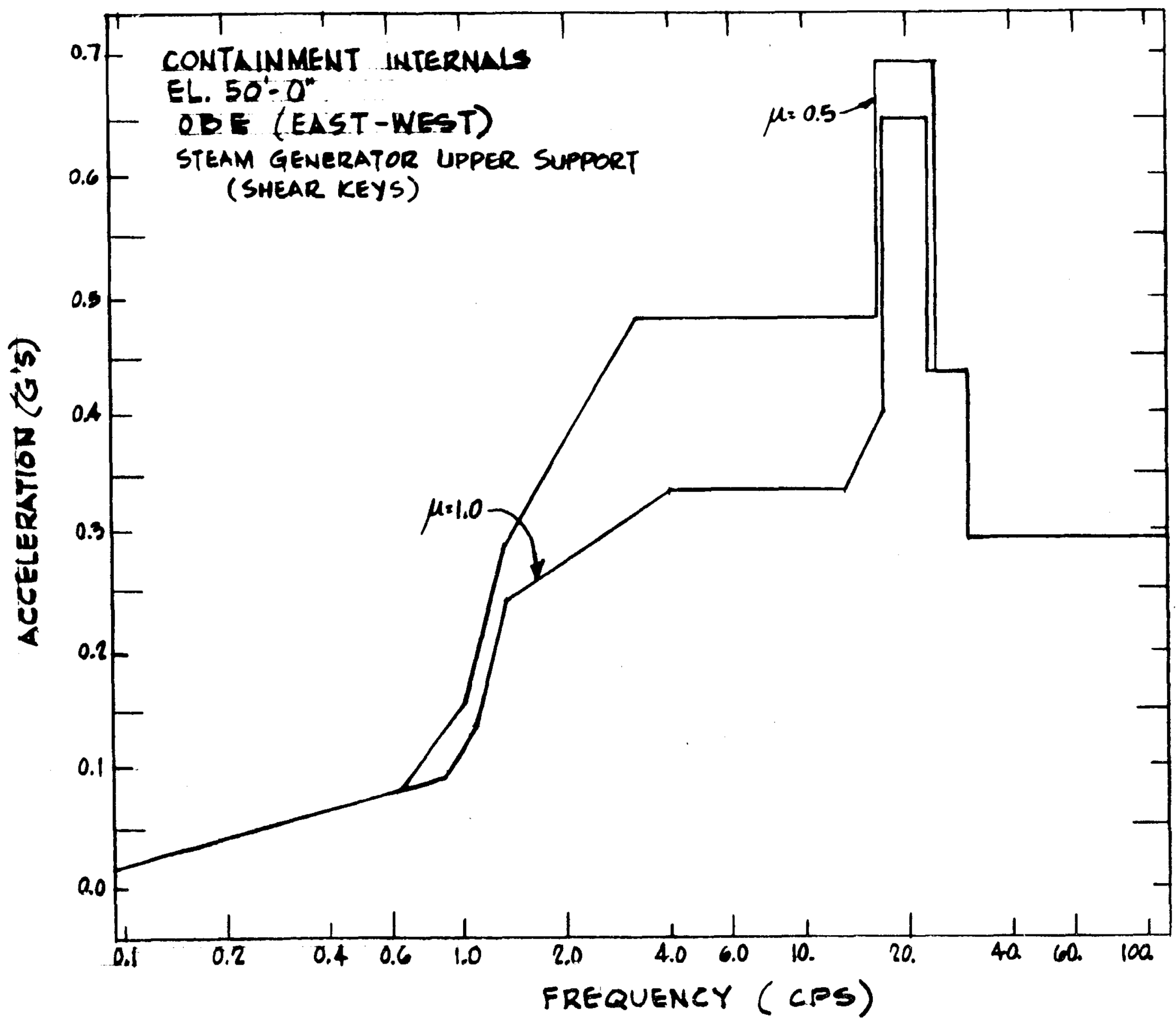

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FIGURE 5.8–36 CONTAINMENT INTERNALS ELEVATION0 FEET 0 INCHES - OBE (NORTH-SOUTH) STEAM GENERATOR LOWER SUPPORT

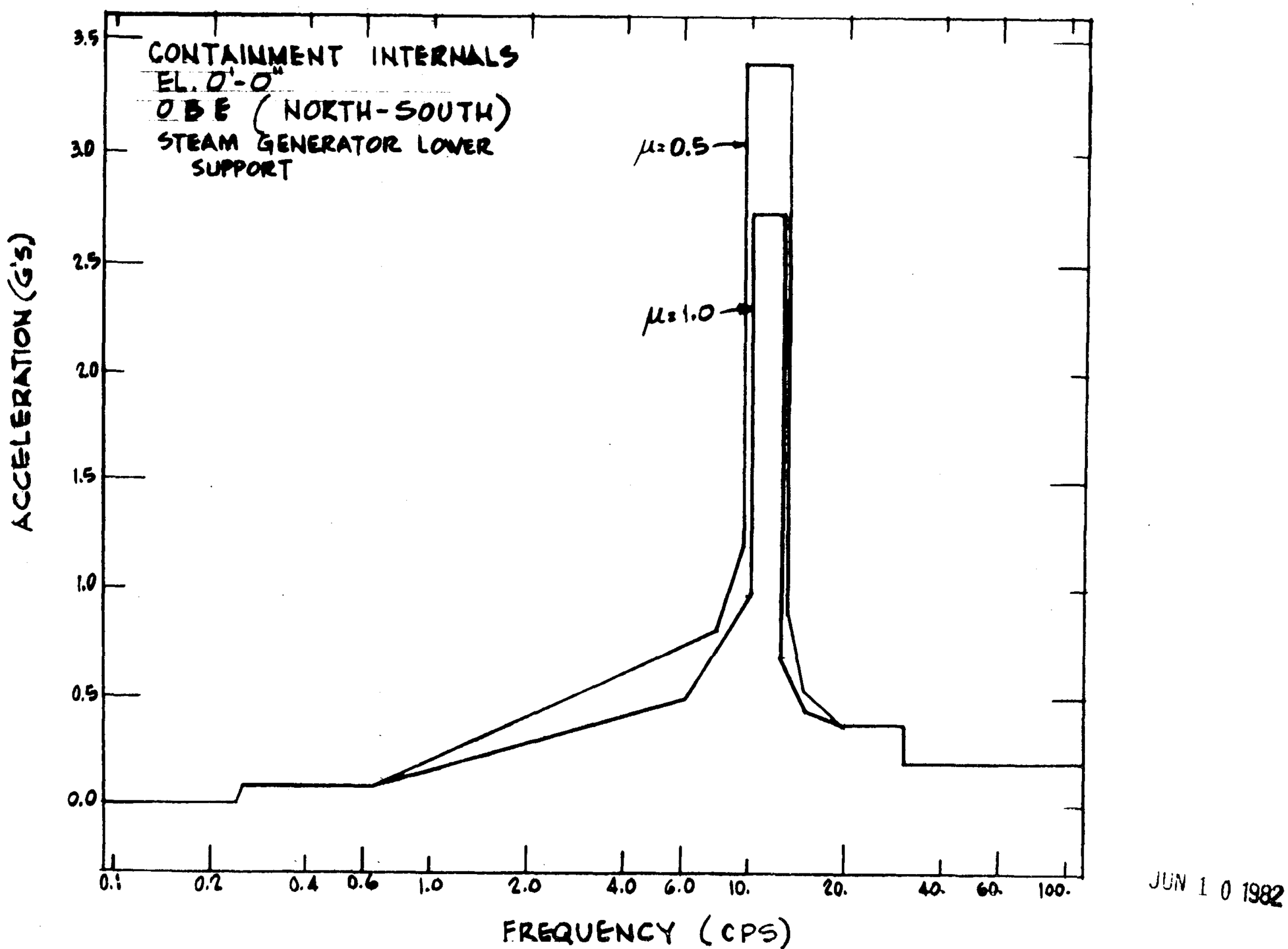

FIGURE 5.8-36

FIGURE 5.8–37 CONTAINMENT INTERNALS ELEVATION 0 FEET 0 INCHES - OBE (EAST-WEST) STEAM GENERATOR LOWER SUPPORT

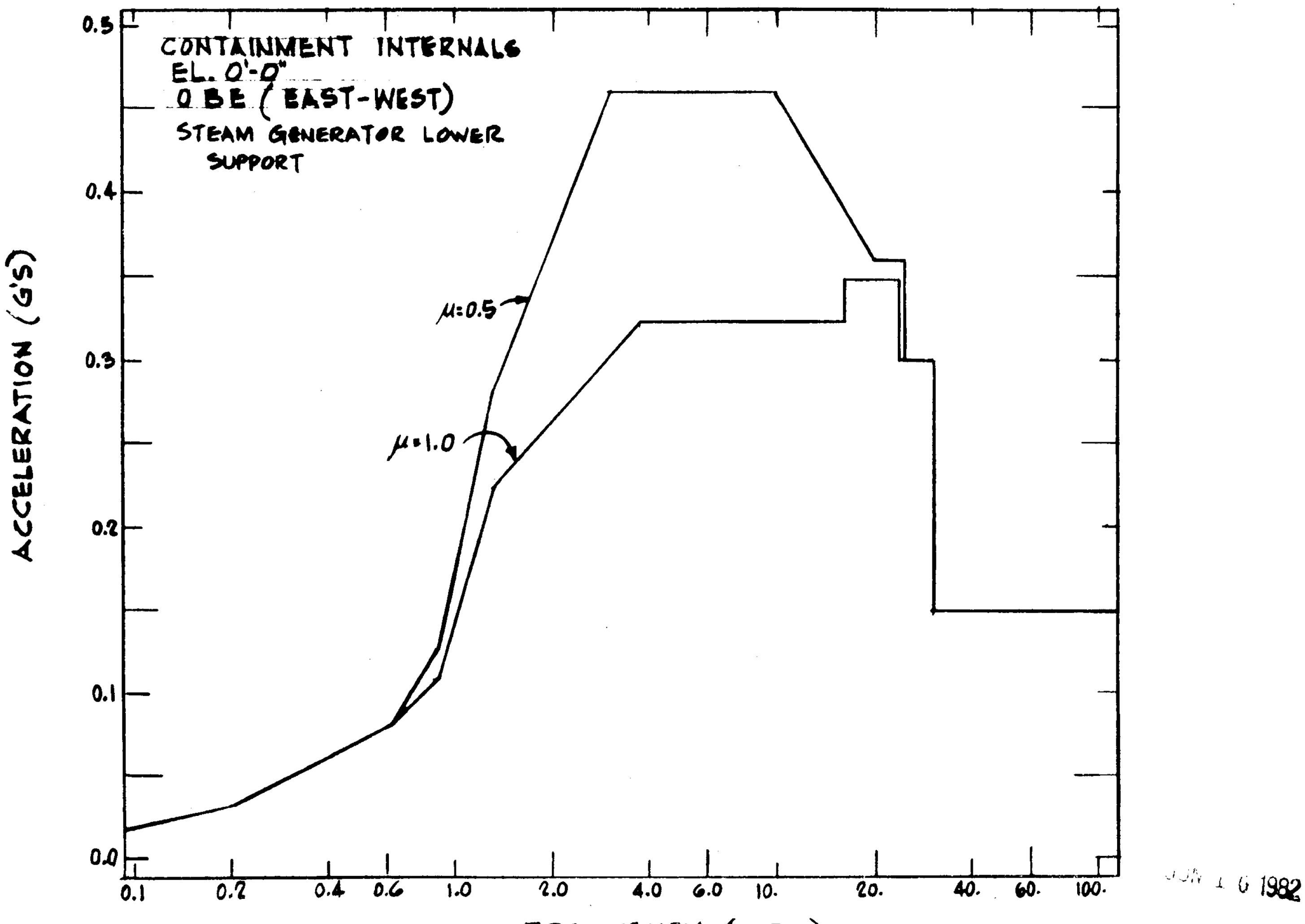

FIGURE 5.8-37

FIGURE 5.8–38 AUXILIARY BUILDING MODEL

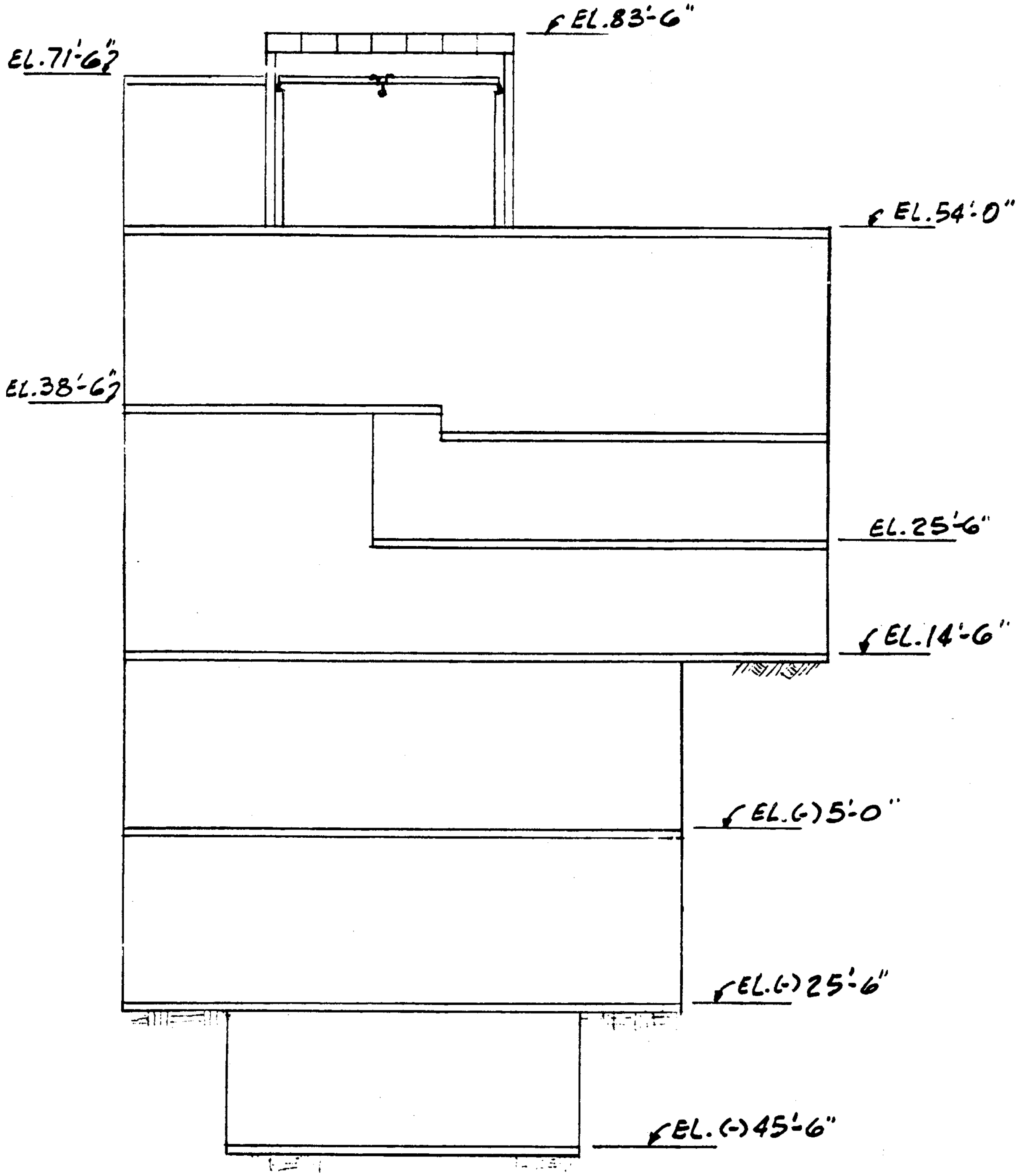

FIGURE 5.8–39 MODE SHAPES & FREQUENCIES AUXILIARY BUILDING - NORTH-SOUTH

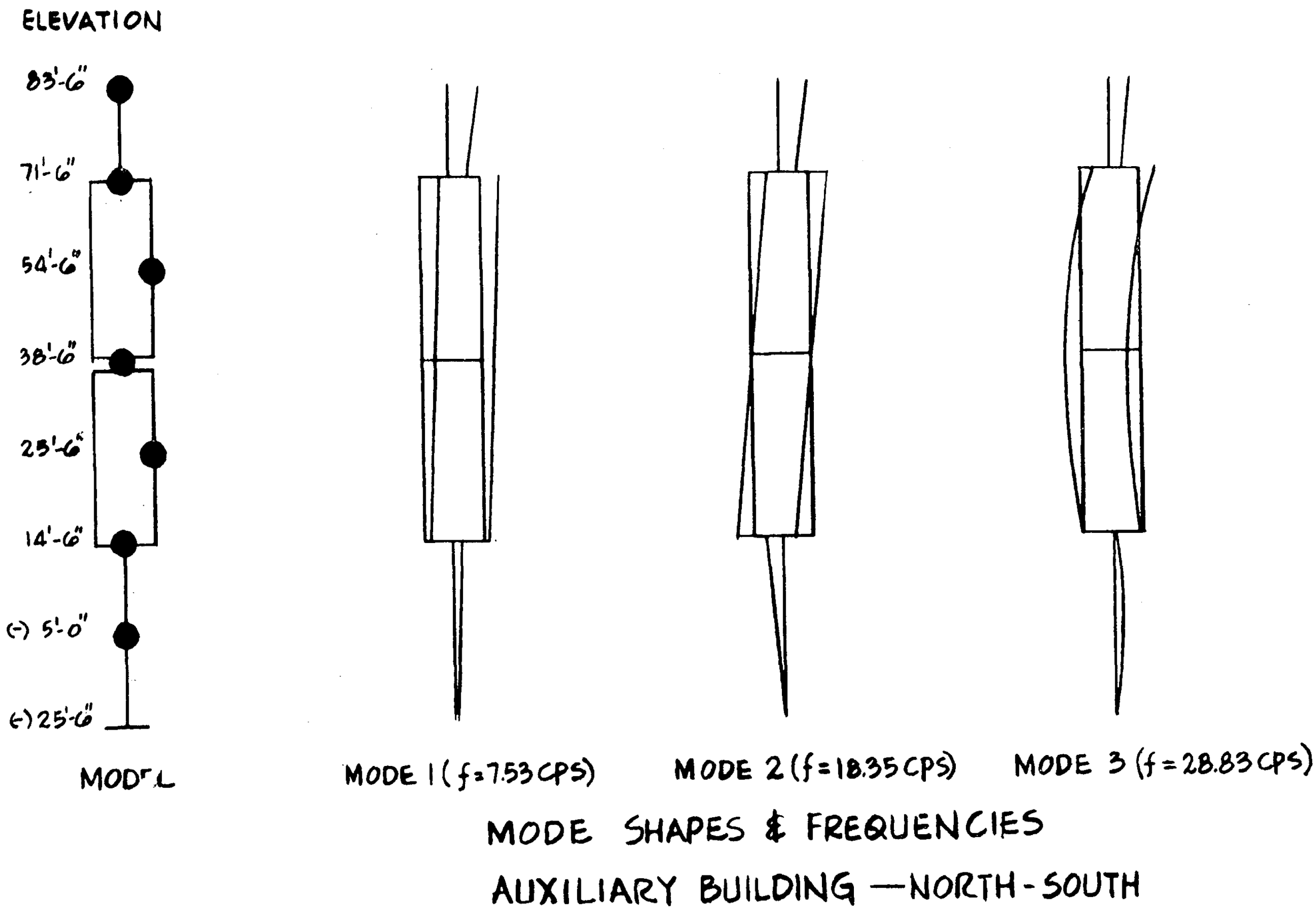

FIGURE 5.8–40 AUXILIARY BUILDING - NORTH-SOUTH OBE (DBE)

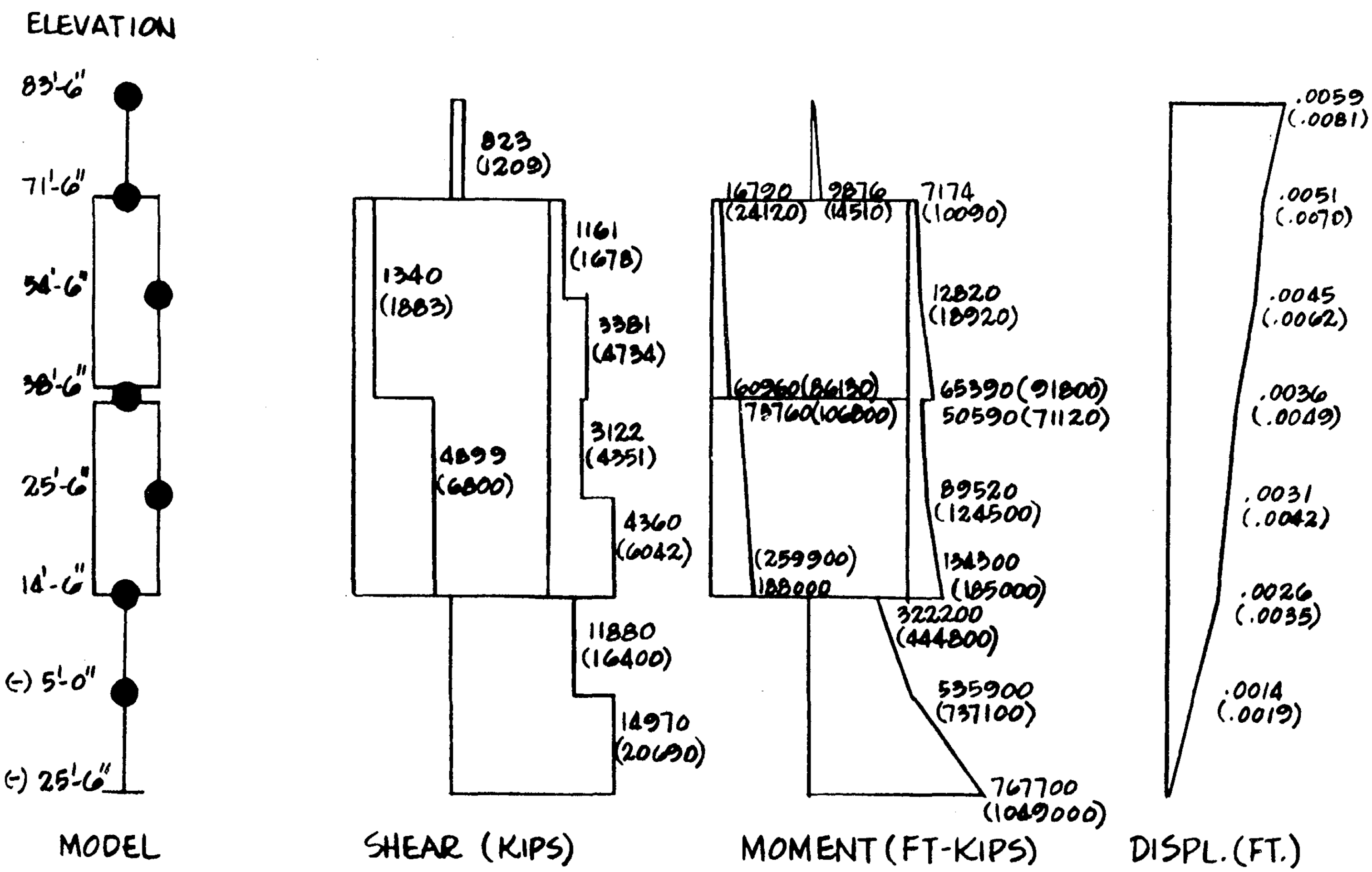

FIGURE 5.8–41 MODE SHAPES & FREQUENCIES AUXILIARY BUILDING - EAST-WEST

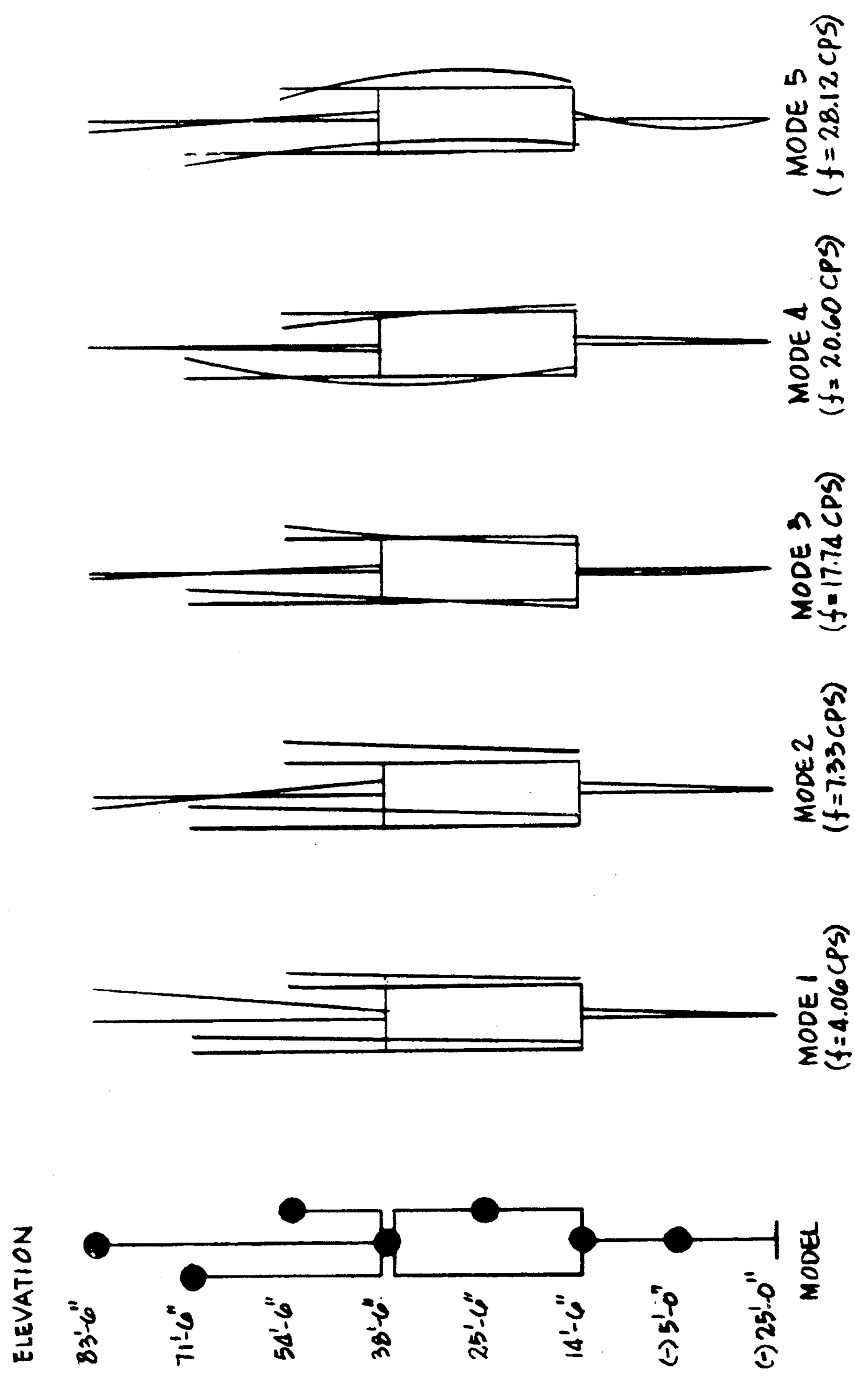

FIGURE 5.8–42 AUXILIARY BUILDING - EAST-WEST OBE (DBE)

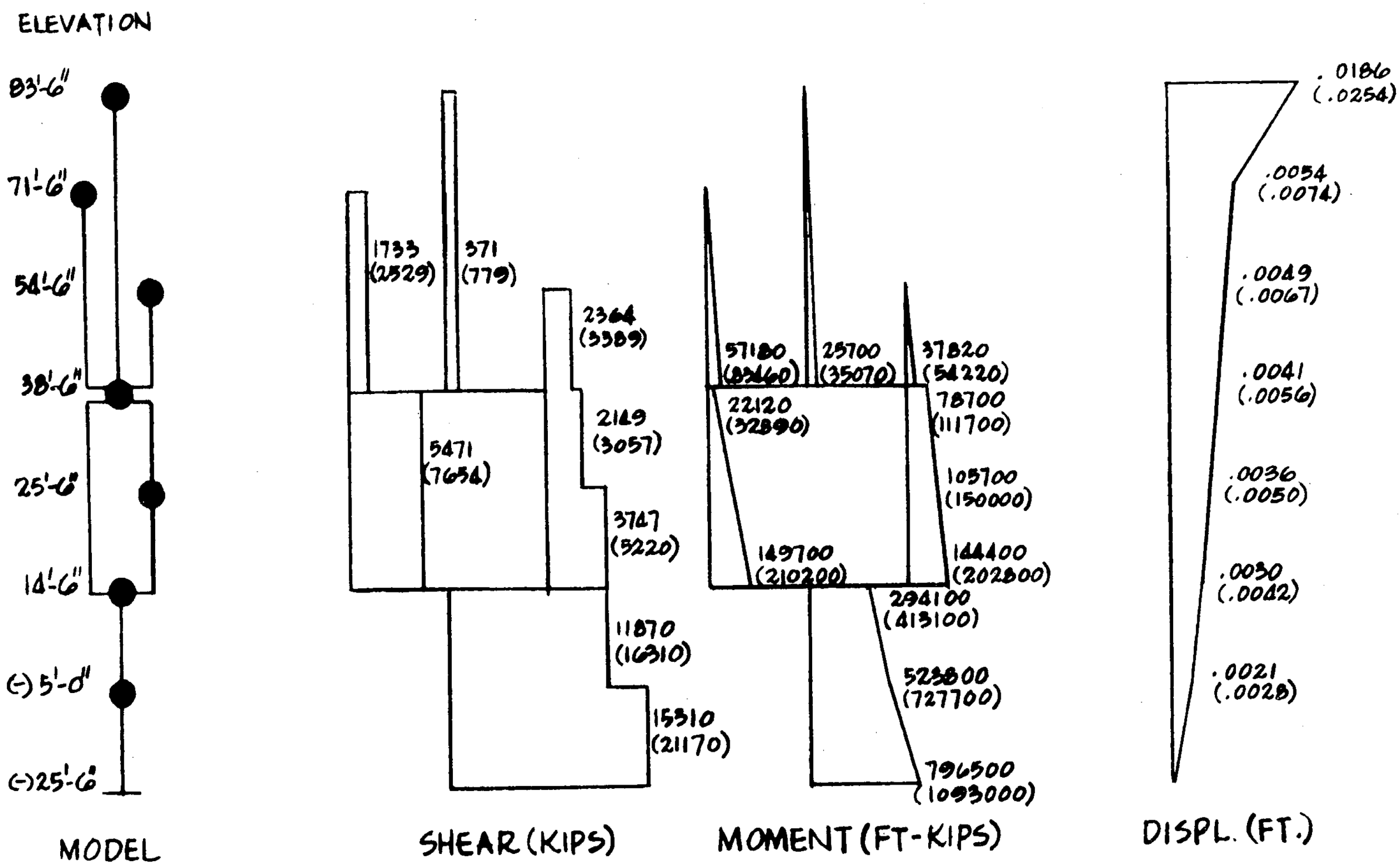

FIGURE 5.8–43 AUXILIARY BUILDING ELEVATION 14 FEET 6 INCHES

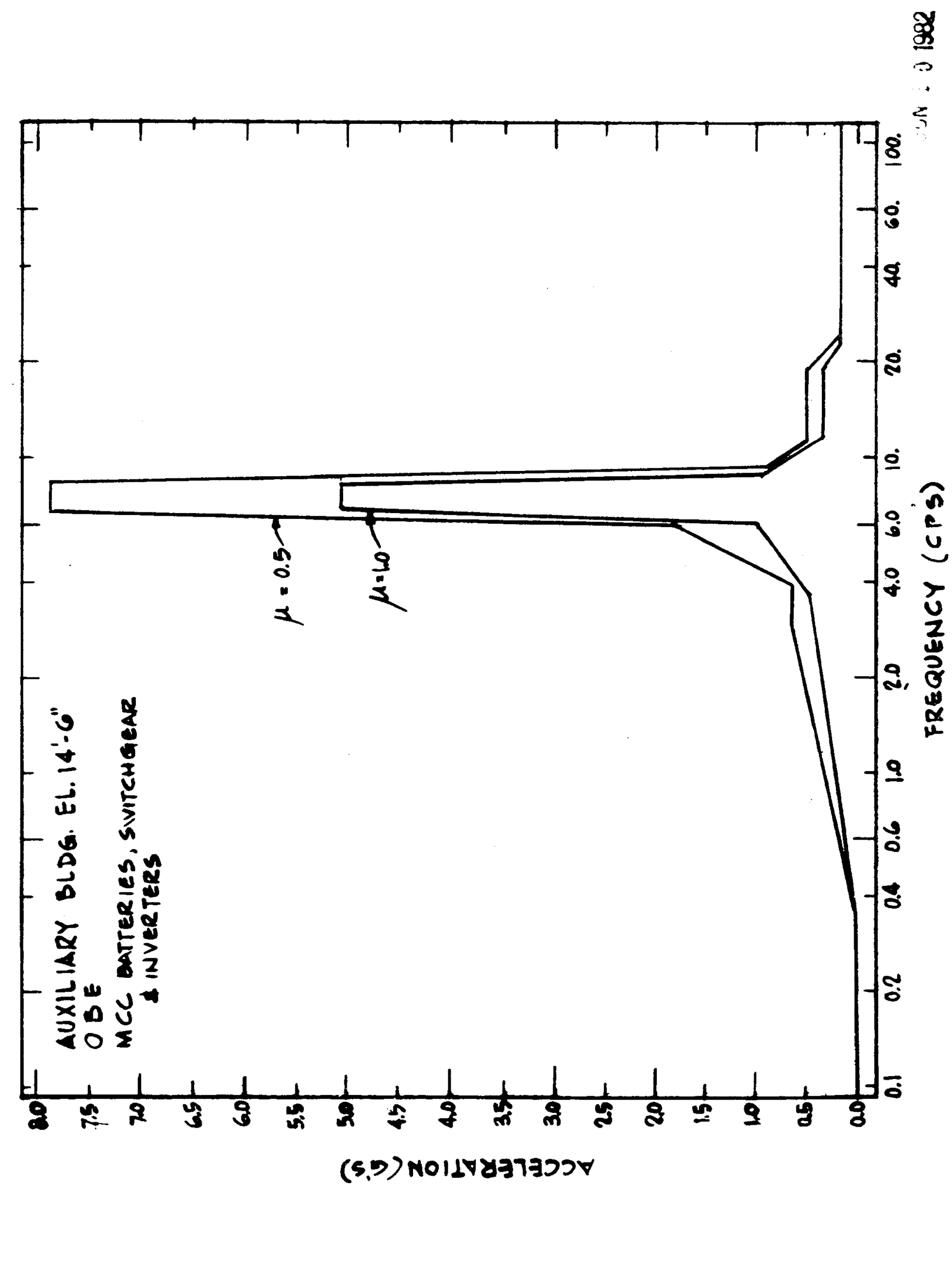

FIGURE 5.8–44 AUXILIARY BUILDING ELEVATION 38 FEET 6 INCHES

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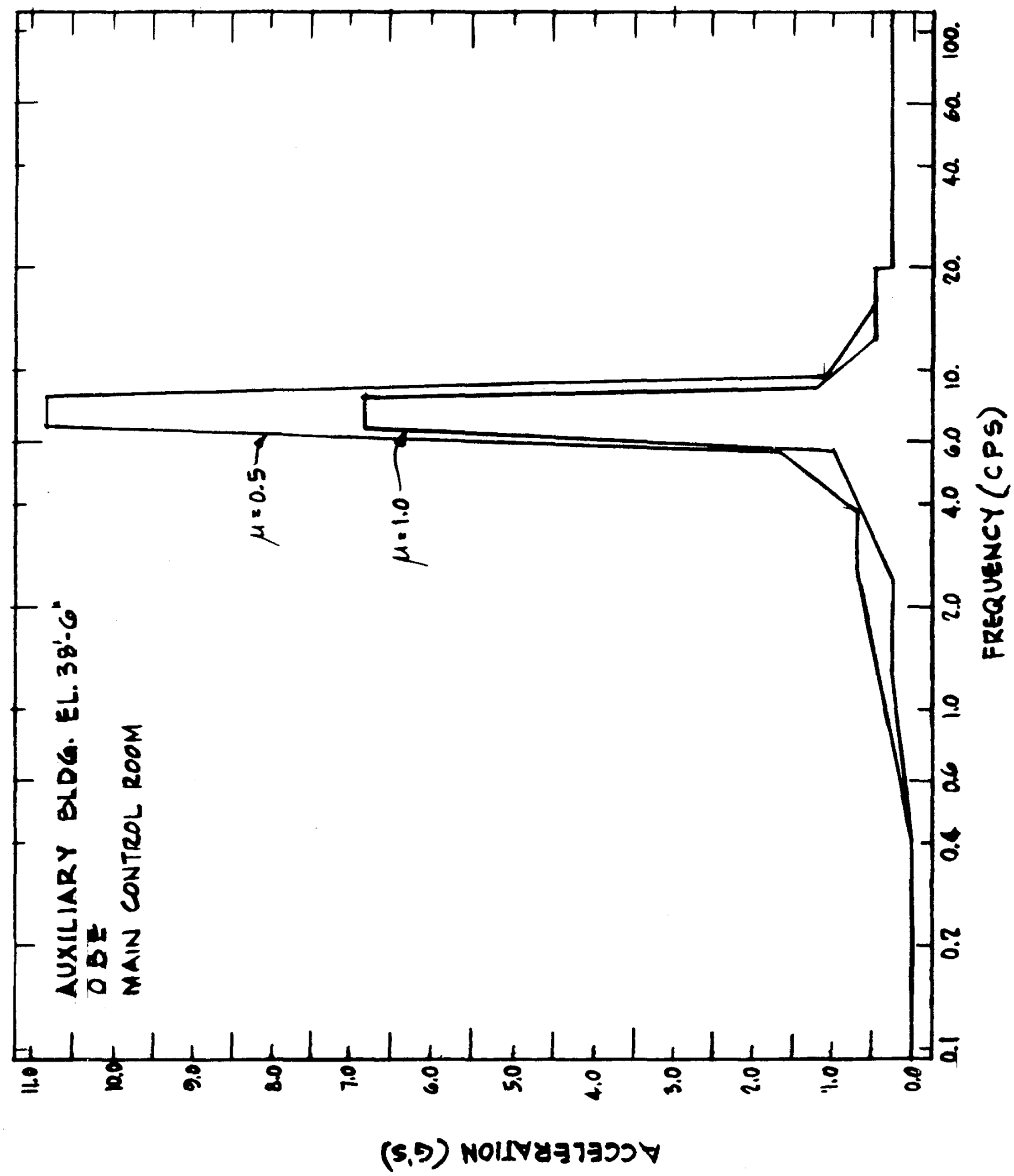

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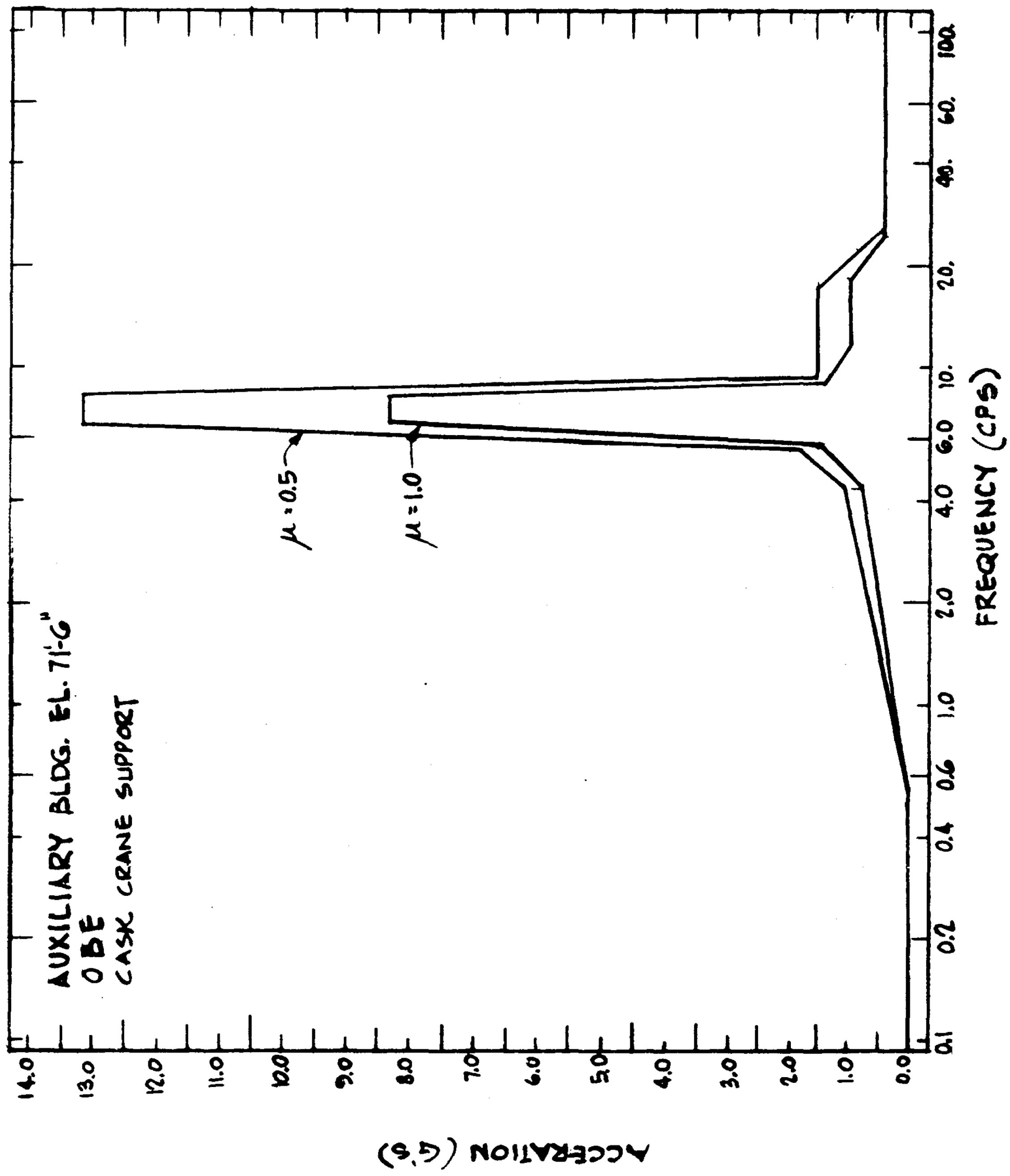

FIGURE 5.8–45 AUXILIARY BUILDING ELEVATION 71 FEET 6 INCHES

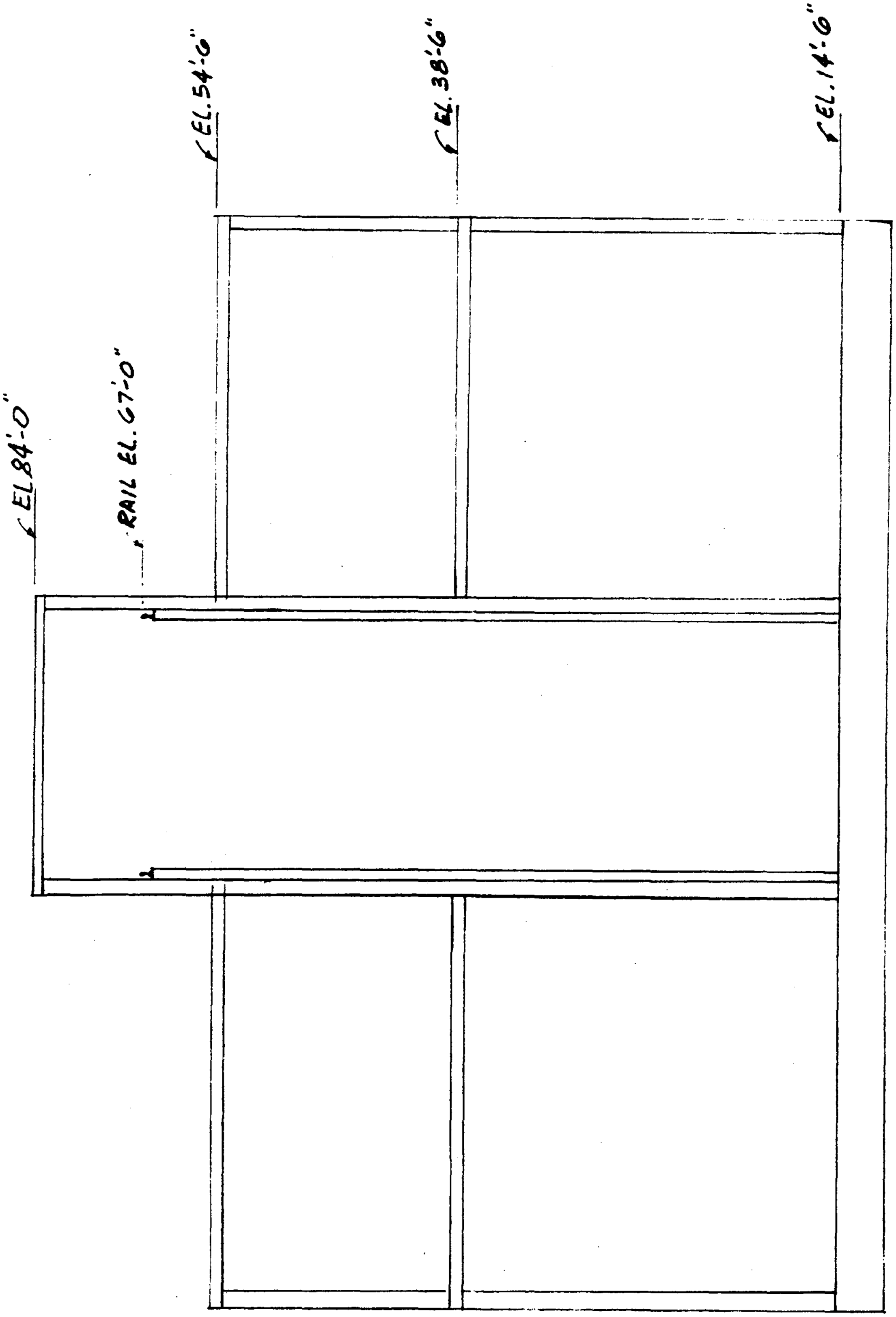

FIGURE 5.8–46 WAREHOUSE MODEL

FIGURE 5.8–47 MODE SHAPES & FREQUENCIES WAREHOUSE BLDG. - EAST-WEST

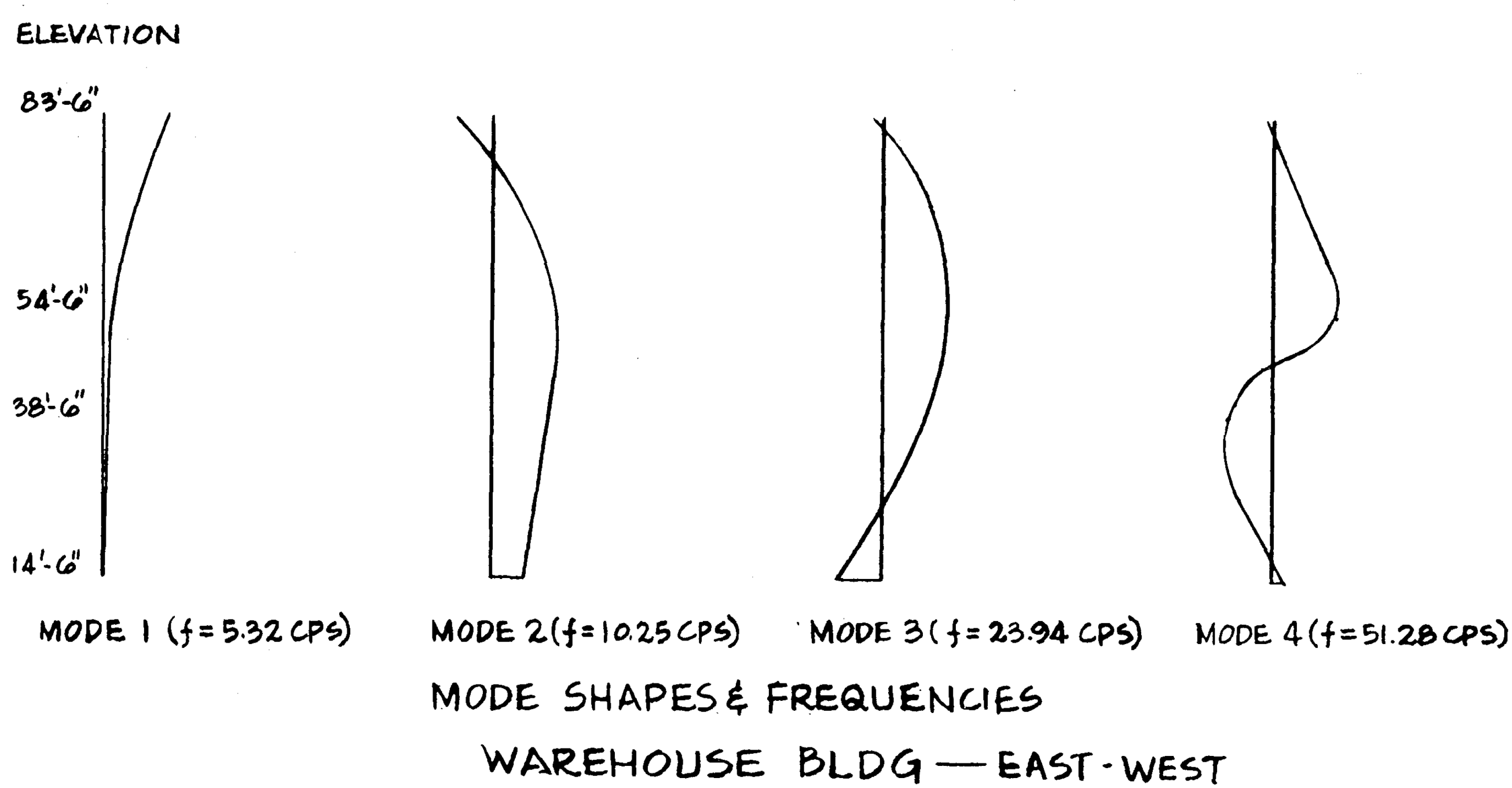

FIGURE 5.8–48 MODE SHAPES & FREQUENCIES WAREHOUSE BLDG. - NORTH-SOUTH

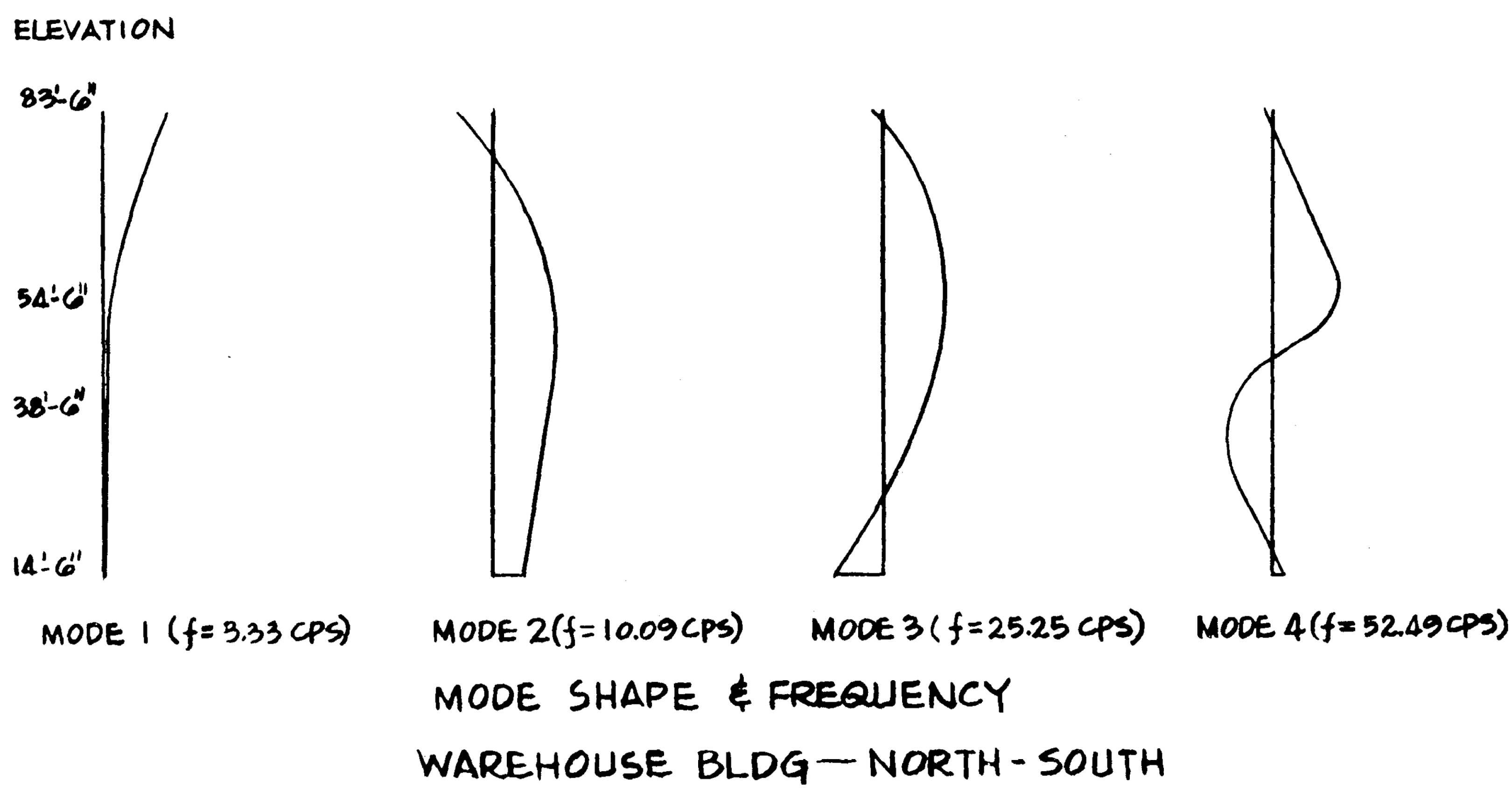

FIGURE 5.8–49 WAREHOUSE BLDG. - OBE (DBE)

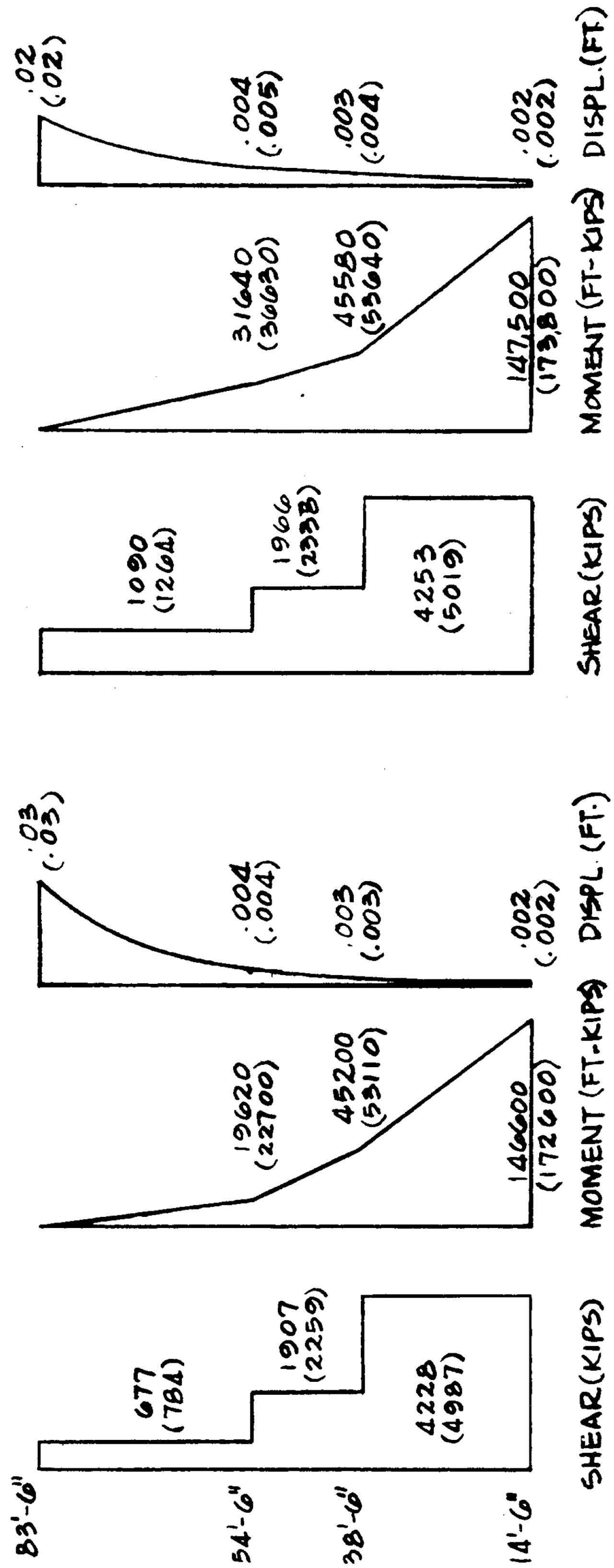

FIGURE 5.8–50 WAREHOUSE ELEVATION 38 FEET 6 INCHES

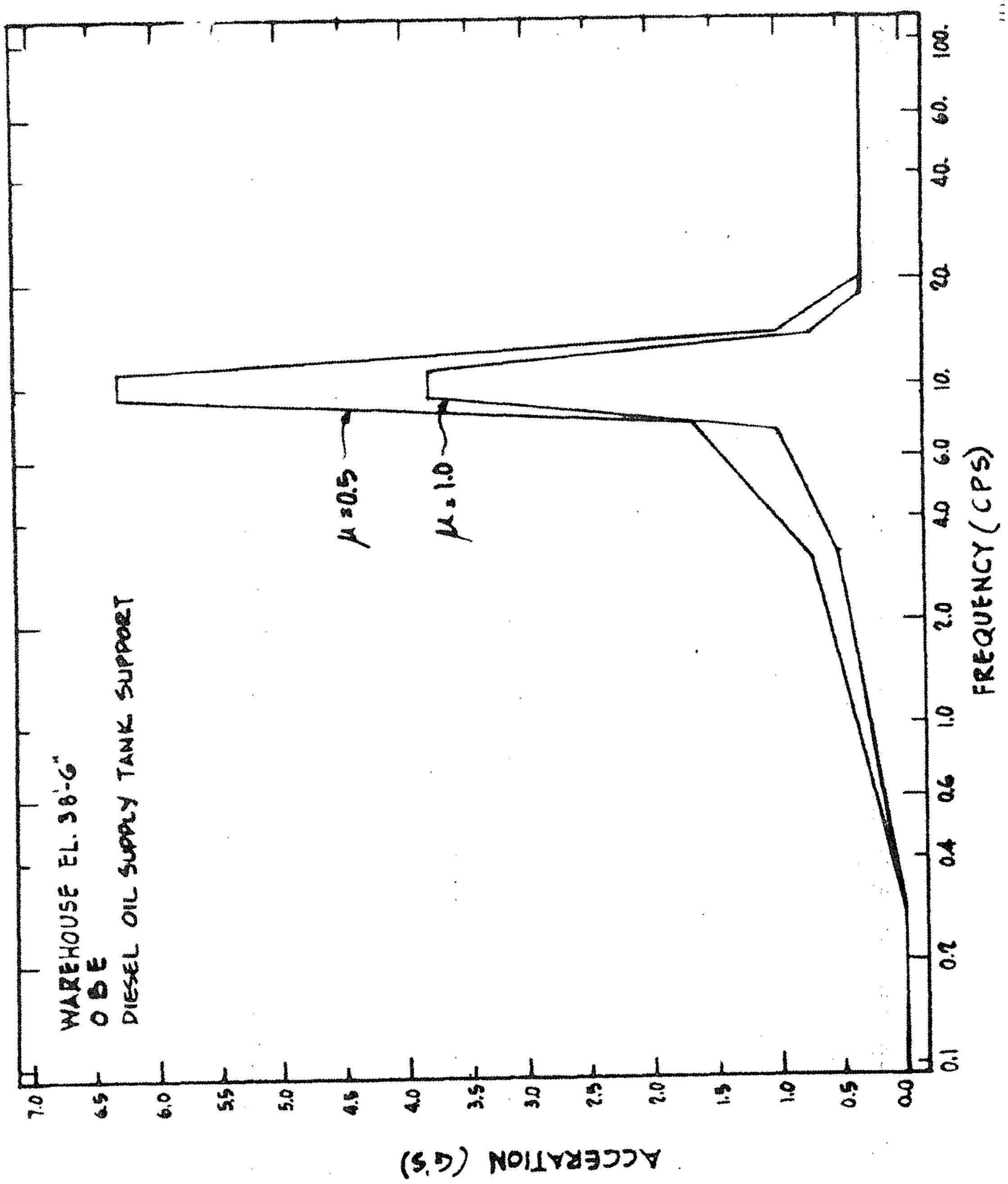

FIGURE 5.8–51 TURBINE BUILDING MODEL

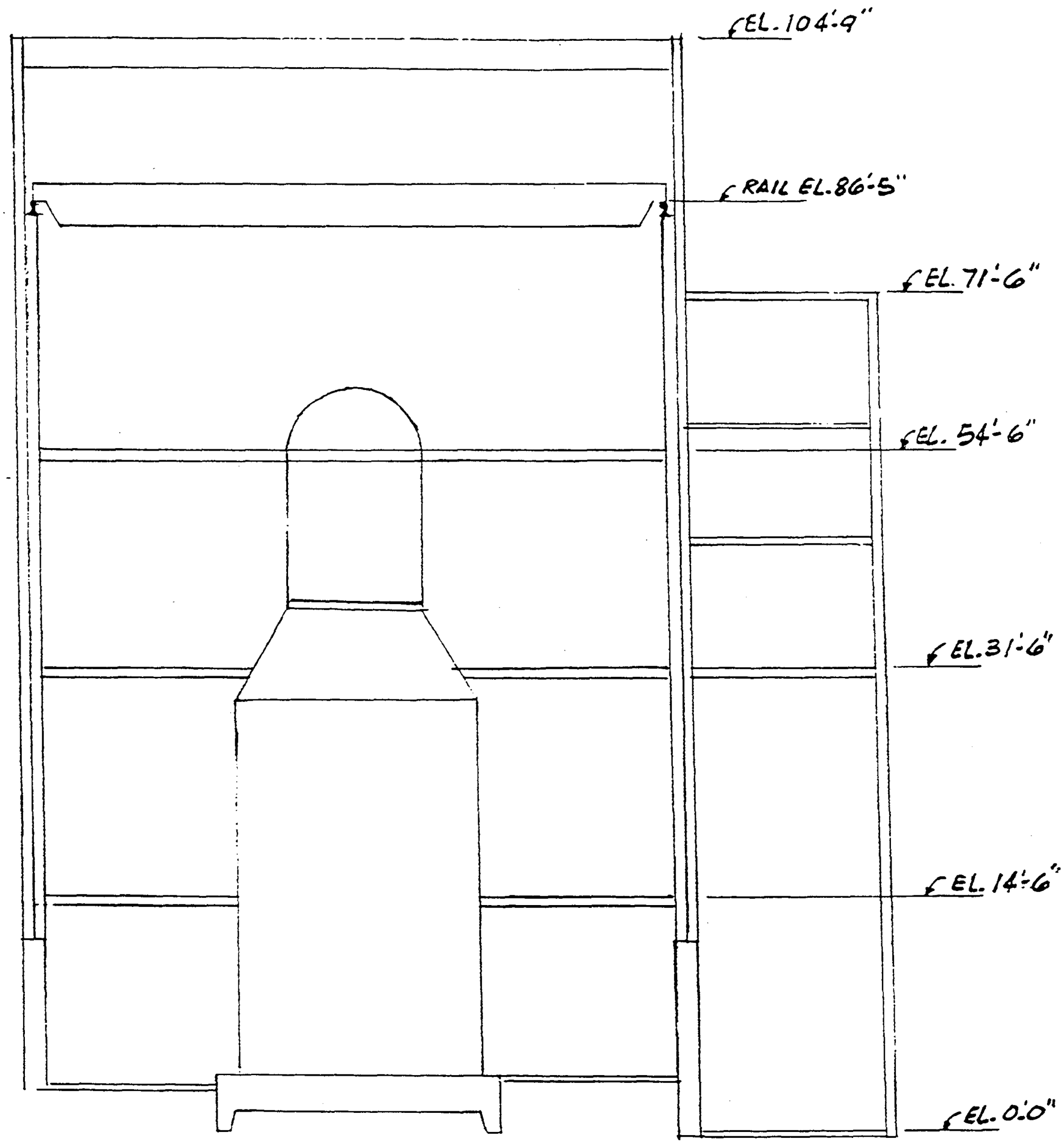

FIGURE 5.8–52 MODE SHAPES & FREQUENCIES TURBINE BLDG. - NORTH-SOUTH

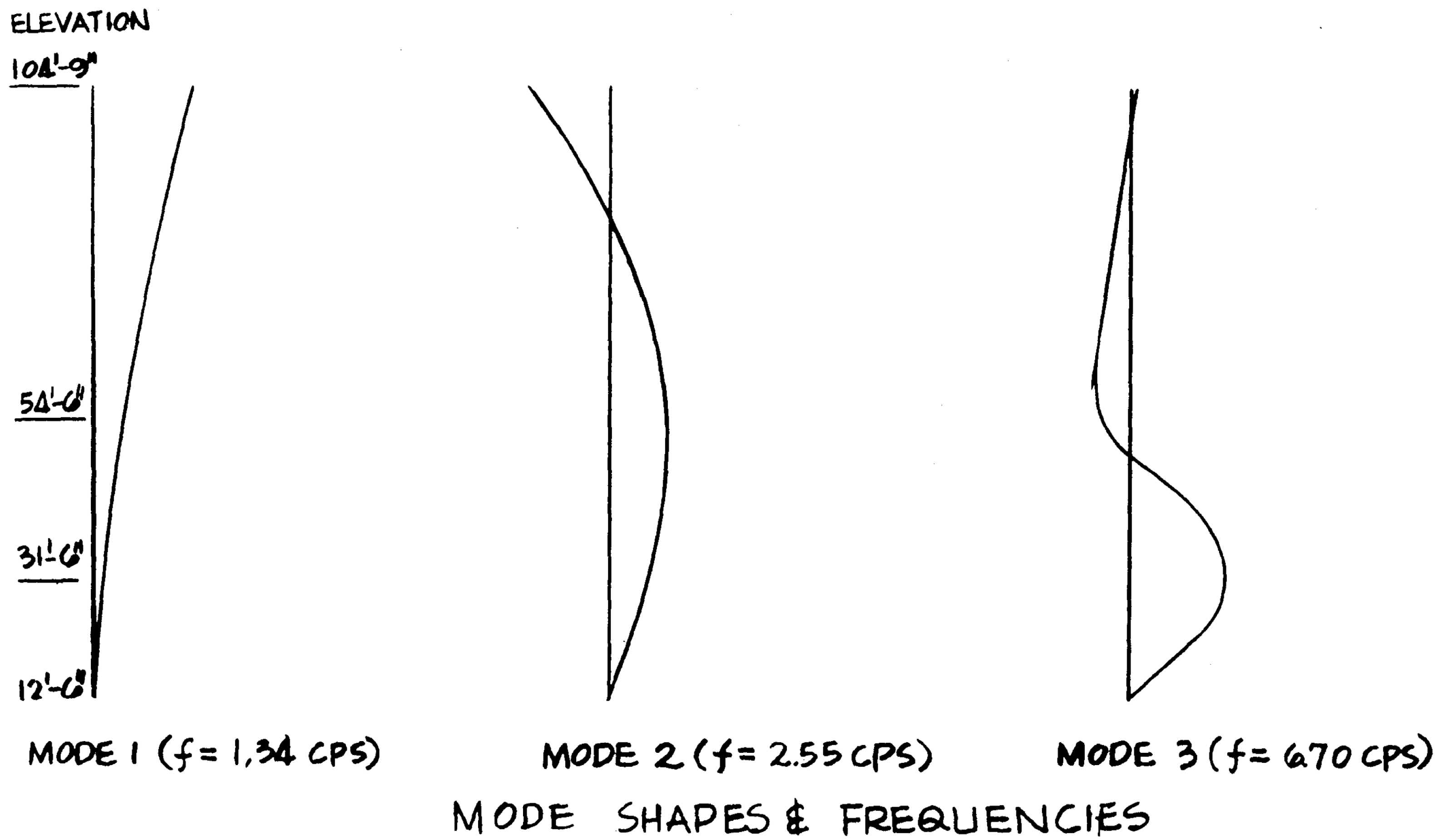

FIGURE 5.8–53 TURBINE BLDG. OBE (DBE) EAST-WEST

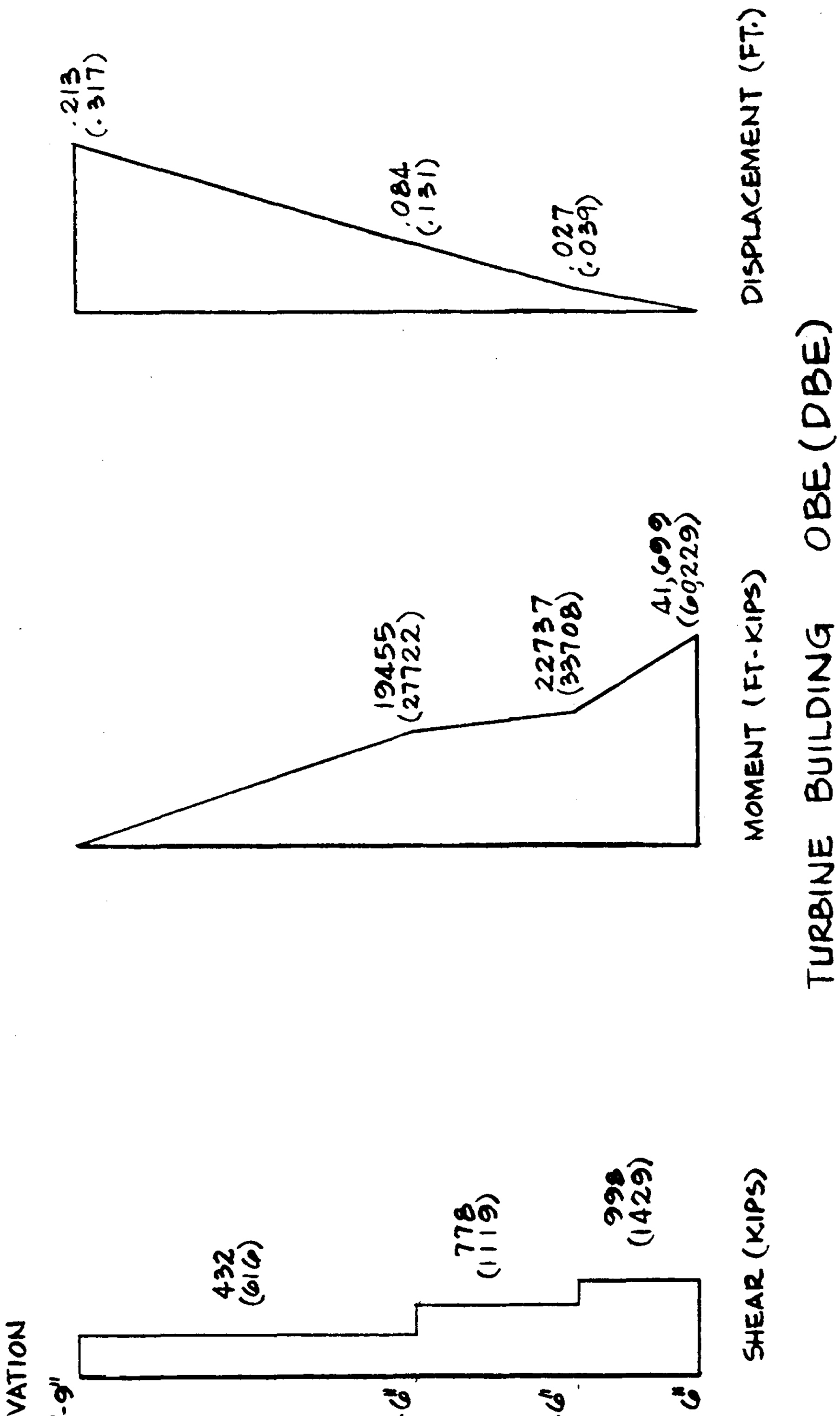

FIGURE 5.8–54 MODE SHAPES & FREQUENCIES TURBINE BLDG. - EAST-WEST

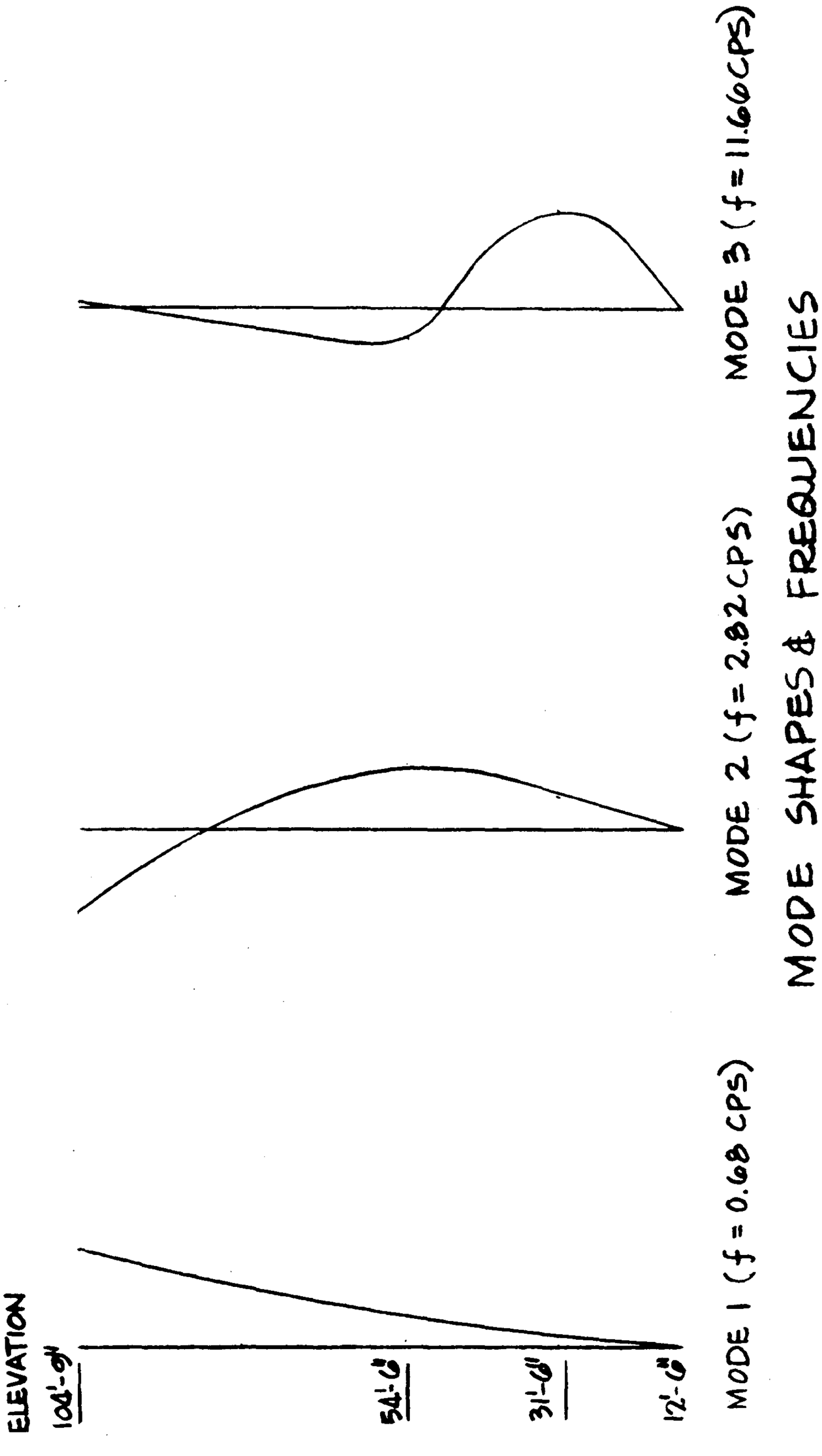

FIGURE 5.8–55 TURBINE BLDG. OBE (DBE) NORTH-SOUTH

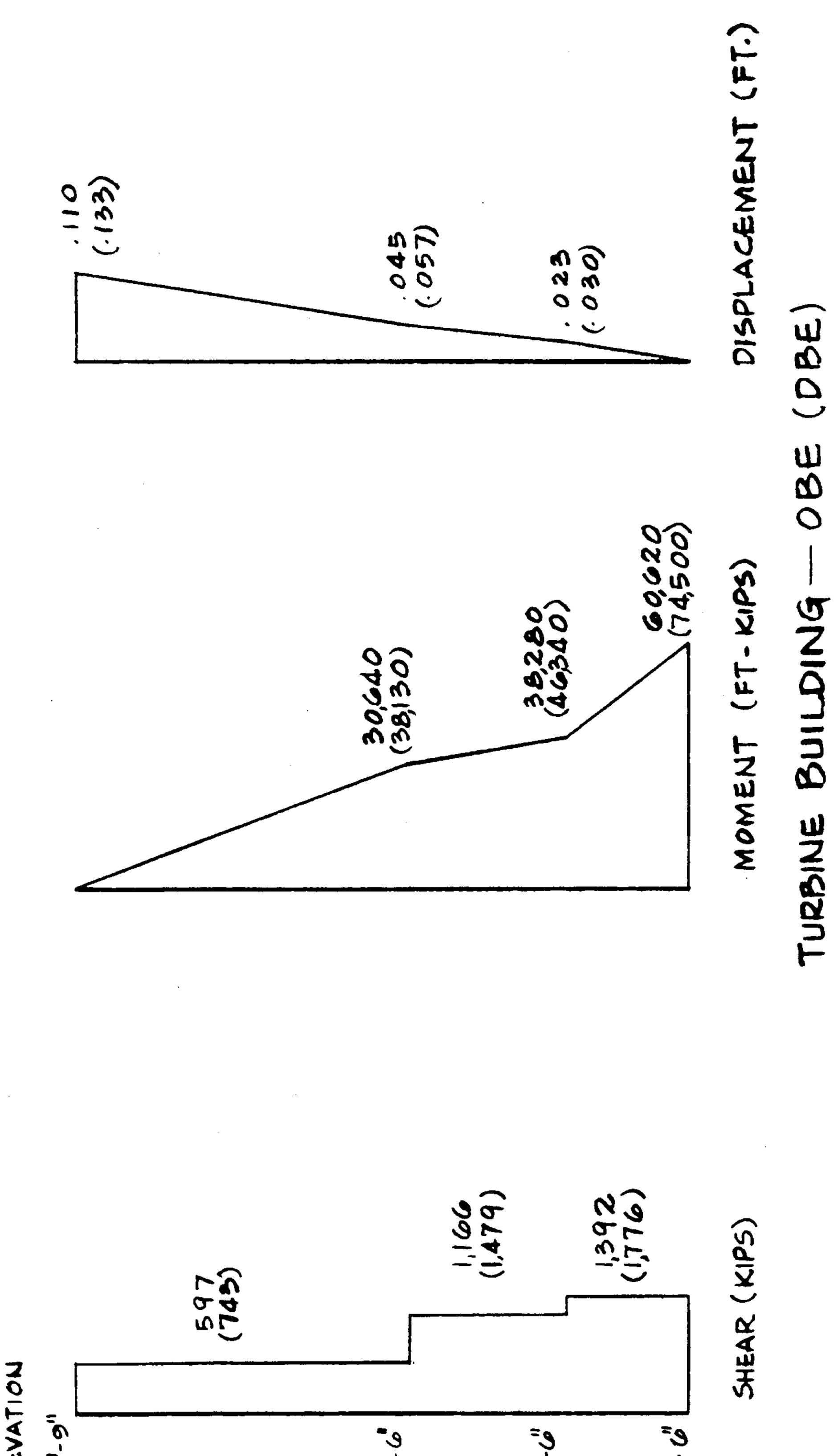

FIGURE 5.8–56 TURBINE BUILDING ELEVATION 31 FEET 6 INCHES

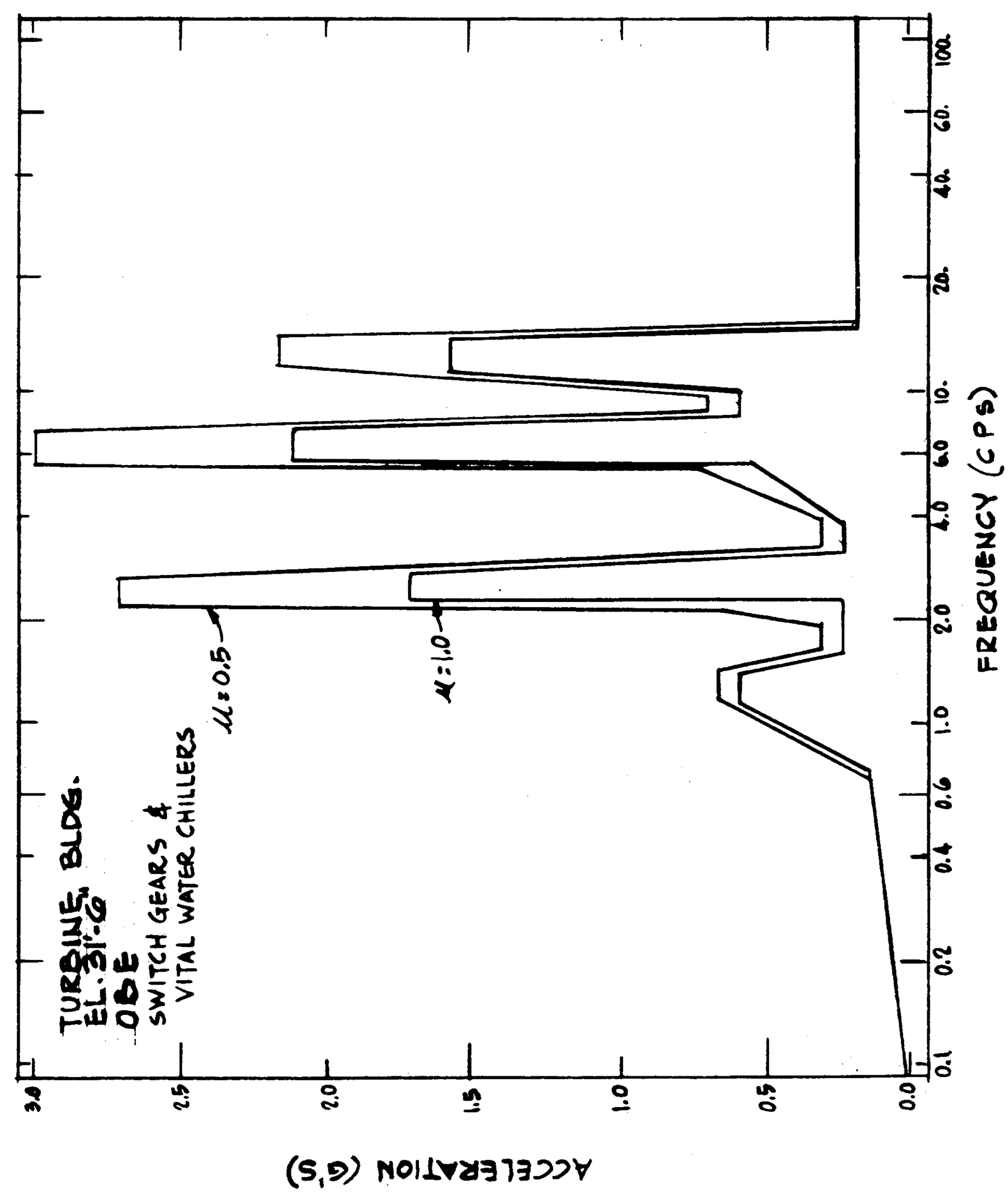

FIGURE 5.8–57 TURBINE BUILDING CRANE - OBE

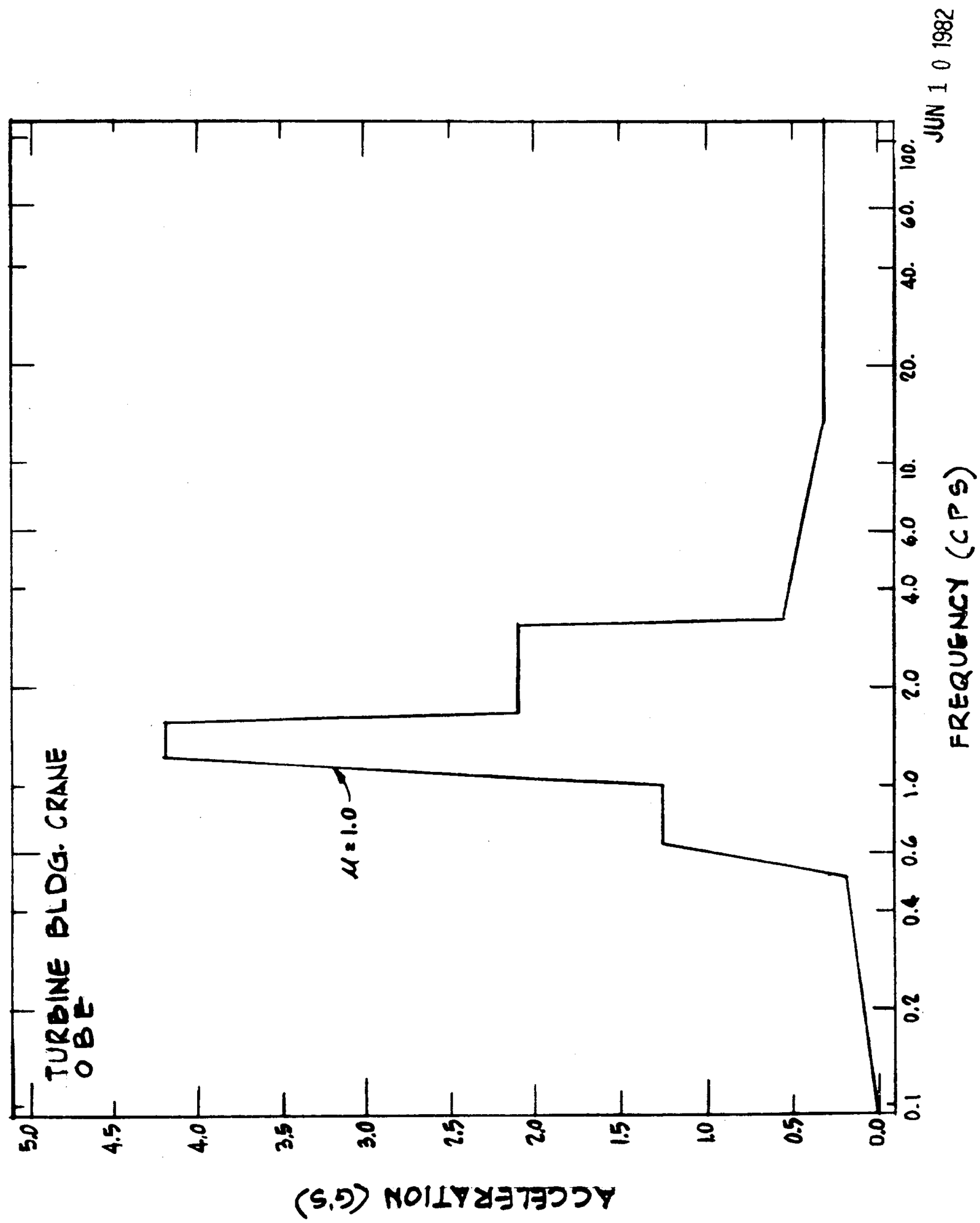

FIGURE 5.8–58 INTAKE STRUCTURE MODEL

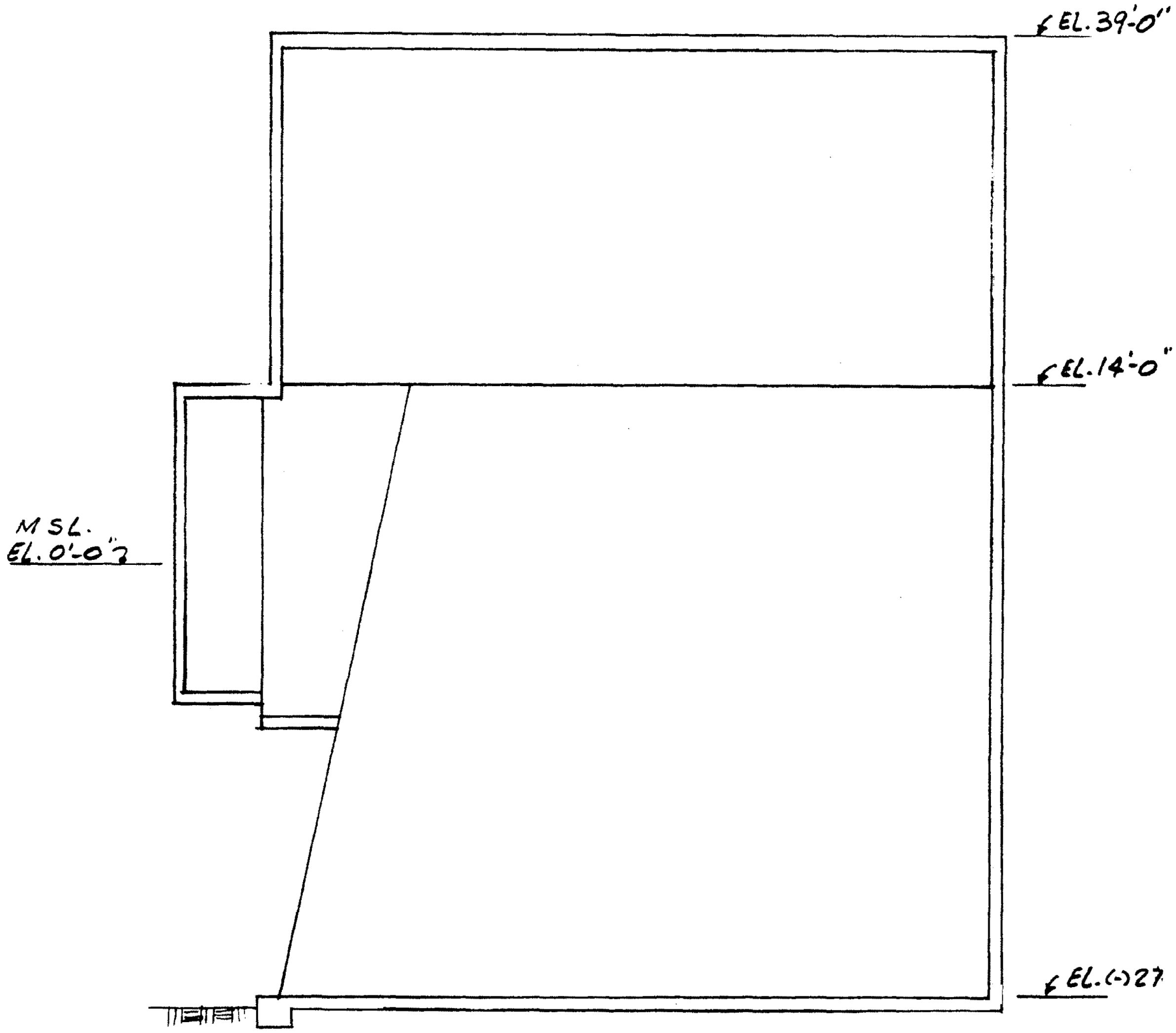

FIGURE 5.8–59 INTAKE STRUCTURE OBE (DBE)

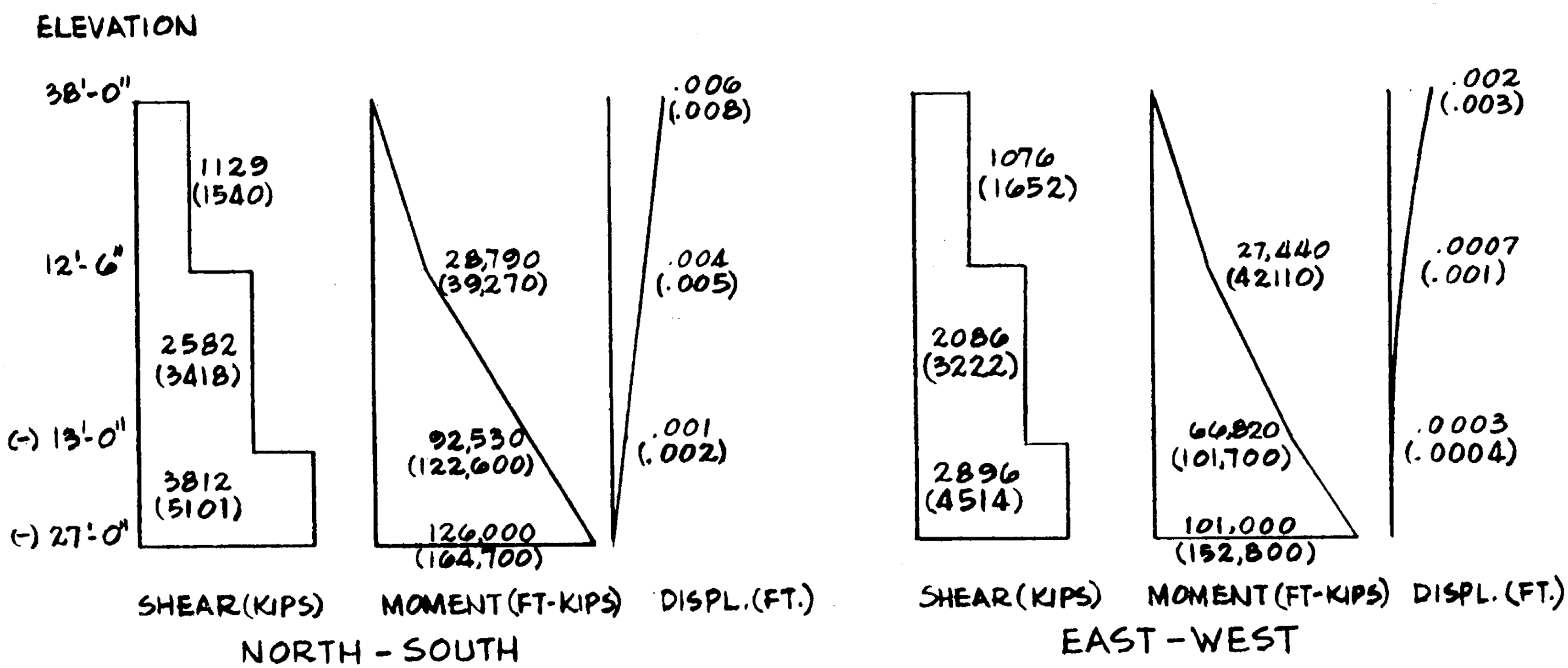

FIGURE 5.8–60 MODE SHAPES & FREQUENCIES INTAKE STRUCTURE

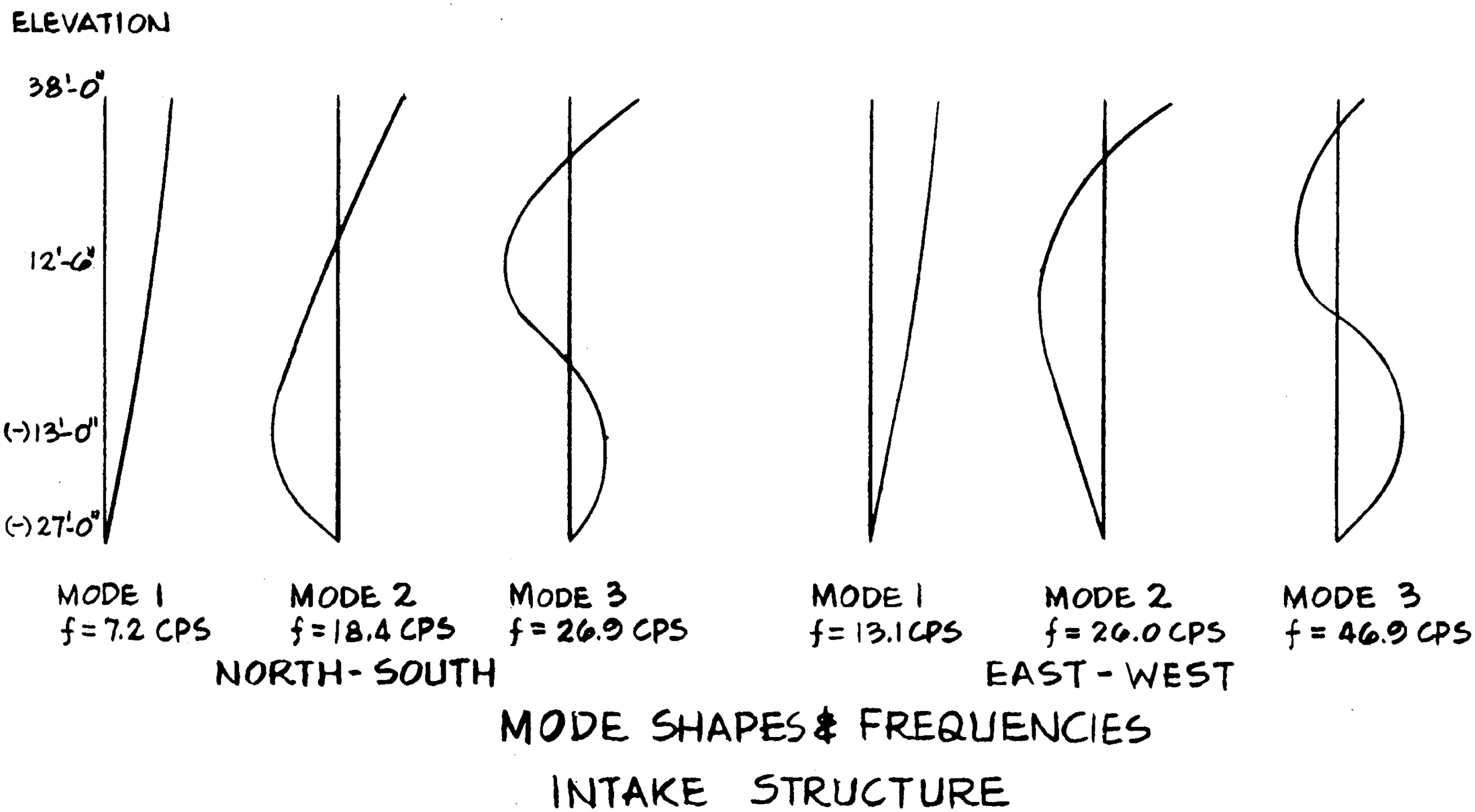

FIGURE 5.8–61 INTAKE STRUCTURE ELEVATION 14 FEET 0 INCHES OBE

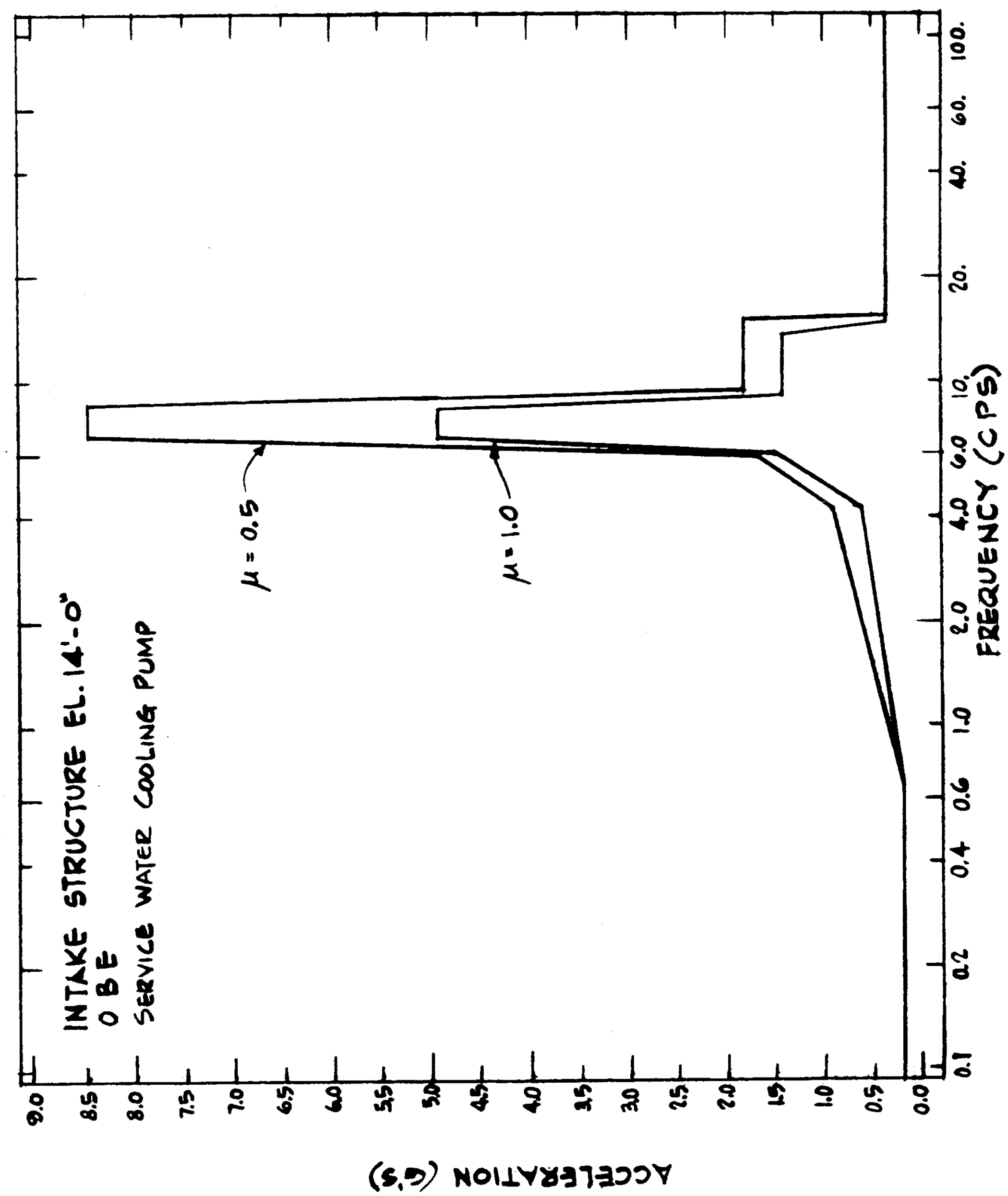

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FIGURE 5.8-61

5.9 CONSTRUCTION PRACTICE AND QUALITY ASSURANCE

5.9.1 APPLICABLE CONSTRUCTION CODES

The following codes of practice are used to establish standards for construction procedures:

ACI 214-1	Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-65)
ACI 301-1	Specification for Structural Concrete for Buildings (ACI 301-66)
ACI 306-1	Recommended Practice for Cold Weather Concreting (AC 306-66)
ACI 315	Manual of Standard Practice for Detailing Reinforced Concrete Structures
ACI 318-1	Building Code Requirements for Reinforced Concrete (ACI 318-63)
ACI 347-1	Recommended Practice for Concrete Formwork (ACI 347-68)
ACI 305-1	Recommended Practice for Hot Weather concreting (ACI 605-59)
ACI 211-1	Recommended Practice for Selecting Proportions for Concrete (ACI6B-54)
ACI 304-1	Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-59)
ACS	Manual of Concrete Inspection
PCI	Inspection Manual
AISC	Manual of Steel Construction
AWS	Code for Welding in Building Construction (D1.0-69)
AWS	Specifications for Welded Highway and Railroad Bridges (D2.0-69)
ASME	Boiler and Pressure Vessel Code, Section VIII, Part UW – Requirements for Unfired Pressure Vessels. Fabricated by Welding.
CRD	U.S. Army Corps of Engineers Waterways Experiment Station

Dimensional tolerances for construction, unless otherwise stated in design drawings, are in compliance with the ACI 301-66 and ACI 318-63 for placing reinforcing bars and concrete, and with the AISC Code of Standard Practice for erection of steel.

5.9.2 QUALITY ASSURANCE PROGRAM

A Quality Assurance Program has been developed and implemented to assure conformance to regulatory requirements and accepted industry standards. This is generally explained in FSAR Section 12.8.

5.9.3 CONSTRUCTION MATERIALS INSPECTION AND INSTALLATION

Basically, materials used in the construction of the structures are as follows:

a. Concrete

b. Reinforcing steel

c. Structural and miscellaneous steel

d. Prestressing steel tendons, anchorages and sheaths

e. Steel liner plate

f. Interior coatings

The basic specifications for material inspection and installation are discussed in the following sections.

5.9.3.1 Concrete

All concrete work is done in accordance with ACI 318-63, "Building Code Requirements for Reinforced Concrete," and to ACI 301-66, "Specifications for Structural Concrete for Buildings," except as otherwise stated herein or in the appropriate job specifications or design drawings.

The concrete is a dense, durable mixture of sound coarse aggregates, fine aggregates, cement, and water. In some areas, fly ash is substituted for portions of cement used in the concrete. Admixtures are added to improve the quality and workability of the plastic concrete during placement and to retard the set of the concrete. The sizes of aggregates, water-reducing additives, and slumps are selected to maintain low limits on shrinkage and creep.

The concrete is placed in a manner which assures sound concrete, free of cold joints and defects. Careful attention is given to the placing of concrete around tendon anchorage bearing plates in the containment so that high quality concrete is obtained at these critical locations.

5.9.3.1.1 Aggregates

Aggregates comply with ASTM C-33, "Specifications for Concrete Aggregates." Acceptability of the aggregates is based on the User Tests listed in Table 5.9-1.

The initial tests in Table 5.9-1 are performed by the supplier. The user tests are performed by an independent laboratory for every 5000 tons delivered to the jobsite.

The following aggregate testing are performed at least once per shift, when concrete is being placed, and more frequently when required at the direction of the Contractor.

Samples are taken from weighing hoppers or the belt as directed by Contractor in the batch plant:

a. Two sand samples for gradation.

b. One coarse aggregate sample from each nominal size group for gradation.

a. Test for flat and elongated particles as directed by Contractor.

b. One sand organic test.

5.9.3.1.2 Cement

Cement is Type II, low-alkali cement as specified in “Standard Specification for Portland Cement,” ASTM C-150, and is tested to comply with the requirements of ASTM C-114. The inspection and testing of cement, in addition to the test performed by the cement manufacturers, is performed in accordance with Table 5.9-2.

The purpose of these tests is to ascertain conformance with ASTM C-150.

The initial tests are performed by the supplier. The user tests are performed by an independent laboratory for every 5000 cubic yards of concrete produced. During construction, the periodic tests are made to check storage environmental effects on cement characteristics. These tests are in addition to visual inspection of material storage procedures.

5.9.3.1.3 Fly Ash

Fly ash conforms to ASTM C-618 Class F, “Fly Ash and Raw or Calcined Natural Pozzolans for Use in Portland Cement Concrete,” and is tested to comply with the requirements of ASTM C-311, “Sampling and Testing Fly Ash for Use as an Admixture in Portland Cement Concrete.” All tests performed on the fly ash are listed in Table 5.9-3.

The user tests in Table 5.9-3 are performed by an independent laboratory for every 100 tons delivered to the job site. During construction, tests are made to check storage environmental effects on properties of fly ash. These tests are in addition to visual inspection of material storage procedures.

A typical chemical analysis from each source of fly ash is presented in Table 5.9-4. Fly ash from NUSCO Devon Plant is used in the concrete work, while the fly ash from the RG&E Company Russell plant is a standby and has not been used to date. The analyses on both fly ashes indicate that sulfates and chlorines are not present and that sulfur trioxide (SO_3) content is well within the limit set by the ASTM C-618.

5.9.3.1.4 Water and Ice

Water and ice used in mixing concrete are free from injurious amount of acid, alkali, organic matter, and other deleterious substances as determined by AASHO-T-26. Water does not contain impurities in amounts that will cause either a change in the time of setting of Portland cement of more than 25 percent or a reduction in the compressive strength of mortar of more than 10 percent compared with results obtained with distilled water. In addition, mixing water (including ice for cooling) complies with the following criteria:

(*Criteria*)	Percent
Alkalinity in terms of calcium carbonate	0.025 maximum
Total organic solids	0.025 maximum
Total inorganic solids	0.05 maximum
Total chlorides	0.025 maximum

These tests are performed quarterly.

5.9.3.1.5 Admixtures

The selected water-reducing agent MBHC, manufactured by the Masters Builders Company, possesses a shrinkage reduction effect similar to the type prescribed by ASTM C-494, “Specifications for Chemical Admixtures for Concrete.”

An air entraining agent, Vinsol Resin, manufactured by the Masters Builders Company, is added to the concrete mix to increase workability.

Admixtures containing chlorides are not used.

5.9.3.1.6 Concrete Mix Design

Concrete mixes are designed in accordance with ACI-211-1, “Recommended Practice for Selecting Proportions for Concrete,” using materials qualified and accepted for this work. Only concrete mixes meeting the design requirements specified for the structures are used.

Mixes are tested in accordance with the applicable ASTM Specifications as indicated:

ASTM	Test
C-39	Compressive strength tests
C-143	Slump
C-192	Making and curing cylinder in laboratory
C-231	Air content
C-232	Bleeding

For the containment, concrete test cylinders are cast from the basic mix designed for the structure. The following properties were determined by Professor David Purtz at the University of California, Berkeley.

a. Compressive strength (ASTM C-39)

b. Thermal diffusivity (ASTM C-342 and CRD C-36)

c. Autogeneous shrinkage (ASTM C-342)

d. Thermal coefficient of expansion (ASTM C-342 and CRD C-124)

e. Modulus of elasticity and Poisson's ratio (ASTM C-469)

f. Uniaxial creep (ASTM C-512)

g. Tensile strength (ASTM C-496)

Concrete design compressive strength for the elements of the structures are defined in the respective sections under Construction Materials.

5.9.3.1.7 Concrete Production and Testing

The concrete batch plant is located on the site and operates in a fully automatic mode. The rated capacity of the plant is 136 cubic yards per hour. A full time inspector from an independent testing laboratory is assigned to the plant to continually monitor the concrete batching operation.

Concrete samples are taken from the mix as prescribed in ASTM C-172, “Sampling Fresh Concrete.” Cylinders for compression tests are prepared from these samples which are cured in accordance with ASTM C-31, “Making and Curing Concrete Compressive and Flexural Strength Test Specimens in the Field.” Slump, air content, temperature, and unit weight are determined and recorded when the compression cylinders are cast.

Slump tests are performed in accordance with ASTM C-143, “Test for Slump of Portland Cement Concrete.” In addition to the performance of a slump test when compressive cylinders are cast, slump is measured at the batch plant for every 50 cubic yards of concrete mixed for delivery.

Air content tests are performed in accordance with ASTM C-231, “Test for Air Content of Freshly Mixed Concrete by the Pressure Method.”

Compressive strength tests are performed in accordance with ASTM C-39, “Test for Compressive Strength of Molded concrete Cylinders.” Evaluating of compressive strength tests is done in accordance with ACI 214, with the standard of control that which is required for “excellent” concrete.

Six cylinders, three sets of two each, are prepared for each placement of concrete as shown in the following tabulation:

Placing	Class I (cubic yards)	Class II (cubic yards)
Conventional, Plant	100	200
Conventional, Field	300	300
Pumping, Plant	100	200
Pumping, Field	100	300

Two cylinders are tested for compressive strength at each time internal of 7, 28, and 90 days, except that when correlation test data have been established for each design mix, test cylinders for the 90 day interval are disregarded with the exception of prestressed concrete.

To provide for accurate testing and concrete production, the equipment is calibrated using the following schedule:

Equipment Calibration Schedule

Testing Agency

Items	Calibration Interval
Platform Scales	6 months
Laboratory Scales	3 months
Meters, Air	3 months

Testing Agency

Items	Calibration Interval
Cylinder Compression Machine	12 months
Thermometers	6 months
Slump Cone	Examine for wear and replace as necessary

Concrete Supplier

Items	Calibration Interval
Pen Recorders	Daily
Batching System, Low-Limit and High-Limit Setpoints for Cement Aggregate and Flyash	1 month

5.9.3.2 Reinforcing Steel

5.9.3.2.1 Reinforcing Steel Materials

All reinforcing steel, except column ties and beam stirrups for some areas of the structures, is deformed billet steel bars conforming to ASTM A-615, Grade 60. Spiral reinforcing steel conforms to ASTM A-82.

Mill test reports are obtained from the reinforcing steel supplier for each heat of steel to ensure that the physical and chemical properties of the steel are in compliance with the applicable ASTM specifications. User tests to determine the strength and ductility of the reinforcing steel are used to supplement the standard mill tests. These are witnessed by an independent testing company. Procedures for obtaining samples for the reinforcing steel user test are defined in Section 5.9.3.2.2.

Procedures for splicing reinforcing bars using the Cadweld process is defined in Section 5.9.3.2.3.

5.9.3.2.2 Reinforcing Steel User Test Sampling

5.9.3.2.2.1 Procedures

The following procedure is used for sampling and testing reinforcing bars at the supplier's plant at Steelton, Pennsylvania. All tests are performed in accordance with ASTM A-615 except as otherwise noted herein. Only full size specimens are utilized in the user test sampling.

a. Upon receipt of a notice from the supplier that the stock reinforcing bars are available for sampling and testing, an inspector from an independent testing laboratory is sent to the plant to select random specimens and witness testing.

b. The inspector obtains the information regarding the heat numbers and bar sizes which require sampling and testing, and verifies the information by the rolling marks on the bars, the sources, grades and sizes of bars being supplied.

c. Specimens for tensile strength and cold bent tests are taken from the bars which have been selected at random by the inspector, in accordance with Sections 11.1 and 11.2 of ASTM A-615. The test specimens are then properly tagged and delivered to the supplier's metallurgical laboratory.

d. After test specimens are taken, the stockpiled materials are marked by tags and held until the results of the user tests are available.

e. When the results of the test specimens verify that the requirements of the specification have been met, the stockpiled materials are released for fabrication.

f. If during the tests, a specimen develops flaws, it may be discarded and another substituted.

g. If the test results of a specimen indicate that the tensile strength does not meet the minimum requirements, a second full size specimen from the same heat is taken and tested. If the second set of test results meets the minimum strength requirements, the tensile strengths from these two tests and the mill test are combined and averaged. If the average result meets the minimum strength requirements, the heat is accepted.

   If the averaged results of all three tests do not meet the minimum strength requirements, the heat is rejected.

h. Prior to shipping the fabricated materials to the site, the inspector returns to the plant for verification of the materials. Certified copies of the mill test reports, showing the physical properties and the ladle chemical analysis, are reviewed for compliance with the applicable ASTM Specifications.

The mill test reports, which accompany the materials to the site, and the shipping notices are initialed, dated and stamped by the inspector.

5.9.3.2.2.2 Deviation from Safety Guide 15

The procedure deviates from the Safety Guide 15 in that it allows a second tensile test to be conducted in the event that the first tensile test does not meet the minimum requirements. However, no tensile test failure was encountered during the user test and, consequently, the second tensile test was never required.

5.9.3.2.3 Splicing Reinforcing Bars

Reinforcing bars Number 11 and smaller are generally lap-spliced in accordance with ACI 318-63 except in congested areas where some bars are Cadweld-spliced. Reinforcing bar greater than Number 11 are Cadweld-spliced exclusively.

Reinforcing bars are not spliced by welding.

5.9.3.2.3.1 Scope

These procedures cover the mechanical splicing of deformed reinforcing bars using the Cadweld process, for full tension loadings.

All splices are made in accordance with the manufacturer's instructions as described in Erico Products Bulletin RBIS-10M269 “Cadweld Rebar Splicing,” except as modified hereinafter.

A manufacturer's representative, experienced in Cadweld splicing of reinforcing bars, is present at the site at the outset of the work to demonstrate the equipment and techniques used for making quality splices. He is also present for the first 50 production splices to observe and verify that the equipment is being used correctly and that quality splices are being obtained.

5.9.3.2.3.2 Materials

Cadweld T-Series materials are used for full tension, reinforcing-to-reinforcing splices. These are used for full tension, reinforcing to structural plates or shapes, where indicated on design drawings. C-Series splice materials are not used.

5.9.3.2.3.3 Qualifications of Operators

Prior to production splicing reinforcing bars, each operator or crew, including the foreman or supervisor for that crew, prepares and tests a splice for each of the positions used in the production work. These splices are made and tested in strict accordance with these procedures, using ASTM A-615, Grade 60 and the largest size bar spliced during production work. To qualify, the completed splices shall meet the acceptance standards of Section 5.9.3.2.3.7 for workmanship, visual quality and minimum tensile strength. A list of qualified operators and their qualification test results are maintained at the jobsite.

5.9.3.2.3.4 Care and Handling of Splice Kits and Equipment

a. The splice sleeves, cartridges, asbestos wicking, ceramic inserts and graphite parts are stored in a clean, dry, temperature controlled area with adequate protection from the elements to prevent absorption of moisture.

b. Splice sleeves are wrapped in a special rust inhibiting paper and are not unwrapped until ready to be used in the joining procedure.

c. Each splice sleeve is visually examined immediately prior to its being used to ensure that there is no rust or other foreign material on the inside surface.

d. The graphite pouring basins and crucibles are preheated with an oxyacetylene or propane torch to burn off moisture at the beginning of each shift when the molds are cold, when a new mold is put to use, or when the mold temperature is below ambient.

e. All graphite parts (except crucible covers) are cleaned with a whisk broom, rag, coarse brush or rolled up newspaper before reusing. A wire brush is not used on graphite parts.

5.9.3.2.3.5 Reinforcing Bar End Preparation

a. The bar ends to be spliced must be in good condition with full size, undamaged deformations.

b. The reinforcing bar deformations, except longitudinal ribs, which become engaged in the Cadweld splice are not ground, flame cut or altered in any way. Oversize longitudinal ribs are ground down to match the diameter of the bar deformations, but are not ground to a diameter less than the bar deformations.

c. For a minimum distance of two inches beyond where the ends of the sleeve are located, the bar ends are heated with an oxyacetylene “rosebud” torch to 300°F minimum to remove all moisture and burn off any organic foreign material.

d. After the heating operation, the bar ends are thoroughly cleaned by power brushing or sandblasting to remove all loose mill scale, concrete, dirt and other foreign material not removed by burning.

e. A reference line is painted 12 inches from the end of each bar that is to be spliced to confirm that the bar ends are properly centered in the sleeve.

f. Immediately before the splice sleeve is placed in final position, the previously cleaned bar ends are again surface preheated to 200° to 300°F minimum with an oxyacetylene rosebud torch to ensure complete removal of moisture.

g. Special attention is given to maintaining the alignment of sleeve and guide tube to ensure a proper fill.

h. When the temperature is below freezing or the relative humidity is above 65 percent, the splice sleeves are externally preheated with an oxyacetylene or propane torch after all materials and equipment are in position. The Cadweld operation is suspended during any form of precipitation.

i. A hairpin piece of soft twisted wire may be inserted at the top of the horizontal splices between the bar and the sleeve. This provides an escape route for the gases generated during the casting of the filler material.

j. The packing material at the ends of the horizontal splices and at the top of the vertical splices are not hard packed. Although the material is held firmly in place, it has to be loose enough to allow the escape of gases.

5.9.3.2.3.6 Splice Tensile Testing

Selected splices are tensile tested for each position, bar size and grade of bars. The test splices consist of sister splices, each three feet long, spliced in sequence in position and adjacent to the production works. Production splices, cut from in-place reinforcement material, are included. The following schedule is used:

a. Test splices, reinforcing-to-reinforcing:

    1. Production splice from the first 10 production splices.

    2. One production and three sister splices from the next 90 production splices.

    3. One production and two sister splices from the next and subsequent units of 100 splices.

b. The splices to be used in reinforcing structural steel shapes and plates consist of four sister splices from each unit of 100 production splices or fraction thereof.

5.9.3.2.3.7 Splice Acceptance Standards

a. All completed splices are visually inspected at both ends of the splice sleeves and at the tap hold in the center of the splice sleeves.

b. Sound, nonporous filler metal should be visible at both ends of the splice sleeve and at the tap hole in the center of the splice sleeve. Filler metal is usually recessed one-quarter inch from the ends of the sleeve due to the packing material, and is not considered a poor fill.

c. Splices that contain slag or porous metal in the riser, tap hole or at the ends of the sleeves (general porosity) are rejected. A single shrinkage bubble, present below the riser, is not considered demineralized and should be distinguishable from the general porosity described in Item b.

d. There must be evidence of filler material between the sleeve and the bar for the full 360 degrees; however, the splice sleeves need not be exactly concentric or axially aligned with the bars.

e. Both horizontal and vertical Cadweld splices may contain voids at either or both ends of the Cadweld splice sleeve. The allowable limits for end voids are as shown on Table 5.9-5. The area of the voids is assumed to be the maximum depth of the wire probe minus 3/16 inch multiplied by the width at the inside surface of the sleeve.

f. The average tensile strength of the Cadweld spliced joints are equal to or greater than the minimum tensile strength for the particular grade of reinforcing steel as specified in ASTM A-615. The minimum acceptable tensile strength of any splice is 125 percent of the specified minimum yield strength for the particular bar size.

5.9.3.2.3.8 Deviation from the Safety Guide 10

This procedure deviates from the Safety Guide 10 in that only one qualification splice is required for each crew member, including the foreman. The Safety Guide recommends that two qualification splices be required.

On the first 245 sister and production splices tested, only one failed below the specified minimum strength of 125 percent of the yield strength of the reinforcing bar. This failure was attributed to a niche in the bar which was used for identification purposes, and was not caused by the failure of the splice itself.

The exceptional performance of the test splices more than justifies the omission of a second qualification splice.

5.9.3.3 Post-Tensioning System

5.9.3.3.1 Tendons

The tendons are composed of stabilized, low relaxation wires of one-quarter inch diameter with a tensile strength of 240,000 psi in accordance with ASTM A-421. The pertinent features of the tendons are as follows:

Number of wires	186
Ultimate tensile capacity (kips)	2190
Design tensile capacity (kips)	1315

End anchorages	Buttonheads

Sampling and testing of the tendon material conform to ASTM A-421. The following procedure is used.

a. One buttonhead test on each end of each reel of wire to establish the suitability and acceptance of the wire for buttonheading.

b. One buttonhead rupture test from each reel of wire.

c. Random samples test of each lot of wire in accordance with ASTM-421. With each sample of wire tested, a certificate was submitted stating the manufacturer's minimum guaranteed ultimate tensile strength of the wire sampled. Stress-strain curves were plotted for each of these tests and the yield and tensile strength of the wire was verified.

5.9.3.3.2 Anchorages

The basic performance requirements for the end anchors of the tendons are stated qualitatively by the Seismic Committee of the Prestressed Concrete Institute and published in their Journal of June 1966 as follows:

> "All anchors of unbonded tendons should develop at least 100 percent of the guaranteed ultimate strength of the tendon. The anchorage gripping should function in such a way that no harmful notching would occur on the tendon. Any such anchorage system used in earthquake areas must be capable of maintaining the prestressing force under sustained and fluctuating loads and the effect of shock. Anchors should also possess adequate reserve strength to withstand any overstress to which they may be subjected during the most severe probable earthquake. Particular care should be directed to accurate positioning and alignment of end anchors."

The end anchors used are capable of developing 100 percent of the minimum tensile strength of the prestressing steel as defined by ASTM A-421.

The end anchors are capable of maintaining integrity for 500 cycles of loads corresponding to an average axial stress variation between 0.7 and 0.75 f'_s at a repetition rate of one cycle in 0.1 second. This requirement sets minimum acceptable limits on fatigue effects due to notching by the end anchor and tendon performance in response to earthquake loads.

The number of cycles was set by increasing to 500 from the 100 predicted. The stress variation was increased from a conservatively predicted 0.6 to 0.64 f'_s to the 0.7 to 0.75 f'_s. Further, the number of cycles caused by the earthquake loads was predicted as only 30 of the total of 100 resulting from using all the strong ground motions which exceed one/half of the peak ground motion of the earthquake.

The stress variations due to the earthquake motion alone were predicted as being 10 percent of the total of the predicted stress variations of 0.04 f'_s. The predicted 0.04 f'_s stress variation, in turn, resulted from the combinations of earthquake, wind, and incident loadings. Analyses made during the investigation included consideration of tendon excitation, both parallel and perpendicular to the tendon axis.

The anchorage assemblies, including the bearing plates, are capable of transmitting the ultimate loads of the tendons into the structure without brittle fracture at an anticipated lowest service temperature of -30°F.

The anchorage assemblies used are capable to prelude brittle fracture at a design temperature of -50°F.

5.9.3.3.3 Sheathing

Sheaths for the tendons are classified as concrete forms and are not subjected to any standard codes. They provide a void in the concrete wherein the tendons were installed, stressed and greased after the concrete was placed.

The sheaths are made from 22 gauge, galvanized ferrous metal, and have an internal diameter of five inches clear of corrugations. Couplers are provided at all field splices and sealed by tape.

After sheathing installation, and prior to concrete placement, the sheathing is surveyed to assure accurate alignment. An inspection is also performed to ascertain that all sheaths are continuous and unblocked by obstructions.

Before installation of the tendons, the sheathing is carefully cleaned to remove all water and debris.

Vent tubing and temporary valves were provided to permit drainage at all low points.

Splash caps at the ends of all sheaths, to prevent concrete and laitance from entering into the sheaths during construction, were provided.

5.9.3.3.4 Corrosion Protection

Suitable atmospheric corrosion protection was maintained for the tendons from the point of manufacture to the installed locations. The atmospheric corrosion protection provided assurance that the tendon integrity was not impaired due to exposure to the environment.

Prior to shipment, they were all coated with a thin film of petroleum that contained rust inhibitors. After the tendons were installed in the sheaths and stressed, the interior of the sheathing was pumped full of a modified, thixotropic, refined petroleum oil-based product to provide corrosion protection. The tendons and end anchors were also surrounded with the corrosion-protection material which was encapsulated in the sheathing and gasketed end caps that were sealed against the bearing plates. When the sheaths and end caps are filled, the corrosion-protection material displaces air and water vapor before thickening to a soft gel. Once the filler material had cooled, contracted, and gelled, the vertical tendons were topped-off by injecting additional filler material through the upper grease cap fittings.

As a result of the Millstone Unit Number 2 tendon surveillance program, sixteen horizontal tendons have been identified as subject to ground water intrusion. To prevent ground water intrusion, the corrosion protection material is continuously supplied to the subject tendons at a pressure slightly above hydrostatic pressure of the ground water. The tendons so pressurized are horizontal tendons 12H01 through 12H06, 12H08 through 12H10, 31H01 through 31H04, 31H01, 32H02, and 32H03.

Testing of the permanent corrosion-protection material indicates that there were no significant amounts of chlorides, sulfides, or nitrates present. However, to further verify the chemical composition of the filler material, test samples are taken from each shipment with at least one sample per factory batch. The samples were analyzed as follows:

a. Water-soluble chlorides (C1) are determined in accordance with ASTM D512-67 with a limit of accuracy of 0.5 ppm.

b. Water-soluble nitrates (NO_3) are determined by the Water and Sewage Analysis Procedure of the Hach Chemical Company, Ames, Iowa.

c. Water-soluble sulfides (S) are determined in accordance with American Public Health Association Standards (APHA) with a limit of accuracy of 1 ppm. The APHA Standards methods may be modified to use standard reagents and procedures such as those available from Hach Chemical Company.

No significant traces of the impurities are allowed. The chemical composition of the filler material, being about 98 percent petroleum jelly, indicates that it possesses the normal stability of linear hydrocarbons for the site temperature ranges.

NOTE: For description of water intrusion into the tendon gallery during construction and methods of repair, see Appendix 5.F.

5.9.3.4 Structural and Miscellaneous Steels

All structural and miscellaneous steels conform to the following ASTM specifications:

Rolled shapes, plates, tubing and bars	A-36
Crane rails	A-1
High strength bolts	A-325 or A-490
Anchor bolts (nonwelding)	A-575, Grade 1020
Stainless steel	A-240, Type 304

Mill test reports are obtained for all materials used with the exceptions of hand rails, toe plates, kick plates, stairs and ladders.

Detailing, fabrication, and erection of the structural and miscellaneous steels are in accordance with AISC Manual of Steel Construction.

Welding is accomplished in accordance with AWS D1.0, "Code for Welding in Building Construction," and where applicable, AWS D2.0, "Specifications for Welded Highway and Railroad Bridges."

Quality control procedures for field welding are defined in Section 5.9.4.

5.9.3.5 Steel Liner Plate and Penetration Sleeves

5.9.3.5.1 General

The containment is lined with a one-quarter inch welded steel plate to ensure leak tightness. The design, construction, inspection, and testing of the liner plate are not covered by an recognized code or specification, since it is not a pressure vessel and serves only as a leak-tight membrane. However, components of the liner which must resist the full containment design pressure, such as the penetration sleeves, are designed, fabricated, constructed, and tested to meet the requirements of Paragraph N-1211 of Section III, Nuclear Vessels, 1968 Edition through the summer 1969 addenda of the ASME Code, except where otherwise noted herein.

The liner is designed to function only as a leak-tight membrane. It is not designed to serve as a structural element to resist the tensile loads from an internally applied pressure such as might result from a loss-of-coolant incident. Structural integrity of the containment is maintained by the pre-stressed, post-tensioned concrete. Since the principal stresses of the liner due to thermal expansion are in compression, and no significant tensile stresses are expected from the internal pressure loading, special nil ductility transition temperature requirements are not applied to the liner plate materials. However, all materials for the liner components which must resist tensile stresses resulting from internally applied pressure, such as the penetration sleeves are impact tested in accordance with the requirements of Paragraph N-1211 of Section III, Nuclear Vessels, 1968 Edition through the summer 1969 addenda of the ASME Code.

Materials used in the construction of the steel liner plate and penetration sleeves are defined in Section 5.2.3.1.

5.9.3.5.2 Fabrication and Erection

A basic requirement for the fabrication and erection of the steel liner plate is that all welding procedures and welding operators be qualified by tests as specified in Section IX of the 1968 ASME Code.

Penetration sleeves are shop fabricated in accordance with the requirements of Paragraph N-1211 of Section III, Nuclear Vessels, 1968 Edition through the summer 1969 addenda of the ASME

Code, including welding and NDT procedures and third party inspection, but excluding pressure testing. A modified code data sheet is prepared for each sleeve. The sleeves are not code stamped because they are not included as part of the vessel under the ASME Code.

Quality control procedures for field welding and nondestructive examination are defined in Section 5.9.4.

5.9.3.5.3 Inspection and Testing

The following inspection and testing are performed on the steel liner plate.

5.9.3.5.3.1 Radiography

For quality control purposes, completed liner plate weld seams are spot radiographed by the subcontractor in accordance with the following schedule.

a. One 12 inch film is taken during the first 50 feet of each welder's work, in each welding position.

b. Thereafter, a minimum of 10 percent of the welding is progressively spot examined as welding is performed, using a 12 inch film. Locations for spot radiograph are designated by the engineer in such a manner that an approximately equal number of spot radiographs is taken from each welder's work.

c. If a radiograph discloses welding which does not meet the acceptance criteria, two additional spot radiographs, each 12 inches in length are taken in the same weld seam, at locations away from the original spot designated by the engineer.

   If the two additional radiographs show welding which meets the acceptance criteria, the entire weld seam represented by the three radiographs is considered acceptable except at the area of defect. This area is removed and repaired.

d. If either of the two additional radiographs show welding which does not meet the acceptance criteria, the entire portion of the weld seam represented by these radiographs is rejected and the defective welding removed and repaired.

e. The repaired weld seams are completely reradiographed to ensure compliance with the acceptance criteria.

The techniques used for radiographic examinations are in accord with Paragraph UW-51 of Section VIII of the 1968 ASME Boiler and Pressure Vessel Code, using X-ray and fine grain films. The double film, single viewing technique is used for all radiography.

The acceptance criteria for examinations are in accordance with Paragraph UW-52, Section VIII, of the 1968 ASME Boiler and Pressure Vessel Code. All radiographic films are submitted by the subcontractor to the engineer for review, interpretation and record.

The Safety Guide 19 was not conformed with where weldments could be radiographed. In these instances, 10 percent of the weldments are inspected by radiography. This exceeds the Safety Guide suggested minimum of two percent. The first 50 feet of welding performed by each welder is inspected instead of the first 10 feet, as suggested by the Safety Guide.

When radiographic inspection is not feasible, magnetic particle inspection is substituted.

Leak chase channel testing is performed in accordance with the suggested Safety Guide requirements with the exception that the system is pressurized for a minimum of 25 minutes and a maximum of 30 minutes, instead of two hours as stated therein. This testing may be repeated at any time during the plant life.

The limited deviations from the Safety Guide do not in any way affect the functional integrity of the steel liner plate.

5.9.3.5.3.2 Visual Examination and Dye Penetrant Testing

All weld seams are 100 percent visually examined in accordance with Section 5.9.4.5.3. Weldments which on the basis of visual examination are judged to be of questionable quality by either the subcontractor or engineer, are also inspected by dye penetrant testing.

All dye penetrant inspection is in accordance with Section VIII, of the 1968 ASME Boiler and Pressure Vessel Code.

5.9.3.5.3.3 Magnetic Particle Testing

Where nonradiographable welds are used, magnetic particle testing is substituted for radiography. A minimum of 10 percent of such welding, including splices with welded backing strips, is examined as the welding is performed.

All magnetic particle testing is performed in accordance with Appendix VI of Section VIII of the 1968 ASME Code, Dry Particle, Direct Current Production Method.

5.9.3.5.3.4 Vacuum Box Testing

All liner welds which must maintain leak-tightness integrity, including plates, shell plates, and dome plates are tested by the subcontractor as the work proceeds, using a vacuum box that can be placed over the test area and evacuated.

A 5 psi minimum pressure differential with respect to the atmospheric pressure is maintained for a minimum of 20 seconds, and verified by a gauge. The soap suds solution is continuously observed for bubbles which indicate leaks, from the time evacuation of the box is started until 20 seconds after the required vacuum has been obtained.

All leaks, regardless of size, are repaired by completely removing the defects and rewelding. The area needing repair and a minimum of two inches either side is reinspected by vacuum box testing.

Welds which cannot be vacuum box tested due to configuration and space limitations are dye penetrant tested in accordance with Section VIII, of the 1968 ASME Boiler and Pressure Vessel Code.

5.9.3.5.3.5 Halogen Testing

The leak chase system over the floor liner plate is Freon tested. A standard high sensitivity industrial halogen leak detector capable of detecting leakage in the order of 1 x 10^{-9} scc/second was used.

The leak chase system is initially charged with a tracer gas until a pressure of 15 psig is attained. It is then pressurized with air until a test pressure of 60 psig is reached.

The pressurized leak chase system is allowed to stand for 25 minutes minimum, before starting the probe test. Pressure measurements are taken at the beginning and the end of the holding period. Any pressure decay greater than 2 psig on a 0 to 100 psig, 4.5 inch test gauge, is rejected.

Prior to each test, the leak detector is calibrated in accordance with the manufacturer's instruction against a standard leak of 1.0 x 10^{-5} scc/second With the control unit of the leak detector set on automatic balance, the tip of the probe is placed on the weld seam to be tested and scanned at the rate of 1 ips. All leaks larger than 1.0 x 10^{-5} scc/second are located, removed, repaired and retested.

5.9.3.5.4 Quality Control of Field Welding Electrodes

The quality control procedure for the electrodes used in the field welding of the containment steel liner plate is as follows:

a. Approved procedures for handling and storing of welding electrodes are used.

b. The subcontractor’s welding and quality control supervisors control the welding electrodes by:

    1. Performing receipt inspection and verifying the electrode identification with the certificate of compliance.

    2. Physically separating the various types of electrodes and ensuring that the approved storage requirements are met.

    3. Instructing the welders on type(s) of electrodes and welding procedures to be used.

4. Monitoring welders at their work stations to ensure proper welding electrodes and welding procedures are being used.

5. Surveying the work areas near the end of each work day to ensure electrodes are either discarded or returned to the proper storage facilities.

c. Bechtel quality control inspectors monitor the liner erection and welding on a continuous basis. Weekly inspection reports are prepared.

5.9.3.6 Interior Coatings (Original Construction)

All of the coating materials given have been tested by their manufactures under simulated operating and incident conditions and certified to fully comply with all the requirements of the ANSI Standard N.101.2 (1972) "Protective Coatings (Paints) for Light Water Nuclear Reactor Containment Facilities."

In addition, all shipments of these materials are accompanied by vendor certifications of compliance.

5.9.3.6.1 Containment Steel Liner Plate Coatings

Surface preparation of the interior (exposed) surfaces of the containment steel liner plate is accomplished in the shop by blast cleaning each plate from edge to edge in accordance with the Steel Structures Painting Council (SSPC) Specification SSPC-SP-6-63, "Commercial Blast Cleaning." The plates are then primed with one coat of an inorganic zinc primer, Carbo-Zinc II, to within two inches of the edges of the plate. The minimum dry film thickness (dft) of the primer is 3 mils.

In applying the primer to the plates, the coating manufacturer's written instructions and the SSPC-SP1-63, "Solvent Cleaning" are followed explicitly.

After the liner is erected, the field weld seams and the limited burnback of the Carbo-Weld 11 coating are power tool cleaned and recoated with Carbo-Zinc 11 of 3 mils dft.

Areas which are damaged by welding, such as arc strikes, or due to the removal of temporary attachments for erection, are required and recoated so that they are equivalent to the original conditions.

Finish coats for the steel liner plate are as follows:

a. Wainscot

Two coats of a modified, organic phenolics, Phenoline Number 305 finish, at 3 mils dft each coat.

b. Above wainscot, including:

One coat of Phenoline Number 305 at 3.0 mils minimum dft.

5.9.3.6.2 Containment Interior Coatings

5.9.3.6.2.1 Steel Surfaces Wainscot

Carbon steel surfaces, including structural and miscellaneous steels, uninsulated piping, and equipment which are located in areas subject to hard usage or radioactive contamination, are blast cleaned in accordance with Steel Structures Painting Council Specification SSPC-SP6-63, “Commercial Blast Cleaning.”

Within eight hours after blast cleaning, the surfaces are primed with one coat of an inorganic zinc primer, Carbo-Zinc 11, at 3 mils dft. This is followed by two coats of a modified phenolic, Phenoline Number 305 Finish, at 3 mils dft.

Insulated piping is blast cleaned and primed the same as above, but receives no finish coating.

5.9.3.6.2.2 Steel Surfaces Above Wainscot

Carbon steel surfaces above wainscot height are blast cleaned and primed the same as in Section 5.9.3.6.2.1 except that they receive one organic finish coat of Phenoline Number 305 at 3.0 mils minimum dft.

5.9.3.6.2.3 Galvanized and Stainless Steel Surfaces are not Painted.

5.9.3.6.2.4 Concrete and Masonry Surfaces

All concrete and masonry surfaces, including floors, wainscot, walls, columns, pilasters, and ceilings are chemically cleaned by either caustic wash or acid etching, or by blasting. An organic surfacer, Keeler and Long Number 6548 Epoxy Block Filler is then applied over the surfaces at a thickness of 5.5 mils dft, and one coat of organic Keeler and Long Epoxy Enamel at 2.5 mils dft.

5.9.3.7 Interior Maintenance Coatings (first implemented during Mid cycle 13, 1997)

All maintenance coating materials applied to surfaces inside or to be installed in the reactor containment have been tested to withstand Millstone Unit 2 design basis loss of coolant accident (DBA-LOCA) conditions. The coating materials and their application comply with the intent of Regulatory Guide 1.54 within the following clarification and exception.

<u>Clarification</u>

Compliance with Regulatory Guide 1.54 will not be invoked for equipment of a miscellaneous nature and all insulated surfaces. It is impracticable to impose Regulatory Guide requirements on the standard shop process used in painting valve bodies, handwheels, electrical cabinetry, control panels, loud speakers, emergency light cases, and other miscellaneous equipment. Wrapped or rigid insulation captures and retains any coating which may come off equipment surfaces, thereby preventing the coating material from reaching and blocking sump drains or interrupting water flow in the containment spray system.

Exception

Quality Assurance Program recommendations stated in Regulatory Guide 1.54 are followed except that inspection will be in accordance with Section 10 of ANSI N5.12-1974 in lieu of Section 7 of ANSI N5.9 as referenced in Section 6.2.4 of ANSI N101.4.

Each coating was tested in accordance with ASTM D3911, "Evaluating Coatings Used in Light-Water Nuclear Power Plants at Simulated Design Basis Accident (DBA) Conditions," to the DBA conditions represented by the pressure (70 psig) and temperature (340°F) curve of Figure 1, therein.

Prior to exposure to DBA conditions, each coating was irradiated to an accumulated dose of at least $1x10^9$ Rads in accordance with ASTM D4082, "Effect of Gamma Radiation on Coatings for Use in Light-Water Nuclear Power Plants."

The coating manufacturers provide certification with each shipment that the supplied coating materials are identical to the batches of coating materials satisfactorily tested to DBA conditions.

Maintenance coating materials applied to containment surfaces are used to repair and maintain the existing coatings, to coat existing surfaces that were intended to be, but were not previously coated, and to coat new surfaces to be installed into the containment. Application of the new coating materials requires complete prior removal of the existing coating from the surface within the repair area. Overcoating the existing coatings with the new coating materials requires prior testing of the material combinations to radiation and simulated DBA conditions in accordance with Regulatory Guide 1.54.

5.9.3.7.1 Stainless Steel Surfaces

Stainless steel surfaces are not painted.

5.9.3.7.2 Galvanized Surfaces

Galvanized surfaces are spot repaired with a qualified organic coating material, as required, to maintain corrosion protection. Galvanized surfaces are not otherwise coated.

5.9.3.7.3 Carbon Steel Surfaces

Existing carbon steel surfaces are repair coated as required. New carbon steel surfaces are coated prior to or upon installation into the containment. Carbon steel surfaces may remain uncoated when substantiated by appropriate engineering evaluation.

5.9.4 QUALITY CONTROL PROCEDURES FOR FIELD WELDING AND NONDESTRUCTIVE EXAMINATIONS

5.9.4.1 Scope

These procedures outline the general quality control requirements for welding to ensure that all field welding is performed in full compliance with the applicable job specifications.

5.9.4.2 Qualifications for Welding Inspectors

All welding inspectors who inspect welds covered by this specification are qualified by meeting the following minimum requirements:

a. Inspectors must have a thorough knowledge of the various welding processes and techniques employed in field construction and be able to demonstrate the proper methods for shielded metal-arc welding, gas tungsten-arc welding, gas metal-arc welding, and oxyacetylene welding.

b. A minimum of two years previous welding inspection experience or equivalent experience and training in welding fabrication and nondestructive testing is required for all inspectors.

c. Inspectors are required to demonstrate to the satisfaction of the responsible Bechtel Material, Fabrication and Quality Control Services Representative, their knowledge of the fundamentals, techniques, and applications of the inspection methods set forth in this standard, i.e., visual, vacuum box, magnetic particle, dye penetrant and radiographic inspections.

5.9.4.3 Welding Performed by Bechtel Construction Personnel

5.9.4.3.1 Welding Procedures

All welding performed by Bechtel construction personnel is in strict accordance with the approved Bechtel Welding Procedure Specifications. The appropriate Bechtel Welding Procedure Specifications for field welds are prepared and qualified by the Bechtel Material, Fabrication and Quality Control Services Department and issued to the field by the Bechtel Project Engineer.

5.9.4.3.2 Welder Qualification

All welders who are welding under Bechtel Welding Procedure Specifications are qualified by performing the test required in the applicable Bechtel Welder Performance Specification WQ-F-1 for ferrous materials and WQ-NF-1 for nonferrous materials. These Bechtel specifications encompass the requirements of Section IX of the ASME Code. Number welder is permitted to perform production welding until he has passed the necessary tests and has the appropriate Welder Performance Qualification Test Record.

5.9.4.4 Welding Performed by Bechtel Subcontractors

5.9.4.4.1 Welding Procedures

All welding performed by Bechtel subcontractors are in strict accordance with the applicable job specifications. All welding procedures used on the project are submitted to Bechtel Engineering for approval. Production welding is not permitted without prior approval of these procedures. In all cases, field welding inspectors are responsible for determining that the subcontractor’s welding is being performed in accordance with properly qualified and engineering approved welding procedure specifications.

5.9.4.4.2 Welder Qualification

All welders are welding operators employed by subcontractors who are making welds under a code or standard which requires qualification of welders are tested and qualified accordingly before beginning production welding. Each subcontractor is responsible for testing and qualifying his own welders. The Bechtel field welding inspector is responsible in all cases for determining that the subcontractor’s welders have successfully passed the necessary qualification tests and that the subcontractor has the proper qualification test records for each qualified welder on file at the jobsite.

5.9.4.5 Instructions for Field Welding Inspectors

The general instructions for field welding inspectors which follow cover welding performed by both Bechtel construction and Bechtel subcontractors.

5.9.4.5.1 Welding Procedures

It is the responsibility of the field welding inspectors to assure that all welding is performed in strict accordance with the appropriate qualified welding procedure specifications. Specific items to be checked follows:

a. Determine that the proper welding procedure specification has been selected to match the base materials being welded and the welding processes being employed.

b. Permit only welders who are properly qualified under the essential variables of each welding procedure specification to make welds under that procedure.

c. Check to see that the welding electrodes, bare filler rod, consumable insert strips, and backing strips all match those which have been specified.

d. Inspect weld joints as necessary prior to welding to ensure proper edge penetration, cleaning, and fit up.

e. Check to see that the welding machine settings are correct and fall within the range of current and voltage specified.

f. Check for proper preheat and interpass temperature.

g. Inspect the inprocess welding for proper techniques, cleaning between passes, and appearance of individual weld beads.

5.9.4.5.2 Postweld Heat Treatment

The field welding inspectors inspect each postweld heat treatment (thermal stress relieving) operation to ensure conformance with the applicable job specifications. Specific items to be checked include the following:

a. A sufficient number and proper location of thermocouples are selected to accurately record temperatures.

b. The thermocouples are connected to temperature indicator recorders which provide a permanent record of the heating rate, holding temperature and time, and the cooling rate.

c. Temperature charts are checked for proper heating rate, holding temperature, holding time, cooling rate, and to see that the proper weld identification is recorded on the chart.

5.9.4.5.3 Visual Inspection of Weldments

The field welding inspectors are responsible for carrying out the necessary welding surveillance to ensure that all welding meets the following requirements for visual qualify and general workmanship. Visual inspections are performed during and after welding.

a. All weld beads, passes, and completed welds are free of slag, cracks, porosity, incomplete penetration and lack of fusion.

b. Cover passes are free of coarse ripples, irregular surfaces, nonuniform bead patterns, high crowns, deep ridges or valleys between beads, and that all blend smoothly and gradually into the surface of base metal.

c. Butt welds are slightly convex, of uniform height, and have full penetration.

d. Fillet welds are of specified size, with full throat and, unless otherwise specified, the legs are of approximately equal length.

e. Repairing, chipping, or grinding of welds is done in such a manner as not to gouge, groove, or reduce the base metal thicknesses.

f. Where different base metal thicknesses are joined by welding, the finished joint is tapered no steeper than one to four (1:4) between the thick and the thin sections.

5.9.4.5.4 Magnetic Particle Inspection

The field welding inspector is responsible for determining that all magnetic particle inspection is properly performed. He ensures that the proper techniques are followed and that the results properly interpreted. The field welding inspector requires that the subcontractor's responsible inspection personnel demonstrate their knowledge and understanding of the applicable specifications prior to performing any production testing.

Special attention is given to the following items for all magnetic-particle inspection:

a. Determine that surfaces to be inspected have been properly cleaned and are free of crevices which could produce false indications by trapping the iron powder.

b. Determine that the power source, current density, prod spacing and application of iron powder comply with the applicable requirements.

c. Permit no arcing between the prods and weld surfaces.

d. Interpret all linear or linearly disposed indications as defects.

e. Probe questionable indications by thermal cutting, chipping, grinding, or filing to confirm the presence or absence of actual defects.

5.9.4.5.5 Dye Penetrant Inspection

The field welding inspector is responsible for determining that all dye penetrant inspection is properly performed. He ensures that the proper technique is followed and that the results are properly interpreted. The field welding inspectors require the subcontractor's responsible inspection personnel to demonstrate their knowledge and understanding of the applicable specifications prior to performing any production testing.

Special attention is given to the following items for all dye penetrant inspection:

a. Determine that surfaces to be inspected have been properly cleaned and are free of crevices which can product false indications by trapping the dye penetrant.

b. Check to see that cleaner, dye penetrant, and developer are properly applied and the specified time intervals for dye penetration and developing are followed.

c. Determine that indications are properly interpreted. Defects will be identified as red stains against the white developer background. Red lines or linearly disposed red dots are indicative of cracks. Porosity and pinhole leaks appear as local red patches or dots.

d. Examine questionable indications by a 5X or stronger hand lens, and probe by grinding or filing to confirm the presence or absence of defects.

5.9.4.5.6 Radiographic Inspection

The field welding inspector is responsible for determining that all radiographic inspection is properly performed. He ensures that radiographic techniques are followed and that the completed films are properly interpreted. The field welding inspectors require the subcontractor's responsible inspection personnel to demonstrate their knowledge and understanding of the applicable specifications prior to beginning the radiographic inspection. The field welding inspector also reviews each completed radiograph.

Special attention is given to each of the following items for all radiographic inspection:

a. Check the type of film intensifying screens, penetrameters, and sources of radiation for conformance to the job specifications.

b. Check the relative location of film, penetrameters, identifying numbers, and radiation source for each typical exposure.

c. Review all completed film for quality and interpretation of defects. Check the exposed and developed film for proper density and visibility of penetrameters. Radiographic film of unacceptable quality or with questionable indications of defects are reradiographed.

5.9.4.5.7 Other Welding Inspections

The field welding inspectors are responsible for determining that all other types of welding inspection, where specified, are properly performed.

5.9.4.5.8 Repairs

It is the responsibility of the field welding inspectors to determine that all weld defects in excess of specified standards of acceptance are removed, repaired, and reinspected in accordance with the applicable job specifications.

5.9.4.5.9 Records

It is the responsibility of the field welding inspector to ensure that proper records of welding and nondestructive testing are kept on file at the jobsite.

TABLE 5.9-1 AGGREGATE TESTS

ASTM Number	Title	Results To Be Achieved	Initial Test	User's Test	Periodic Test
C-33	Specification for concrete aggregates	To conform with specification	X	X	X
C-40	Organic impurities in sands for concrete	To conform with specification	X	X	X
C-87	Effect or Organic Impurities in Fine Aggregate on Strength of Mortar	To conform with specification	X	X	
C-88	Soundness of Aggregates	To conform with specification	X	X	
C-117	Materials finer than Number 200 sieve	Design mix calculations	X	X	
C-127	Specific gravity and absorption (coarse aggregates)	Design mix calculations	X	X	
C-128	Specific gravity and absorption (fine aggregates)	To conform with specification	X	X	
C-131	Los Angeles Machine abrasion	To conform with specification	X	X	
C-136	Sieve analysis of fine and coarse aggregates	To conform with specification	X	X	
C-142	Clay lumps	To conform with specification	X	X	
C-227	Potential Alkali reactivity (mortar bar)	To conform with specification.	X	X	
C-289	Potential reactivity (chemical)	To conform with specification	X	X	
C-295	Petrographic examination of aggregates	To conform with specification	X		

TABLE 5.9-2 CEMENT TESTS

ASTM Number	TYPE OF TEST	INITIAL TEST	USER'S TEST	PERIODIC TESTS
C-109	Compressive Strength	X	X	X
C-114	Chemical Analysis	X	X	
C-115	Fineness-Turbidimeter	X	X	
C-151	Auto-clave expansion (soundness)	X	X	
C-183	Sampling	X		
C-185	Air content of mortar	X		
C-186	Heat of hydration	X		
C-191	Time of setting by Vicat needle	X	X	X
C-204	Fineness by air permeability	X		
C-266	Time of setting by Gillmore needles	X		
C-451	False set (paste)	X		

TABLE 5.9-3 FLY ASH TESTS

ASTM Number	TYPE OF TEST	INITIAL TEST	USER'S TEST	PERIODIC TESTS
C-109	Compressive strength	X	X	X
C-114	Chemical analysis	X	X	
C-151	Autoclave expansion (Soundness)	X	X	
C-188	Specific gravity	X	X	
C-204	Fineness	X	X	
C-311	Sampling and testing	X	X	

TABLE 5.9-4 TYPICAL CHEMICAL ANALYSIS OF FLY ASH USED

Chemical Analysis	NUSCO Devon Plant	RG&E Co. Russell Plant	ASTM C-618
Silicon dioxide (SiO_2) plus aluminum oxide ($A1_2O_3$) plus iron oxide (Fe_2O_3) (minimum %)	93.22	86.20	70.0
Magnesium oxide (MgO) (maximum %)	1.20	1.22	-
Sulfur trioxide (SO_3) (maximum %)	0.60	1.16	5.0
Moisture content (maximum %)	0.13	0.13	3.0
Loss on ignition (maximum %)	4.01	5.90	12.0
Available alkalies as Na_2O (maximum %)	0.77	0.73	1.5

TABLE 5.9-5 ALLOWABLE VOID LIMITS FOR CADWELDING

Bar Size Number	Splice Catalog Number	Maximum Allowable Void Limits	
		Void Area Standard Splices, (1) (2) (in^2)	Vertical - Full Circumference Low, (3) (inches)
6-6	RBT-6101-(-H)	1.05-1.05	5/8 - 5/8
6-7	RBT6-7101 (-H)	1.05-1.03	5/8 - 9/16
7-7	RBT-7101 (-H)	1.03-1.03	9/16 - 9/16
7-8	RBT7-8101 (-H)	1.03-1.02	9/16 - 1/2
8-8	RBT-8101 (-H)	1.02-1.02	1/2 - 1/2
8-9	RBT-8-9101 (-H)	1.02-1.02	1/2 - 1/2
9-9	RBT-9101 (-H)	1.02-1.02	1/2 - 1/2
9-10	RBT9-10101 (-H)	1.02-1.03	1/2 - 7/16
10-10	RBT-1091 (-H)	1.03-1.03	7/16 - 7/16
10-11	RBT10-11101 (-H)	1.03-1.53	7/16 - 9/16
11-11	RBT-11101 (-H)	1.53-1.53	9/16 - 9/16
11-14	RBT-11-14101 (-H)	1.53-1.52	9/16 - 5/8
11-18	RBT11-18101 (-H)	1.53-1.99	9/16 - 1/2
14-14	RBT-1476 (-H)	2.15-2.15	5/8 - 5/8
14-14	RBT-14101 (-H)	2.15-2.15	5/8 - 5/8
14-18	RBT14-18101 (-H)	2.15-1.99	5/8 - 1/2
18-18	RBT-1876 (-H)	2.64-2.64	9/16 - 9/16
18-18	RBT-1891 (-H)	3.00-3.00	5/8 - 5/8
18-18	RBT-18101 (-H)	3.00-3.00	5/8 - 5/8

Void Area = W (D-3/16) (Normal void due to asbestos packing)

NOTES:

(1) The maximum allowable void area computed separately for each end of the splice sleeve as shown in the sketch.

(2) This column is used for all standard splices including vertical, horizontal, horizontal side fill, angled splices and B-series structure splices.

(3) This column is used for vertical splices only with low filler metal around entire circumference. For spot voids in vertical splices, standard splices column is used.

5.A DESCRIPTION OF FINITE ELEMENT METHOD USED IN CONTAINMENT ANALYSIS

5.A.1 INTRODUCTION

The initial development of the finite element method was done by Turner, et al. (Reference 5.A-1) for future application in aerospace technology. Turner, et al. later used the two-dimensional plate elements in the analysis of aircraft structures. These first applications of the method were used to analyze the plane stress problem. (References 5.A-1 and 5.A-2) Continued development of the method has extended its applicability to the plane strain problem, flat plate bending, flat plate stability studies, three dimensional axisymmetric stress analysis and for general shell analysis. (References 5.A-3 through 5.A-6) Most recently, a textbook (Reference 5.A-7) by Zienkiewicz was published which contains many examples of solutions to practical problems using finite elements. This text also presents an excellent treatment of this powerful approach to the solution of problems in continuum mechanics.

5.A.2 ANALYTICAL METHOD

The finite element technique is a general method of structural analysis in which the continuous structure is replaced by a system of elements (members) connected at a finite number of nodal points (joints). Conventional analyses of frames and trusses, for example, can be considered as the application of the finite element method using one-dimensional elements. In utilizing the method to an axisymmetric solid (e.g., a concrete containment), the continuous structure is replaced by a system of rings of circumferential joints. Based on the energy principles, a set of force equilibrium equations are formed in which the radial and axial displacements at the circumferential joints are the unknowns of the system. A solution of this set of equations is inherent in the solution of the finite element system.

There are many advantages to the finite element method, when compared to other numerical approaches. The method is completely general with respect to geometry and material properties. Complex bodies composed of many different materials are easily represented; therefore, in the analysis of the containment, concrete, liner plate and foundation material can be realistically considered. Also, axisymmetric thermal, mechanical and gravity loadings can be analyzed.

It can be shown mathematically that the method converges to the exact solution as the number of elements in increased; therefore, any desired degree of accuracy may be obtained within the limits of computational capacity.

5.A.3 COMPUTER PROGRAM

The initial development of the computer program used in the analysis of the containment was conducted at the University of California at Berkeley in 1962, under a National Science Foundation Grant (G18986). Since that time the program has been further modified and refined by Dr. Edward L. Wilson. The validity of the specific program used in the containment analysis has been established by the analysis of axisymmetric solids with known exact linear solutions. It is noted that the results of a finite element analysis always satisfy statistics, since the equations solved within the computer program are based on the force equilibrium requirements.

5.A.4 COMPARISONS WITH KNOWN SOLUTIONS

An exact analysis of the containment structure under consideration is impossible by classic methods. A preliminary approximate analysis of the structure was conducted based on the classical shell theory. In addition to the difficulty in representing the steel liner, thickened portions of the shell and foundation materials, shell theory neglects the members thicknesses and shear deformations. Since the finite element approach includes the members thicknesses and shear deformation, an exact comparison with shell theory cannot be expected. However, forces obtained from the finite element method at sections not near discontinuities or the foundation do agree with the results, based on the shell theory.

Figure 5.A-1(c) illustrates the comparison of stresses from a classical problem for which an exact closed form solution exists and those obtained by the finite element method. The figure also shows the effect of the fineness of the finite element mesh on the degree of accuracy of the solution.

The problem is the determination of the radial and tangential stresses in an infinitely long, thick-walled cylinder of radius r and wall thickness of r/2, which is subjected to an internal pressure, p. (see Figure 5.A-1(b)). The values of the stresses through the wall thickness can be determined by the Lame solution. (Reference 5.A-8) The solution to this problem, using the finite element method, was carried out by E. L. Wilson. (Reference 5.A-9)

For the containment, comparisons have been made between the finite element method and an analysis done in accordance with the general shell theory for homogeneous surfaces of revolution. The matrix of influence coefficients (the unknown forces — deflections, moments and rotations for the dome, ring and cylinder) has been solved for the condition of equal deflection and rotations. Similar analytical methods have been used for the intersection of the base slab and the cylinder wall. The base slab has been analyzed as an elastic plate on an elastic foundation.

In general, the results thus obtained are within five percent of those obtained by the more rigorous finite element method for ring girder, dome, and cylinder wall of a similar containment.

For a typical finite element analysis, the foundation material and base slab interaction is studied by extending the mesh of the finite elements into the foundation material. Since the locations of the boundaries of the finite element mesh within the soil mass are a function of the soil properties, (Reference 5.A-10) studies are performed to determine where these boundaries should be located. After this has been determined, the final analyses of the containment by the finite element method will follow. Agreement between the results of this approach and the results of hand calculations, based on the assumption of an elastic plate on an elastic foundation, can be expected to be only approximate due to the difficulty of representing varying soil properties in the governing boundary conditions for the hand calculations.

Stresses in the regions of anchors have been determined using a plane strain analysis by the finite element method. Not only have the stresses resulting from the prestressing forces been determined, but also the effects due to the thermal and pressure loadings have been studied in the anchorage regions. The plane strain analysis is a better approximation than plane stress for the three-dimensional problem. However, since the program is prepared for plane stress analysis, modifications must be made in the elastic constants. The modulus of elasticity, E, is modified to $E/(1-v^2)$; Poisson's ration, v, is modified to $v/(1-v)$ and the linear thermal coefficient of expansion α, is modified to $\alpha(1+v)$.

5.A.5 REFERENCES

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5.A-2 Clough, R. W., "The Finite Element In Plane Stress Analysis," Proceedings of the Second ASCE Conference on Electronic Computation, Pittsburgh, PA., September 1960.

5.A-3 Wilson, E. L., "Finite Element Analysis of Two-Dimensional Structures," Structures and Materials Research, Department of Civil Engineering, Report Number 63-2, University of California, June 1963.

5.A-4 Hermann, L. R., "Finite Element Bending Analysis for Plates," Journal of Engineering Mechanics Division, ASCE, Volume 93, No. EM5, October 1967, pp 13-26.

5.A-5 Kapur, K. K., and B. J. Hartz, "Stability of Plates Using the Finite Element Method," Journal of Engineering Mechanics Division, ASCE, Volume 92, Number. EM2, April 1966, pp. 177-195.

5.A-6 Rashid, Y. R., "Analysis of Axisymmetric Composite Structures by the Finite Element Method," General Atomic Division of General Dynamics Corporation, Johns Hopkins Laboratory for Pure and Applied Science, San Diego, California. USAEC Contract at (04-3)-1967, Project Agreement Number 17.

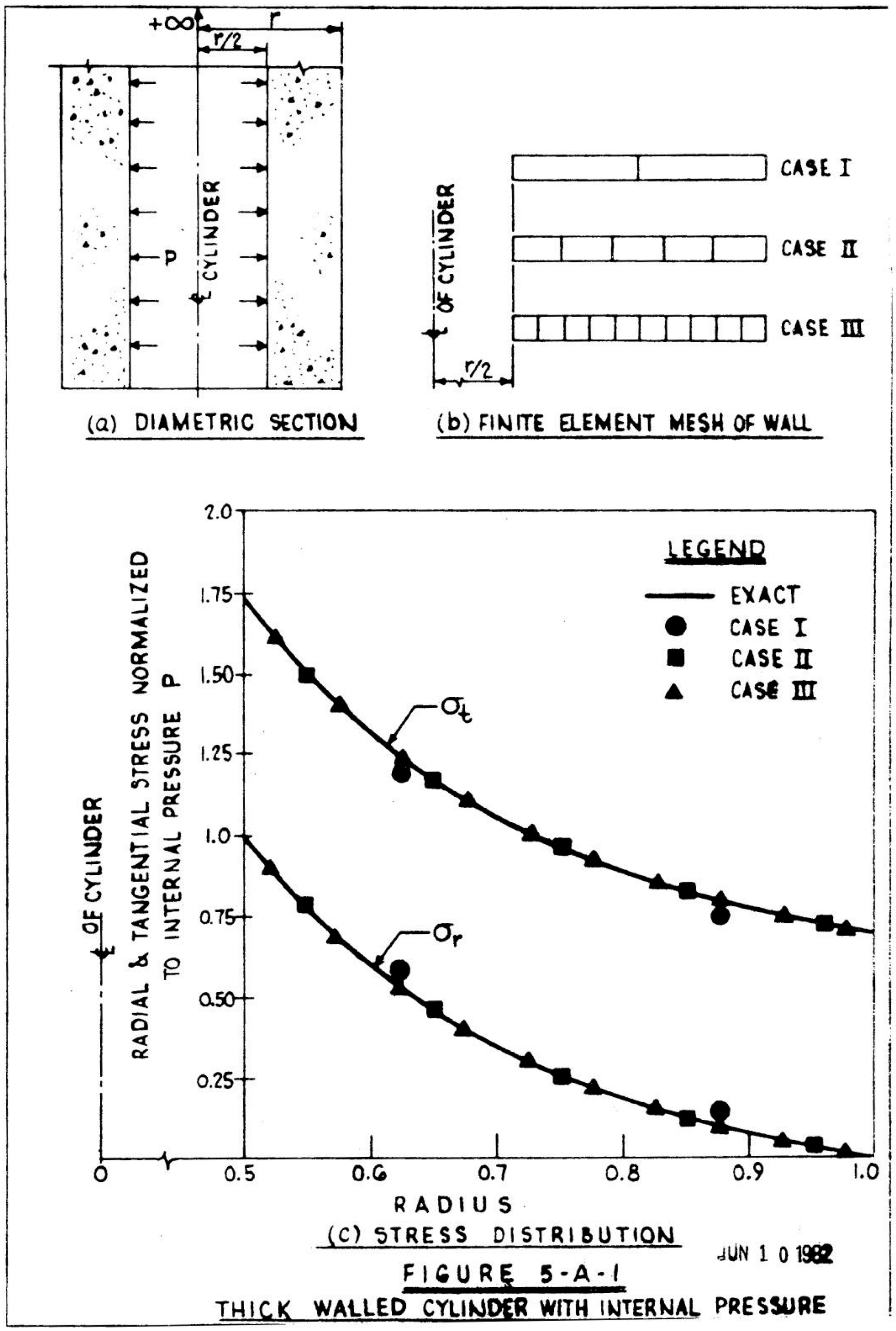
5.A-7 Zienkiewicz, O. C., "The Finite Element Method in Structural and Continuum Mechanics," McGraw-Hill, 1967.

5.A-8 Seely, F. B., and J. O. Smith, "Advanced Strength of Materials," Second Edition, John Wiley & Sons, Inc., New York, pp 295-304.

5.A-9 Wilson, E. L., "Structural Analysis of Axisymmetric Solids," Preprint 64-443 from AIAA 2nd Aerospace Sciences Meeting, New York, January 25, 1965.

5.A-10 Hoeg, K., J. T. Christian and R. V. Whitman, "Settlement of A Strip Load on Elastic-Plastic Soil," MIT Department of Civil Engineering, July 1967.

FIGURE 5.A–1 THICK WALLED CYLINDER WITH INTERNAL PRESSURE



5.B JUSTIFICATION FOR LOAD FACTORS AND LOAD COMBINATIONS USED IN DESIGN EQUATIONS OF CONTAINMENT

5.B.1 GENERAL

The load factors and load combinations in the design equations represent the consensus of the individual judgments of a group of Bechtel engineers and consultants who are experienced in both structural and nuclear power plant design. Their judgment has been influenced by current and past practice, by the degree of conservativeness inherent in the basic loads, and particularly by the probabilities of coincident occurrences in the case of incident, wind and tornadoes, as well as seismic loads.

The following discussions explain the justification for the individual factors, particularly as they apply to containment structures.

5.B.2 DEAD LOADS

Dead loads in a large structure such as this are easily identified and their effects can be accurately determined at each point in the structure. For dead loads in combination with the incident, seismic, or wind and tornado loads, a load factor representing a tolerance of five percent is chosen to account for dead load inaccuracies. The ACI Code allows a tolerance of +25 percent and -10 percent, but the code is written to cover a variety of conditions where weights and configurations of materials in and on the structure may not be clearly defined and are subject to change during the life of the structure.

5.B.3 LIVE LOADS

The live loads that would be present along with the incident, seismic, or wind and tornado loads would produce a very small portion of the stress at any point. Also, it is extremely unlikely that the full live loads would be present over a large area at the time of an unusual occurrence. For these reasons, a low load factor is felt to be justified and the live loads are considered together with dead loads at a load factor of 1.05.

5.B.4 SEISMIC LOADS

The operating basis earthquake that has been selected is considered to be the possible earthquake which could occur during the life of the plant. In addition, a design basis earthquake which defines the maximum credible earthquake which could occur at the site, is also considered in the design. Class I structures, systems, and equipment are designed so that no loss of function would result from the design basis earthquake. Consequently, the probability of an earthquake causing a loss-of-coolant incident is very small. For this reason, the two events, earthquake and the loss-of-coolant incident is very small. For this reason, the two events, earthquake and the loss-of-coolant incident, are considered together, but at much lower load factors than those specified for each separate event.

The seismic load factors of 1.25 and 1.00 are conservative for, respectively, the operating basis and the design basis earthquake in combination with the factored loss-of-coolant incident.

5.B.5 WIND AND TORNADO LOADS

Wind and tornado loads are determined from the velocities of the design wind and design tornado, respectively. With the containment designed for the extreme wind, it is inconceivable that the wind would cause a loss-of-coolant incident. Therefore, wind loads are not considered with the incident loads.

A load factor of 1.0 is applied to the tornado loads.

5.B.6 LOSS-OF-COOLANT INCIDENT

The design pressure and temperature are based on the operation of partial safeguards equipment using emergency diesel power.

European practice has been to use a load factor of 1.5 on the design pressure (Reference 5.B-1). This factor is reasonable and has been adopted for this design.

In all cases the design temperature is defined as that corresponding to the unfactored pressure. At 1.5 P, the temperature will be somewhat higher than the temperature at P. It would be unrealistic to apply a corresponding temperature factor of 1.5 since this could only occur with a pressure much greater than a pressure of 1.5 P.

5.B.7 REFERENCES

5.B-1 T. C. Waters and N. T. Barrett, “Prestressed Concrete Pressure Vessels for Nuclear Reactors,” Journal British Nuclear Society 2, 1963.

5.C JUSTIFICATION FOR CAPACITY REDUCTION FACTORS (ϕ) USED IN DETERMINING CAPACITY OF CONTAINMENT

The ϕ factors are provided to allow for variations in materials and workmanship. In the ACI Code 318-63, ϕ varies with the types of stresses or members considered; that is, with flexure, bond or shear stress, or compression.

The ϕ factor is multiplied into the basic strength equation or, for shear, into the basic permissible unit shear to obtain the dependable strength. The basic strength equation gives the "ideal" strength assuming that materials are as strong as specified, sizes are as shown on the drawings, the workmanship is excellent, and that the strength equation itself is theoretically correct. The practical, dependable strength may be something less since all these factors vary.

The ACI Code provides for these variables by suing these ϕ factors:

$\phi = 0.90$ for concrete in flexure

$\phi = 0.85$ for diagonal tension, bond, and anchorage

$\phi = 0.75$ for spirally reinforced, concrete compression members

$\phi = 0.70$ for tied compression members

The ϕ value is larger for flexure because the variability of steel is less than that of concrete and the concrete in compression has a fail-safe mode of behavior; that is, material understrength may not cause failure. The values for columns are lower (favoring the toughness of spiral columns over tied columns) because columns fail in compression where concrete strength is critical. Also, it is possible that the analysis might not combine the worst combination of axial load and moment. Since the member is critical in the gross collapse of the structure, a lower value is used.

The additional ϕ values used represent the best judgment of Bechtel as to how much understrength should be assigned to each material and condition not covered directly by the ACI Code. The additional ϕ values have been selected, based on material quality in relation to the existing ϕ values.

Conventional concrete design of beams requires that the design be controlled by yielding of the tensile reinforcing steel. This steel is generally spliced by lapping in an area of reduced tension. For members in flexure, ACI uses $\phi = 0.90$. The same reasoning has been applied in assigning a value of $\phi = 0.90$ to reinforcing steel in tension, which now includes axial tension. However, the Code recognizes the possibility of reduced bond of the bars at the laps by specifying a ϕ of 0.85. Mechanical and welded splices will develop at least 125 percent of the yield strength of the reinforcing steel. Therefore, $\phi = 0.85$ is recommended for this type of splice.

The only significantly new value introduced is $\phi = 0.95$ for prestressed tendons in direct tension. A higher value than that specified for conventional reinforcing has been allowed because: during installation the tendons are each jacked to about 94 percent of their yield strength, so in effect each tendon has been proof tested; and, the method of manufacturing prestressing steel (cold drawing and stress relieving) ensures a higher quality product than conventional reinforcing steel.

5.D EXPANDED SPECTRUM OF TORNADO MISSILES

The spectrum of tornado missiles is expanded to include the following:

1. Utility pole 13.5 inch diameter by 35 feet long with density of 43 lbs/ft^3
2. 1inch solid steel rod 3 feet long with a density of 490 lbs/ft^3
3. 6 inch, schedule 40 pipe, 15 feet long with a density of 490 lbs/ft^3
4. 12 inch, schedule 40 pipe, 15 feet long with a density of 490 lbs/ft^3

For each of the above tornado-borne missiles, the following information is provided:

1. The maximum velocity and height attained. Assuming in the analyses that each of the missiles originates at ground level and at the highest structural elevation on the site capable of producing each missile.
2. The required thickness of a reinforced concrete missile barrier to stop the missiles without their penetrating the missile barrier. Discussing the adequacy of all tornado missile barriers protecting systems and components necessary for safe shutdown.
3. The required thickness of a reinforced concrete missile barrier to preclude the generation of secondary missiles within the structure.
4. The effects that secondary missiles could have on safety related equipment and systems in the event that they occur.

In developing the above information, the analytical approach presented in BC-TOP-9, Design of Structure for Missile Damage with the following exceptions is used, assuming the missiles do not tumble and are at all times oriented such as to have the maximum value of C_dA/W while in flight.

<u>THE TORNADO MODEL</u>: The tornado model will be patterned after the Dallas tornado of April 2, 1957, as studied by Hoecker (Reference 5.D-1). The model is basically that given in WCAP-7897 (Reference 5.D-2) but with a more rigorous extrapolation to the parameters desired for a design tornado than given by Bates and Swanson (Reference 5.D-3).

Hoecker summarized his findings by the use of a “pressure-time profile” for an average translational velocity of 27 mph and as a function of percentage of total pressure drop.

In Attachment A, it is shown that when this time-pressure profile is used to solve the cyclostropic wind equation, the tangential wind velocities correspond with the experimental ones when a total pressure drop of 60 mb, or 0.882 psi, and a translational velocity of 27 mph is substituted into the equation.

When a total pressure drop of 3 psi and a 60 mph translational velocity (88 fps) is substituted into the same equation, a 304 mph maximum tangential velocity at a 300 foot radius is obtained. This corresponds closely to the assumptions which have been made in the past when describing the design tornado.

The two exponential equations used by Hoecker to determine the time-pressure profile cross each other at a radius of 1,240 feet instead of the 300 feet at which they cross when a translational velocity is 27 mph. Therefore, it is only necessary to use one equation since the starting tangential velocity corresponding to this distance is 66 mph, which is less than the minimum 75 mph considered by Bates and Swanson.

By incorporating these two assumptions, namely, that the vertical component is equal to one third of the tangential and the radial component is a function of radial distances between minimum and maximum tangential components being considered, a complete windfield was defined by using the following equations:

$$V_t = 249\sqrt{\frac{\mathrm{Exp}(-48.3V_1^3/R^3)V_1^3D}{R^3}} \qquad (1)$$

$$V_r = -\frac{(1240 - R)R}{(1240 - 300)} \qquad (2)$$

$$V_v = 1/3V_t \qquad (3)$$

Where:

V_t = tangential velocity (fps)

V_r = radial velocity (fps)

V_v = vertical velocity (fps)

V_1 = translational velocity (fps)

D = total pressure drop (psf)

R = radius (feet)

Equation (1) has been left in a general form for use in future models to predict different total pressure drops or translational velocities. However, at this time D is taken as 432 psf and V_1 = 88 fps.

The relative conservatism with reference to the actual Dallas tornado is shown in Figure 5.D–1.

The assumption of constant velocities from the ground to a height of 500 feet is a degree of conservatism which is justified by the expanded view of these velocities. This information has been published by Hoecker and is reproduced in Figure 5.D–2.

<u>TYPES OF MISSILES</u>: Previous studies had considered a car as a missile for low elevations and a wooden plank for high elevations. Later a small cross-section missile in the form of a pipe was added. Five missiles are now being considered.

All the missiles are intended as prototypes of the many missiles generated by tornados. Considering the present (1973) state of the art, a detailed physical description of a missile is of little value when designing the missile proof target. Empirical formulations have to be used in areas where impactive energy and the impactive area are the points to be considered.

The more logical approach is to assume a generalized range of missiles with the required drag factors impacting at given elevations with the highest possible velocity. The impactive kinetic energy per square foot of impact area for each elevation would then be computed.

If a table is made with C_dA/W factors from 0.10 to 0.15, which is the smallest measurement for an airborne missile, it will be found that there is a drag factor that will give the highest velocity at each elevation. This is shown in Table 5.D-1.

It is interesting to show the small range and the gap left for the maximum drag factor proposed.

Wooden plank	0.06
Utility pole	0.026
Steel rod	0.031
6 inch pipe	0.029
12 inch pipe	0.021

Impactive energy per unit area measured in lb/ft as shown on Table II is readily found as follows:

$$K = \frac{W V^2}{2g\ A} = \frac{V^2}{2g\ F}$$

Where:

K = impactive factor (lb/ft)

V = velocity at impact (fps)

A = area of impact (ft^2)

F = C_dA/W for $C_d = 1$

W = weight of missile (pounds)

g = acceleration of gravity (ft/sec^2)

As expected, this impactive factor is much higher at lower elevations: it varies from 49,640 lb/ft for a 10 foot elevation to 22,070 lb/ft for a 110 foot elevation.

Penetrations can be computed by the required empirical formulation which is workable in terms of these impactive factors.

METHODS OF INJECTION AND PROPULSION: Bates and Swanson propose three methods of injection:

a. Explosive injection

b. Aerodynamic injection

c. Ramp injection

These are intended to limit the height at which a given object may be injected into a tornado. So many considerations and assumptions have to be made that they become of no practical value when it is to be assumed that the object will reach the highest point of a structure even if the missile has to be held at a convenient elevation for injection to occur.

If an explosive injection occurs some distance away from a structure, it is concluded that the object could clear the structure, if such an injection could occur. Aerodynamic injection will require aerodynamic objects or else the injection is overestimated. Likewise, a ramp injection will depend on the given ramp, a factor that is hard to generalize.

All three methods of injection required many assumptions which make it difficult for generalization. A fourth method which would be called the “Uplift Injection” offers the advantages of simplicity and applicability.

In the uplift injection it is assumed that the wind finds its way beneath a surface and the object will become airborne at the time when the vertical component of the wind produces an upward force equal to the weight of the object. While on the ground the object is assumed to be free to move on the horizontal plane in a frictionless manner as the tangential and radial components of the wind act on it.

When a missile flight is to be ascertained by applying the three components of the wind (tangential, radial and vertical) simultaneously, a random surface is assumed to be facing all three components. This random surface will produce what is called in WCAP-7897 an “effective drag factor” to be applied in all directions and which is computed as follows:

Cylinder:

$$C_e = \frac{0.389(h + 0.66D)}{P_{obj}\ hD}$$

Parallelpiped:

$$C_e = \frac{0.483\ w(h+d)}{P_{obj}\ w\ h\ d}$$

where:

C_e = effective drag factor

h = length, feet

D = diameter, feet

P_{obj} = density, lb/ft^3

w = width, feet

d = depth, feet

To date, this is the best method of computing an effective drag area for an object thrown into a tornado.

Using these effective drag areas in the computer program, DALLAS MISS GEN, the following results (velocities in ft/sec) were obtained:

Elevation	**Wooden Plank $C_e = 0.03$**	**Utility Pole $C_e = 0.0082$**	**Steel Rod $C_e = 0.0097$**	**6 inch Pipe $C_e = 0.0015$**	**12 inch Pipe $C_e = 0.00078$**
10	223	182	192	98	---
20	264	---	---	--	---
30	279	---	---	--	---
40	270	---	---	--	---
50	261	---	---	--	---
60	258	---	---	--	---

These results show that only the wooden plank type missile could be sustained in the air. It supports WCAP-7897, Chapter 5: "Investigation of Some Specific Missiles" which clearly states: "The results of Figure 3 indicate that objects with a C_dA/W less than 0.012 ft^2/lb will not be sustained by the vertical wind even if injected above immediate obstructions." (Figure 3 is contained in WCAP-7897.)

It appears then that instead of assuming impossible C_dA/W factors for a given set of missiles, it is best to assume an infinite number of missiles, all with possible effective drag factors.

The required thickness of a concrete element that will just be perforated by a missile is given by:

$$T = \frac{427\ W}{\sqrt{f'_c}\ D^{1.8}}\left(\frac{V_s}{1000}\right)^{1.33}$$

where:

T = Thickness of concrete element to be just perforated (inches)

W = Weight of missiles (pounds)

D = Diameter of missiles (inches)

Note: For irregularly shaped missiles, an equivalent diameter is used. The equivalent diameter is taken as the diameter of a circle with an area equal to the circumscribed contact, or projected frontal area, of the non-cylindrical missile.

V_s = Striking velocity of missile (ft/sec)

f'_c = Compressive strength of concrete (psi)

This formula is known as the Ballistic Research Laboratory (BRL) formula as presented in Reference 5.D-5.

The thickness, t_p, of a concrete element required to prevent perforation must be greater than T. It is recommended to increase T by 25 percent, but not more than 10 inches, to obtain the t_p, required to prevent perforation.

$t_p = 1.25T \leq T + 10$ (in inches)

The results obtained by using the above formula are presented in Table 5.D-5. The concrete barriers furnished to protect systems and components necessary for safe shutdown exceed the required thickness to prevent perforation by the missiles.

Spalling of concrete from the side opposite the impact surface of the element is considered as a secondary missile. For an estimate of the thickness that will just start spalling, it is recommended that the following equation be used:

$$Ts = 2T$$

where:

T_s = Concrete element thickness that will just start spalling (inches)

T = Concrete thickness to be just perforated (inches).

The thickness, t_s, of a concrete element required to prevent spalling must be greater than T_s. It is recommended to increase T_s by 25 percent, but not more than 10 inches, to prevent spalling.

$$t_s = 1.25T_s \leq T_s + 10 \text{ (in inches)}$$

The results obtained by using the above formula are presented in Table 5.D-5.

The BRL formula was selected after a thorough study of all available formulae in the literature for concrete perforation and spalling due to missile impact. As with all other available formulae, the BRL formula represents an empirical expression based upon high velocity test data and was developed for use in the high velocity range (i.e., missile impact velocity in excess of 1,000 ft/sec). The range of missile velocities considered in a nuclear facility is generally below 500 ft/sec. In order to provide a confidence margin for the lower velocity range, and to assure that barrier thickness would exceed that at which perforation or spalling impends, the design thickness was increased.

Test data on the impact of a one-inch diameter steel rod having a velocity from 150 ft/sec to 320 ft/sec on concrete barriers of 3 inches, 6 inches and 9 inches in thickness indicate that these formulae provide conservative results for both concrete perforation and spalling in the velocity range as stated. A summary of the test results is presented in Table 5.D-6.

A procedure for determining thickness of spalling is presented in Reference 5.D-4. The spalling effects on concrete wall due to the impacts of wooden plank and utility pole were investigated and, in both cases, no spalling of concrete wall was indicated. Therefore, the secondary missiles are not considered credible.

The thickness, t_s, of a concrete element required to prevent spalling is more than the thickness, T_m, of a concrete element furnished in the cases of wooden plank and utility pole, as indicated in Table 5.D-5. The thickness, t_s, provides a simplified approach of determining a thickness required for a concrete barrier to stop a missile. A margin of safety, an increase of 25% of the calculated values with an upper limit of 10 inches, is a logical safety factor against spalling or perforation and is further reinforced by the test data presented in Figure 5.D–3. The formula used to determine the thickness of spalling does not consider reinforcing steel which tends to reduce the amount of spalling. If t_s is less than T_m, as in the case of 3 inch steel rod and 6 inch pipe, no additional analysis is required.

To determine the thickness of spalling, the following formula is presented in Reference 5.D-4.

$$X_{max} = \frac{282\ N\ W}{\sqrt{f'_c}\ d^{1.8}}\left(\frac{V}{1000}\right)^{1.8}$$

$$C_1 = \left(\frac{T}{X_{max}} + C_3 - 1\right)\left(\frac{C_s}{V}\right)^{1/3}$$

$$C_3 = \frac{0.877\ W^{1/3}}{X_{max}}$$

$$\Delta_{t(initial)} = C_2 X_{max}\left(\frac{V}{C_s}\right)^{1/3}$$

$$\Sigma\Delta_t = \frac{1}{8}[T + 0.877\ W^{1/3} + \Delta_{t(initial)} - X_{max}]$$

where:

V = velocity of missile, ft/sec

W = weight of missile, pounds

T = target thickness, inches

d = diameter of missile, inches

f'_c = concrete strength, psi

$\Delta_{t(initial)}$ = thickness of initial spall, inches

N = nose factor = 0.845 for hemi-spherical nose

C_2 = coefficient from Figure 4.10 of Reference 5.D-4

C_s = dilational velocity in concrete = 9,800 ft/sec

X_{max} = concrete thickness to just perforate, inches

By substituting the following values into the equations, the value of $\sum\Delta_t$, total thickness of all spalling, for the wooden plank and the utility pole was determined to be insignificant.

<parameter>	**Wooden Plank**	**Utility Pole**
Weight	105 pounds	1,500 pounds
Velocity	280 fps	182 fps
Target Thickness	12 inches	24 inches
Diameter of Missile	7.82 inches	13.5 inches
Concrete Strength	3,000 psi	3,000 psi

5.D.1 REFERENCES

5.D-1 W. H. Hoecker, Jr., “Three Dimensional Pressure Pattern of the Dallas Tornado and Some Resultant Implications,” Monthly Weather Review, 89, 12, 533, 1961.

5.D-2 D. F. Paddleford, “Characteristics of Tornado Generated Missiles,” Westinghouse Electric Corporation, WCAP-7897, April 1969.

5.D-3 F. C. Bates and A. E. Swanson, “Tornado Design Considerations for Nuclear Power Plants,” ANS Transactions, November 1967.

5.D-4 “Industrial Engineering Study to Establish Safety Design Criteria for Use in Engineering of Explosive Facilities and Operations Wall Response,” a Report Submitted to Process Engineering Branch, A.P.M.E.D. Picatinny Arsenal, Dover, N. J., Ammann & Whitney Consulting Engineers, April 1963.

5.D-5 “Topical Report: Design of Structures for Missile Impact,” Bechtel Corporation, BC-TOP-9, Revision 1, July 1973.

5.D-6 G. W. Reynolds, “Report to Bechtel Power Corporation,” August 29, 1973.

5.D-7 R. C. Gwaltney, “Missile Generation in Light Water Cooled-Power Reactor Plants,” ORNL-NSIC-22, September 1968.

5.D-8 D. A. Miller and W. A. Williams, “Tornado Protection for the Spent Fuel Storage Pool,” APED 5696, November 1968.

5.D-9 E. M. Brooks, “Quantitative Models of Wind Velocity Components in a Tornado Vortex,” St. Louis University Report No. 1 to the U.S. Weather Bureau, 1957.

TABLE 5.D-1 IMPACTIVE VELOCITIES (fps) OF MISSILES OF DIFFERENT C_dA/W FACTORS AS PICKED FROM THE GROUND BY THE DESIGN TORNADO

Elevation	C_dA/W	0.1	0.09	0.08	0.07	0.06	0.05	0.04	0.03	0.02	0.015
10		142	148	153	161	171	183	200	223	244	219
20		167	175	184	194	206	226	244	264	246	---
30		194	203	214	226	241	259	278	279	238	---
40		223	233	246	259	274	291	300	270	238 [a]	---
50		253	263	276	290	305	313	301	261	---	
60		283	294	307	317	327	321	290	258 [(a)]	---	
70		312	322	333	339	336	313	279	---		
80		339	347	352	349	331	300	273	---		
90		361	364	360	346	318	290	271[(a)]			
100		375	369	355	331	304	283	---			
110		377	363	341	316	294	280	---			
120		367	346	324	303	287	280 [(a)]				
130		349	327	309	293	284	---				
140		330	312	298	287	283 [(a)]	---				
150		311	298	290	285	---					
160		297	290	284	285[(a)]	---					
170		287	283	283 [(a)]	---						
180		279	280	---							
190		276	280[(a)]	---							
200		275	---								

a. Missile reached the condensation funnel at some elevation above the previous one.

TABLE 5.D-2 KINETIC ENERGY PER FT^2 OF IMPACT AREA

C_d A/W	Max Impact Velocity (fps)	At Elevation (ft)	Kinetic Energy/Ft^2 (lb/ft)
0.015	219	10	49,640
0.02	246	20	44,747
0.03	279	30	40,290
0.04	301	40	35,171
0.04	301	50	35,171
0.05	321	60	32,000
0.06	336	70	29,217
0.07	349	80	27,019
0.08	360	90	25,155
0.09	369	100	23,492
0.10	377	110	22,070

Use the last figures for elevations above 110 feet.

TABLE 5.D-3 RADIUS VS. VELOCITY FPS/MPH (D=0.882 PSI/V_1 = 27 MPH)

RADIUS	VELOCITY FPS	VELOCITY MPH
1500	32.63	22.25
1400	33.75	23.01
1300	35.00	23.86
1200	36.39	24.81
1100	37.96	25.88
1000	39.76	27.11
900	41.85	28.53
800	44.29	30.20
700	47.22	32.20
600	50.83	34.65
500	55.40	37.77
400	61.48	41.92
300	127.30	86.80
290	133.15	90.78
280	139.39	95.04
270	146.05	99.58
260	153.15	104.42
250	160.71	109.58
240	168.74	115.05
230	177.22	120.83
220	186.14	126.91
210	195.43	133.25
200	204.97	139.75
190	214.57	146.29
180	223.90	152.66
170	232.49	158.52
160	239.58	163.35
150	244.08	166.42

TABLE 5.D-3 RADIUS VS. VELOCITY FPS/MPH (D=0.882 PSI/V_1 = 27 MPH)

RADIUS	VELOCITY FPS	VELOCITY MPH
140	244.40	166.63
130	238.39	162.53
120	223.34	152.27
110	196.44	133.93
100	156.08	106.41
90	104.68	71.37
80	52.23	35.61
70	15.07	10.27
60	1.45	.99
50	.01	.01
40	0	0
30	0	0
20	0	0
10	0	0

Where: D = 0.882 psi

V_1 = 27 mph

TABLE 5.D-4 RADIUS VS. VELOCITY FPS/MPH (D=3 PSI/V_1 = 60 MPH)

RADIUS	VELOCITY FPS	VELOCITY MPH
1500	88.63	60.43
1400	91.59	62.45
1300	94.88	64.69
1200	98.54	67.19
1100	102.67	70.00
1000	107.35	73.19
900	112.74	76.87
800	119.03	81.16
700	126.50	86.25
600	135.56	92.42
500	146.86	100.13
400	161.49	110.10
300	446.95	304.74
290	440.56	300.38
280	430.85	293.76
270	417.34	284.55
260	399.52	272.40
250	376.98	257.03
240	349.39	238.22
230	316.68	215.92
220	279.11	190.30
210	237.44	161.89
200	193.06	131.63
190	148.07	100.96
180	105.24	71.75
170	67.63	46.11
160	37.97	25.89
150	17.73	12.09

TABLE 5.D-4 RADIUS VS. VELOCITY FPS/MPH (D=3 PSI/V_1 = 60 MPH)

RADIUS	VELOCITY FPS	VELOCITY MPH
140	6.40	4.36
130	1.60	1.09
120	.23	.16
110	.02	.01
100	0	0
90	0	0
80	0	0
70	0	0
60	0	0
50	0	0
40	0	0
30	0	0
20	0	0
10	0	0

Where: D = 3 psi

V_1 = 60 mph

TABLE 5.D-5 VELOCITIES OF VARIOUS MISSILES

Missiles	Maximum Velocity (fps)	Maximum Height (ft.)	t_p (in.)	t_s (in.)	t_m (in.)
Wooden Plank	280	60	6.5	13.0	12.0
Utility Pole	182	10	14.0	28.1	24.0
1 inch Steel Rod	192	10	8.7	17.4	24.0
6 inch Pipe	98	10	5.1	10.1	24.0
12 inch Pipe	-	-	-	-	-

t_m = Minimum concrete thickness furnished.

TABLE 5.D-6 TEST DATA SUMMARY

Test Number	Test Date (July)	Target Thickness (inches)	Missile		Velocity		Penetration		Spalling
			Length (feet)	Weight (lbs)	Electronic (ft/sec)	Movies (ft/sec)	Depth (inches)	Volume (in^3)	
1	2	9	3	8.05		203	1.71	21	None
2	3	6	3	8.04	213	211	1.60	11.6	Small & cracks
3	3	3	3	8.05	218	213	Perforated	6.1/30.5 front/back	Maximum travel of 144 ft.
4	4	6	3	8.05	220	214	1.98	17.4	Large & cracks
5	4	6	1.4	3.78	322	312	1.68	21.6	Moderate & cracks
6	4	9	3 *	8.01	235 **	214	2.34	22	None
7	5	9 ***	3	8.04	217	216	1.84	19.9	None
8	5	6	3	8.04	150	151	1.23	7.6	Small & cracks

* Rounded end.

** Believed to be bad reading due to rounded missile end.

*** No reinforcing in slab.

FIGURE 5.D–1 TANGENTIAL VELOCITIES AS DERIVED FROM HOECKER'S PRESSURE - TIME PROFILE

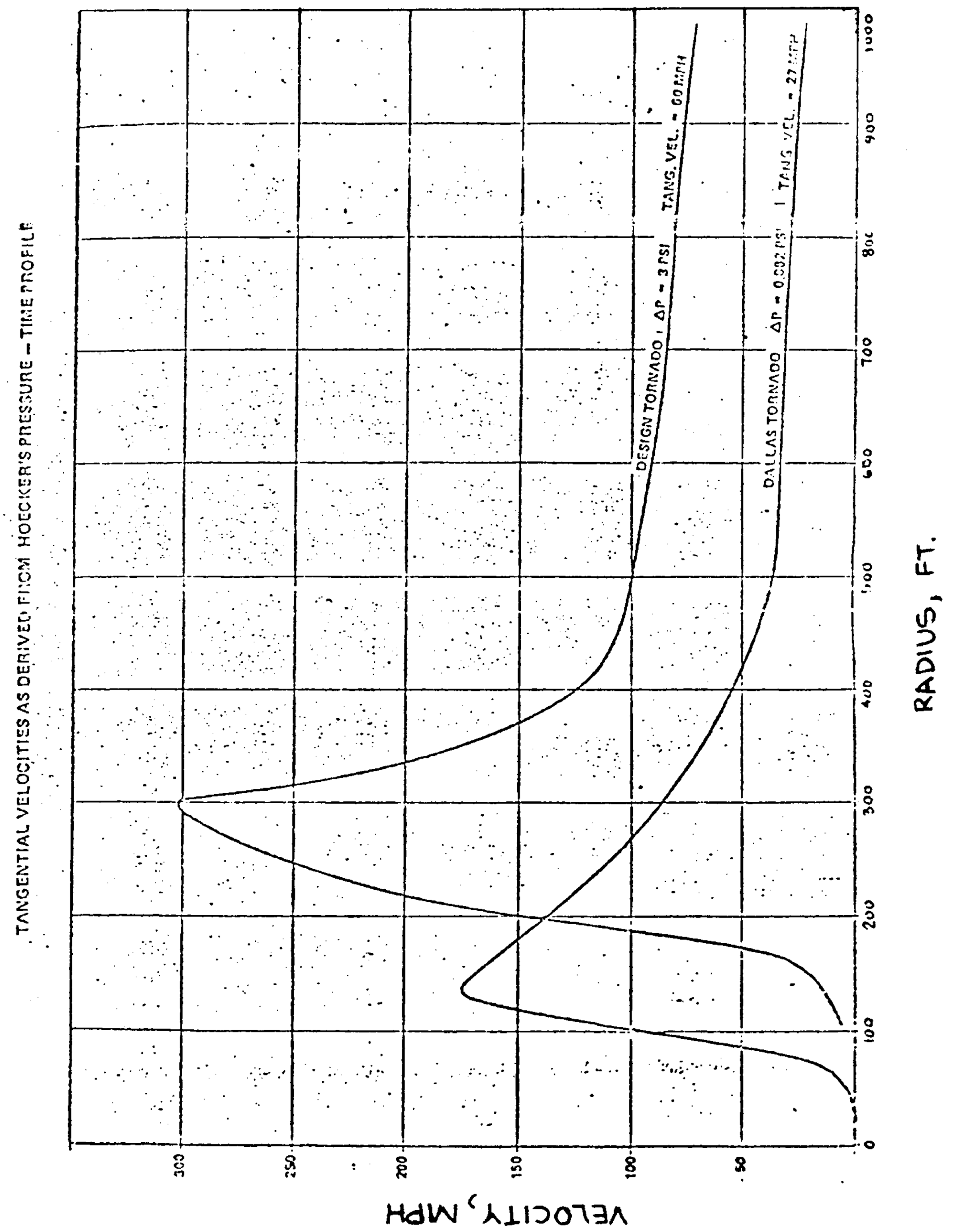


FIGURE 5.D-1

JUN 1 0 1982

FIGURE 5.D–2 MONTHLY WEATHER REVIEW - HOECKER

May 1960 MONTHLY WEATHER REVIEW

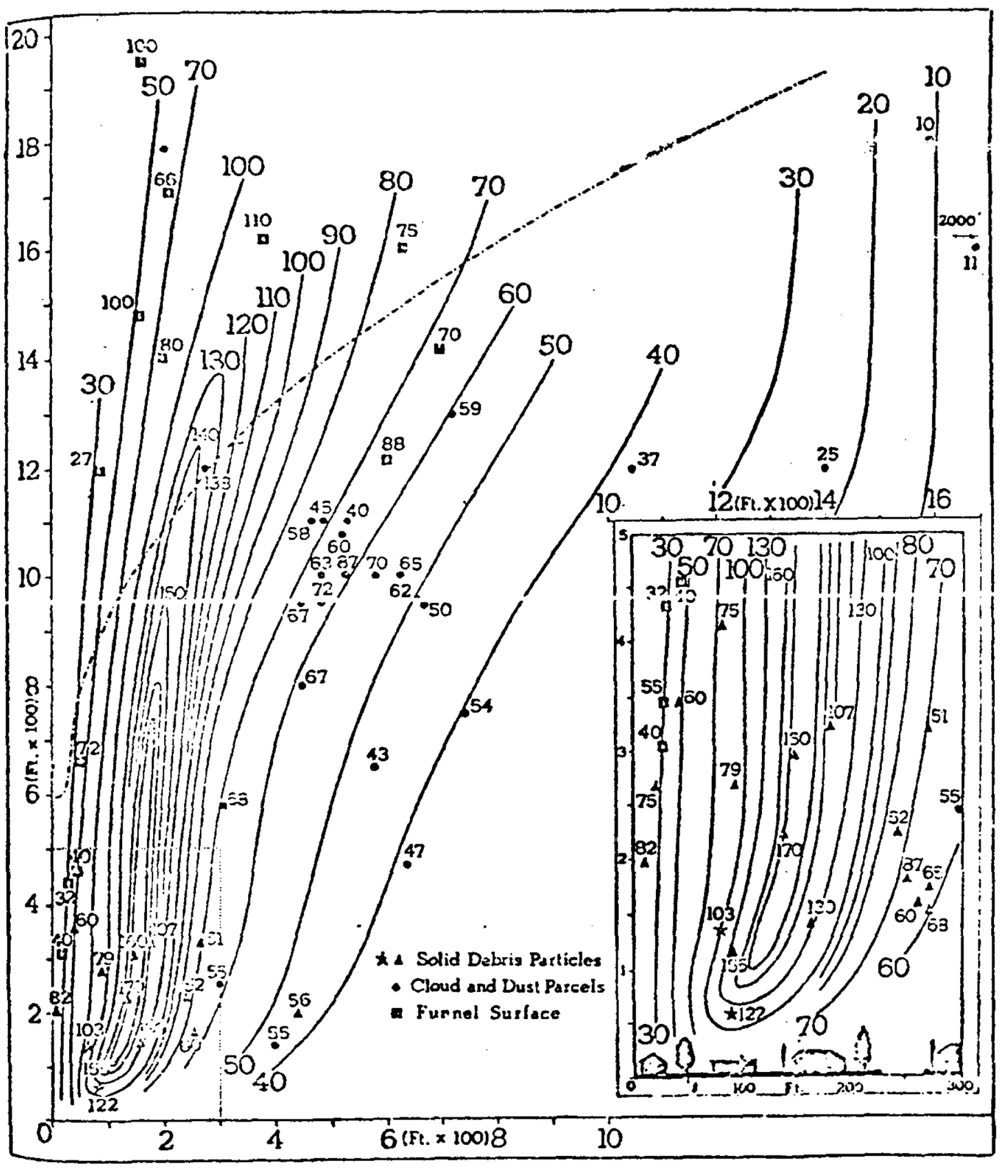


Figure 2.--Distribution of derived tangential speed from the center of the tornado to a radius of 2000 ft. and from near the ground to about 1500 ft. in elevation.

FIGURE 5.D-2

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FIGURE 5.D–3 TEST DATA SUMMARY

FIGURE 5.D-3

TEST DATA SUMMARY

Test No.	Test Date (July)	Target Thickness (ins.)	Missile		Velocity		Penetration		Spalling
			Length (ft)	Weight (lbs)	Electronic (ft/sec)	Movies (ft/sec)	Depth (in)	Volume (in^3)	
1	2	9	3	8.05		203	1.71	21	None
2	3	6	3	8.04	213	211	1.60	11.6	Samm & Cracks
3	3	3	3	8.05	218	213	Perforated	6.1/30.5 front/back	Maximum travel of 144 ft.
4	4	6	3	8.05	220	214	1.98	17.4	Large & cracks
5	4	6	1.4	3.78	322	312	1.68	21.6	Moderate & cracks
6	4	9	3*	8.01	235***	214	2.34	22	None
7	5	9**	3	8.04	217	216	1.84	19.9	None
8	4	6	3	8.04	150	151	1.23	7.6	Small & cracks

* Rounded end
** No reinforcing in slab
*** Believed to be bad reading due to rounded missile end.

[illegible] 1 0 [illegible]

5.D.A THE DEVELOPMENT OF THE WIND-FIELD, TANGENTIAL VELOCITY

A wind-field may be generated by using the experimental findings of Hoecker (1) in the Dallas Tornado of April 1954, which was expressed in mathematical form as a “pressure-time profile” as follows:

$$p = [1 - \mathrm{Exp}(-0.755/t)]\ D \text{ for 7.6 to 37.9 seconds} \quad (1)$$

$$p = [1 - \mathrm{Exp}(-48.3/t^3)]\ D \text{ for 0 to 7.6 seconds} \quad (2)$$

(Intended only for all positive values of the time t)

Where:

t = time of arrival

D = total pressure drop in psf

In this publication it is clearly established that the distance from axis (R) varied from a radius of 1,500 ft at 37.9 seconds to 300 ft at 7.6 seconds, in other words for a translational velocity (V_1) of 27 mph. That is to say a relation between t and R is readily found as follows:

$$t = (R/V_1) \quad (3)$$

Therefore equations 1 and 2 may be written in terms of radius as follows:

$$p = [1-\mathrm{Exp}(-0.775V_1/R)]D \quad (4)$$

$$p = [1-\mathrm{Exp}(-48.3V^3{}_1/R^3)]D \quad (5)$$

The wind cyclostrophic equation was defined in the same publication as

$$dp/dR = \rho V^2/R \quad (6)$$

Where the partial differentiation of pressure to radius is equated to the mass density of the wind (ρ) times the square of the tangential velocity (V) divided by the radius (R).

By differentiating equations 4 and 5 it is found

$$dp/dR = -[\mathrm{Exp}(-0.755\ V_1/R)](0.755V1/R^2)(D) \quad (7)$$

$$\frac{dp}{dR} = -[\mathrm{Exp}(-48.3\ V_1^3/R^3)](144.9\ V_1^3/R^4)(D) \quad (8)$$

And substituting in equation 6 the following expressions are obtained which will relate tangential velocities to radius.

Introducing these differentiations in the cyclostrophic equation:

$$-[\mathrm{Exp}(-0.755\ V_1/R)](0.755\ V_1/R^2)(D) = \frac{V^2}{-R} \tag{9}$$

$$-[\mathrm{Exp}(-48.3\ V_1^3/R^3)](144.9\ V_1^3/R^4)(D) = \frac{V^2}{-R} \tag{10}$$

By assigning a constant value of 0.075/g for the mass density of air and solving for V, the following expressions are obtained:

$$V = 18\sqrt{\frac{[\mathrm{Exp}(-0.755\ V_1/R)](\ V_1)(D)}{R}} \tag{11}$$

$$V = 249\sqrt{\frac{[\mathrm{Exp}(-48.3\ V_1^3/R^3)](\ V_1^3)(D)}{R^3}} \tag{12}$$

This equation will give us the tangential velocities as a function of the radius. Equation 11 and 12 are applied, first to the Dallas Tornado where D = 60 mb = 0.882 psi and V_1 = 27 mph; then to the Design Tornado where D = 3 psi and V_1 = 60 mph. The results are shown in Tables 5.50-3 and 5.50-4.

Equations 11 and 12 coverage where R* = 1,240 feet when both equations give a tangential velocity of about 97 ft/sec about 66 mph which could be taken as the initial wind velocity to be considered at a radius of 1,240 feet. This will add some conservatism to our computations while obviating the use of two equations.

In other words: Starting at 1,240 foot radius with 66 mph wind is more conservative than starting at 600 foot radius with 75 mph winds, as done by WCAP 7897.

RADIAL WIND VELOCITY

The radial component of the wind will be computed using the same expression as that of WCAP except that the radius of 66 mph instead of 75 mph is chosen. So that:

$$V_2 = -\left(\frac{1240 - R}{1240 - 300}\right)R$$

Where 300 is the radius of the maximum tangential velocity.

VERTICAL COMPONENT

The vertical wind component will be taken as one third of the tangential as done previously.

$$V_3 = 1/3(V_1)$$

CONSERVATISM

A. CONSERVATISM IN THE WINDFIELD

a. Three (3) psi Total Pressure Drop

The equation of maximum tangential velocity was presented as follows:

$$V_t = 249\sqrt{\mathrm{Exp}[-48.3(V_1/R)^3](V_1/R)^3 D} \qquad (1)$$

Where:

V_1 = translational velocity (fps)

D = total pressure drop (psf)

R = radius (ft)

Substituting $R/V_1 = t$, Equation (1) becomes

$$V_t = 249\sqrt{D}\sqrt{\frac{1}{e^{48.3/t^3}(t^3)}} \qquad (2)$$

The last term in Equation (2) maximizes at t = 3.64 seconds when its value is equal to 0.08727. Therefore:

$$V_{t(max)} = (249)(0.08727)\sqrt{D} \qquad (3)$$

From Equation (3), we relate the maximum velocities to the total pressure drops.

Pressure Drop (psi)	**Pressure Drop (psf)**	**Maximum Tangential Velocity (fps)**	**Maximum Tangential Velocity (mph)**
0.50	72	184.4	126
1.00	144	260.8	178
1.50	216	319.4	218

Pressure Drop (psi)	Pressure Drop (psf)	Maximum Tangential Velocity (fps)	Maximum Tangential Velocity (mph)
2.00	288	368.8	251
2.50	360	412.3	281
3.00	432	451.7	308

The tornado model, as described above, is applicable only to the Dallas Tornado as studied by Hoecker (Reference 5.D-1). Nevertheless, E. M. Brooks (Reference 5.D-9) submitted to the U.S. Weather Bureau a report in which he equated kinetic energy per unit volume to one-half the work done on the unit volume. He computed the following values:

Pressure Drop (inches Hg)	Pressure Drop (psi)	Tangential Velocity (mph)
5	2.5	285
6	3.0	316
7	3.5	348

There is exceptionally good agreement between these figures and those previously presented. One set is computed by extrapolating experimental evidence, and the other is computed theoretically.

The 308 mph and the 3 psi are the result of extrapolating Hoecker's pressure profile which shows a good agreement with experiments. Therefore, the most important parameter, the tangential velocity, is validated by both experiment and theory.

The degree of conservatism of this windfield should be recognized since the pressure drops recorded are less than the 3 psi proposed here. Dr. Reynolds, in his report (Reference 5.D-6), stated, "As far as I know, no pressure drop of as much as an inch of mercury (about 0.5 psi) has been officially recorded. Although unofficially made, spectacular atmospheric pressure measurements have been reported for tornadoes."

b. Translational Velocity of 60 mph

The conservatism of assuming a three (3) psi total pressure drop is presented above and shows that the maximum tangential velocity of 308 mph with respect to three (3) psi total pressure drop is obtained. The same tangential velocity can be obtained for tornadoes with various translational velocity by varying the radius of a tornado. If 60 mph translational velocity is used, as in the case for Millstone Unit 2, the maximum tangential velocity for a missile is more than that associated with 27 mph translational velocity, as in the case of the Dallas tornado. Therefore, the translational velocity of 60 mph, as assumed in the analysis for Millstone is more conservative than the actual record as illustrated in the following table.

Assume a flight parameter = 0.10

Height	Tangential Velocity with 60 mph	Tangential Velocity with 27 mph
10	142	142
20	167	214
30	194	276
40	223	296
50	253	285
60	283	276
70	312	273
80	339	---
90	366	---
110	377	---

B. CONSERVATISM IN THE COMPUTATIONAL PROCEDURE

Maximum velocity and height associated with a flight parameter are obtained from the following procedures:

a. Relative Wind Velocity

It has been customary for WCAP (Reference 5.D-2) and others to consider the relative wind velocity with respect to the velocity of the flying object in the following manner:

$$\ddot{X} = \frac{C_d A}{W}\frac{\rho}{2}\; V_r(V_x - U_x) \qquad (4)$$

Where:

$$V_r = \sqrt{(V_x - U_x)^2 + (V_y + U_y)^2 + (V_z - U_z)^2}$$

$V_{x,y,z}$ = Wind velocities in the three coordinates

V_r = Relative wind velocity

$U_{x,y,z}$ = Object velocities in the three coordinates

$\overline{X}$ = Acceleration

$C_d = 1$

A= Projected area of missile in flight

W = Weight of missile

ρ = Density of missile

It should be apparent that, as the object reaches the wind velocities, not only will the terms in parentheses in Equation (4) decrease, but the value of V_r also will decrease.

b. Asolute or Actual Wind Velocities

It isn't quite clear that the relative velocity of the fluid should be used when the moving body is assumed to be immersed in the fluid propelling it. Gwaltney (Reference 5.D-7), when considering an object propelled by steam, uses the absolute or actual velocity of the steam in the following manner:

$$\ddot{X} = \frac{C_d A}{W} \frac{\rho}{2} V_a(V_x - U_x) \qquad (5)$$

Where:

V_a = Absolute wind velocity

$V_a = (V^2{}_x + V^2{}_y + V^2{}_x)^{1/2}$

Here, the absolute or actual velocity of the wind obviously will increase to its actual maximum value irrespective of the missile velocity. Consequently, the accelerations are greater.

c. Missile is Assumed to Stay in the Windfield

In the windfield presented by WCAP, at a time when the tangential velocity increases and the radial velocity decreases rapidly, the missile may leave the windfield. An analysis shows that the velocity at which the missile leaves the windfield is less than that calculated above by assuming the missile stays in the windfield. Therefore, it is more conservative to assume a missile strikes a target at its maximum velocity while it is within a windfield.

C. CONSERVATISM IN FLIGHT PARAMETER

The magnitude of the flight parameter depends essentially on the missile area projected to the wind. Accelerations and velocities are directly proportional to this area.

However, the governing parameter, as far as penetration is concerned, is a function of the weight, velocity, and impactive area of a missile.

$$P = f(W,V/A) \tag{6}$$

Where:

P = Penetration

W = Weight of missile

A = Impactive area

This function in most penetration formulae can be identified as the energy/area ratio.

When a flying object is following a spiral trajectory, it is difficult to calculate its velocity unless a common area is used. Dr. Bates (Reference 5.D-3) produced what he called the “effective area” to solve this problem. Justification for using the effective area in our computation is as follows:

The important thing to consider here is the significance of assuming one area for the flight parameter (even if it is the effective area) and another for impact. This situation is maximized in the following table:

<object>	Effective area in flight Minimum area on impact	Maximum area in flight Maximum area on impact
Plank	711,000 lb/ft	29,200 lb/ft
Pole	771,500 lb/ft	30,800 lb/ft
Rod	848,000 lb/ft	24,900 lb/ft
6 inch Pipe	732,900 lb/ft	25,700 lb/ft
12 inch Pipe	414,800 lb/ft	27,800 lb/ft

It is hypothetical to consider that a missile can maintain its maximum projected area flight and strike a target with its minimum area. Therefore, it is indicated from the above table that the assumption of using effective area in flight and minimum area on impact is a very conservative approach.

D. CONSERVATISM IN THE MAXIMUM HEIGHT

A distinction should be made between the height that a tornado missile could reach before its velocity is zero and the height at which it obtains maximum tangential velocity. The damaging effect of a missile depends on its impactive velocity and not the height it may reach.

The heights reported for Millstone are those at which the missiles are traveling at their maximum velocities.

From the analyses, we indicated that only the wooden plank would attain such a height conincident with the height of the missile proof siding. However, from a hypothetical viewpoint, should a missile be assumed to enter the spent fuel pool area through the missile proof siding, the following analysis is performed.

E. Energy Taken by Metal Siding

The formula used to determine the energy (ft-lb) taken by metal siding is given by Reference 5.D-7.

$$E = \frac{DS}{46500}\left(16,000t^2 + 1500\frac{W}{W_s}t\right) \qquad (7)$$

Where:

D = Diameter of missile (inches)

S = Yield strength of target (lb/in²)

t = Target thickness (inches)

W/W_s = Window factor = length of square side between rigid supports/length of a standard width (4 inches)

F. Striking Velocity of Missile at Water Surface

$$E_\gamma = E_i - E,\ V1 = (2E_r/m)^{1/2} \qquad (9)$$

Where:

$E\gamma$ = Residual energy of a missile

E_i = Initial energy of a missile

m = Mass of a missile

G. Impactive Energy on Fuel Rack

The method used to determine the impactive energy on fuel rack is given in Reference 5.D-8.

$$\frac{E_f}{W} = \frac{1}{\varepsilon\rho_w}\left\{\left(\frac{\varepsilon\rho_w V_1^2}{2g} - 1\right)e^{-\varepsilon\rho_w H} + 1\right\} \quad (10)$$

where:

E_F = Kinetic energy at fluid depth H

W = Weight (lb)

$$E = \frac{C_d A}{W}(ft^2/lb)$$

A = Projected area (ft^2)

ρ_w = Fluid density (lb/ft^3)

C_d = Drag coefficient

g = Gravity (ft/sec^2)

H = Fluid depth (feet)

The assumed missile, namely the 1 inch rod with impactive energy of 4600 ft-lb, is analyzed with various possible trajectories entering the spent fuel pool through the missile-proof siding as shown in Figures 5.D.A–1 and 5.D.A–2.

By substituting the following values into Equation 7,

D = 1 inches

S = 50,000 lb/in^2

t = 0.2092 inches

$W/W_s = 2$

then

$$E = \frac{1.075}{\cos\Theta}\left[\frac{700.234}{\cos^2\Theta} + \frac{627.6}{\cos\Theta}\right] \quad (11)$$

where θ = angle of entry as demonstrated in Figure 5.D.A–3.

The values of energy taken by the metal siding, E, and the residual energy of the missile, E_r, with corresponding angles of entry, θ, are given in Table 5.D.A–2A, from which the following summary is drawn:

(a) At 0° entry.

The assumed missile would have no direct impact on the fuel pool. Free falling of the assumed missile is less critical than the trajectories considered below.

(b) At 45° entry.

From Table 5.D.A–2A, the residual energy for the missile is equal to 1,120 ft-lb. The missile velocity at impacting the water surface as obtained by Equation 9 is calculated to be 95 fps. Assuming the missile impacts with its minimum cross sectional area on the pool and travels vertically downward, the kinetic energy per pound impactive at the fuel rack is calculated from Equation 10 to be

$$E \div W = 66$$

Therefore E = 66(8) = 528 ft-lb for the impactive area of the rod. The kinetic energy per impactive unit area thus obtained (528 ft-lb/0.785 in^2 = 672.6 ft-lb/in^2) is the same as the design capability of fuel rack. The fuel racks are designed to withstand a fuel assembly weighing 1,300 pounds dropped from approximately 26 feet (assuming no drag resistance from the water). The resulting impactive energy thus obtained is approximately 33,800 ft-lb which when applied over an eight inch diameter area atop of the fuel rack gives the same impactive energy per unit area as the rod.

(c) At angles greater than 45°.

The residual energy of the missile impacting the siding at angles greater than 45° is less than that discussed in Item b above. As indicated in Table 5.D.A–2A, the missile would not penetrate the missile proof siding at an angle of about 55° or greater.

It is therefore concluded that even if the missile could attain such a height as to enter the spent fuel area through the missile proof siding, no possible damage to the fuel element could occur.

APPENDIX 5D

ATTACHMENT A

TABLE 5.D.A–1A RELATIVE CONSERVATISM IN STEPS A, B, AND C

F.P.	Maximum Velocities (fps)			Maximum Heights (feet)		
	(a)	(b)	(c)	(a)	(b)	(c)
0.10	250	323	377	40.7	79.9	194.8
0.09	243	314	369	36.8	74.4	181.2
0.08	236	303	360	30.0	69.0	166.8
0.07	229	291	349	28.2	61.8	150.1
0.06	220	278	336	23.7	54.6	131.4
0.05	209	262	321	18.8	45.7	110.5
0.04	196	243	301	13.3	35.6	86.7
0.03	180	220	279	7.8	24.4	59.1
0.02	159	189	246	2.1	11.4	27.8
0.015	144	170	219	0.005	4.9	6.16

(a) Relative wind velocity.

(b) Absolute wind velocity. Missile on own path.

(c) Absolute wind velocity. Missile following windfield.

APPENDIX 5D

ATTACHMENT A

TABLE 5.D.A–2A

ANGLE OF ENTRY θ	RESISTANCE ENERGY BY SIDING, E (FT-LB)	RESIDUAL ENERGY, E_r (FT-LB)
0	1430	3170
20	1670	2930
30	2060	2540
45	3480	1120
50	4460	140
55	6030	-
60	8720	-

APPENDIX 5D

ATTACHMENT A

TABLE 5.D.A–3A

ANGLE OF ENTRY θ	RESISTANCE ENERGY BY SIDING, E (FT-LB)	RESIDUAL ENERGY, E_r (FT-LB)
0	1430	3170
20	1670	2930
30	2060	2540
45	3480	1120
50	4460	140
55	6030	-
60	8720	-

FIGURE 5.D.A–1 PLAN OF SPENT FUEL AREA

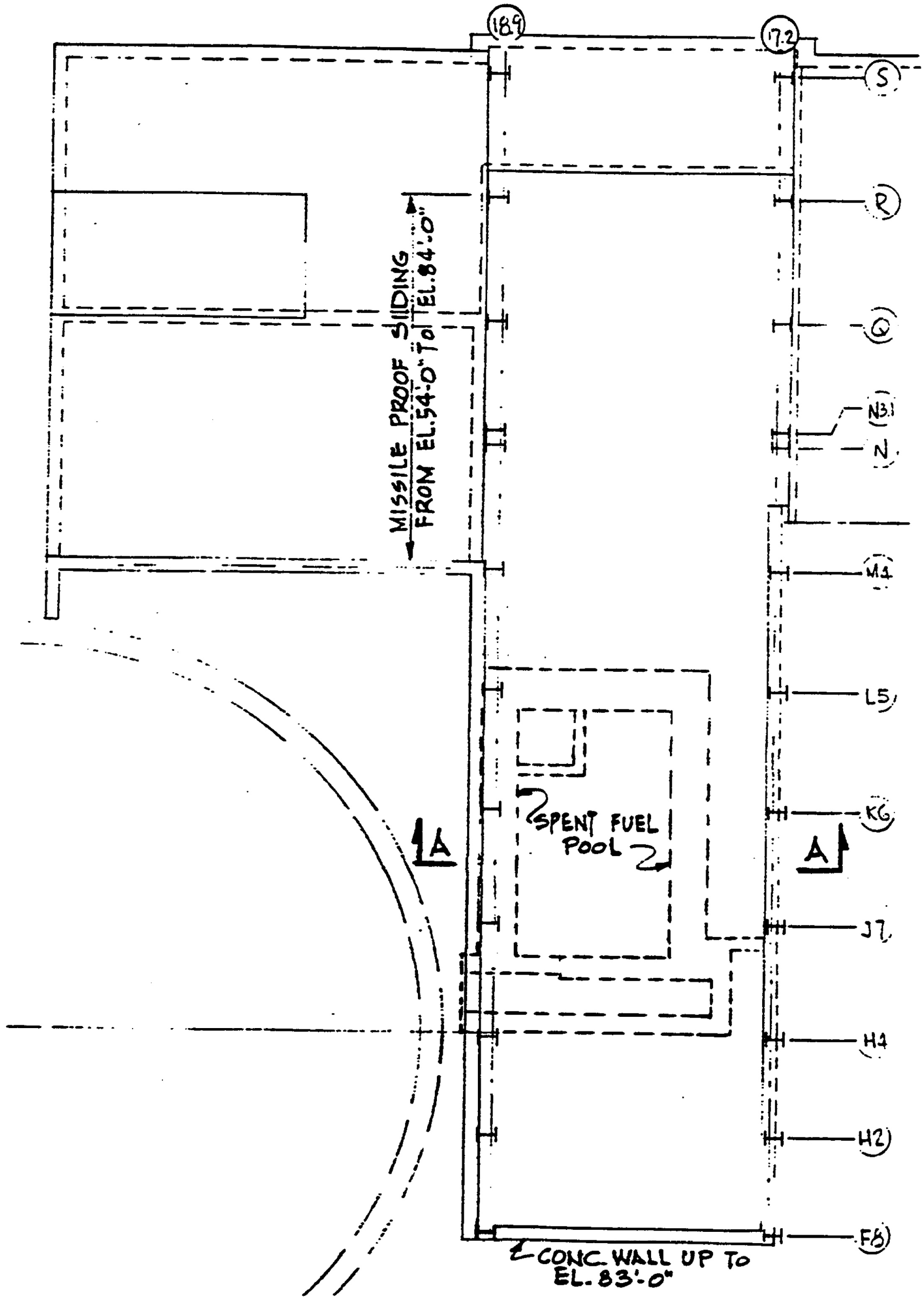


FIGURE 5.D.A–2 SPENT FUEL POOL SECTION VIEW

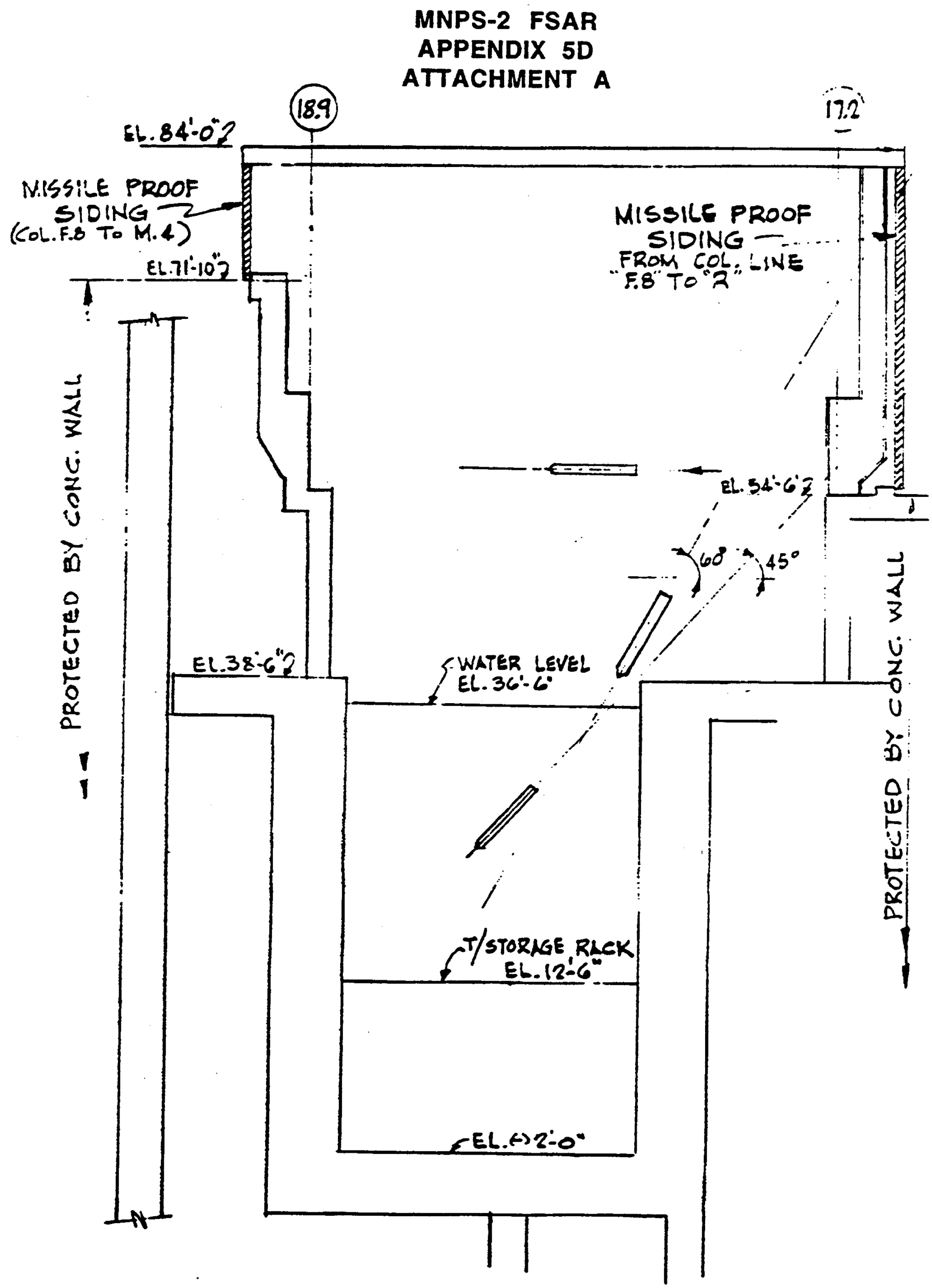
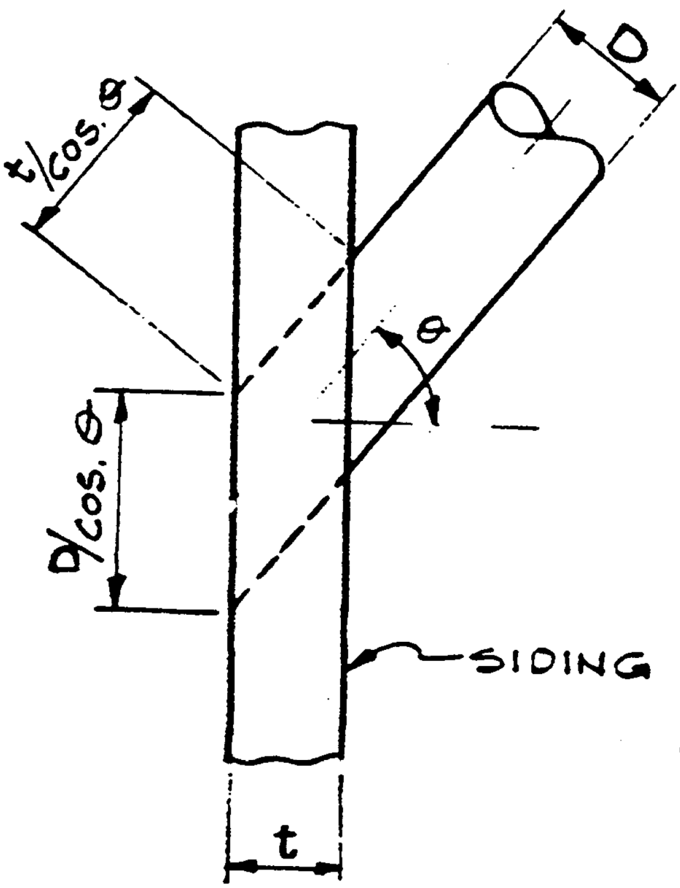


FIGURE 5.D.A–3 MISSILE STRIKE ANGLE TO MISSILE PROOF SIDING



5.E COMPUTER PROGRAM LIST AND DESCRIPTIONS

I. BLOWDOWN LOADS

The following codes provide the basis for the hydraulic forces acting on the structures during the subcooled and two phase periods of blowdown.

WATERHAMMER

a. Description

Fundamental to the determination of the mechanical loads during blowdown is the hydrodynamic solution of the fluid decompression. The major loads imposed on the reactor vessel internals occur during the subcooled period of decompression.

The digital computer program WATERHAMMER[1] is used to describe this subcooled response. The WATERHAMMER code is a digital computer program developed under contract to the AEC for use on the LOFT program. It is based on assumptions of one dimensionality and ignores fluid friction at the walls. These assumptions result in simplified forms of the equations of mass and momentum. These equations are transformed to their wave conjugate form and solved by the superposition method. Details of the method of solution are presented in Reference 1. The basic assumptions, which provide for convenient solution by the wave equations (Hookes' law treatment of fluid properties and neglect of conservation of energy) limit the extent of applicability of the solution to subcooled fluid conditions only.

b. Reference

1. 65-28-RA, "EARLY BLOWDOWN (WATERHAMMER) ANALYSIS FOR LOSS OF FLUID TEST FACILITY", by Stanislav Fabic, dated June 1965 and revised April 1967.

CEFLASH-4

a. Description

CEFLASH-4, is employed to provide information during the Two-phase transition period. CEFLASH-4 is a C-E modified version of the FLASH-4 [1] code. This code, as shown in References 2 and 3, has demonstrated applicability for the two-phase transient. It is a multinode-multiflowpath code which simultaneously solves the finite difference form of the equations of mass, momentum, and energy conservation in conjunction with tabularized values of the fluid properties. The numerical solution technique employs a backward difference integration scheme which leads to numerically stable results for the depressurization predictions.

The two codes described above provide the basis for determination of the hydraulic forces acting on the structures during the subcooled and two-phase periods of blowdown.

b. References

1. WAPD-TM-840, “FLASH-4: A FULLY IMPLICIT FORTRAN IV PROGRAM FOR THE DIGITAL SIMULATION OF TRANSIENTS IN A REACTOR PLANT”, by T. A. Porsching, J. H. Murphy, J. A. Redfield, and V. C. Davis, dated March 1969.

2. CENPD-26, Combustion Engineering Report, “Description of Loss-of-Coolant Calculational Procedures”, August 1971 (Proprietary).

3. CENPD-42, Combustion Engineering Report, “Topical Report on Dynamic Analysis of Reactor Vessel Internals Under Loss-of-Coolant Accident Conditions With Application of Analysis to CE 800 Mwe Class Reactors”, August 1972 (Proprietary).

II. REACTOR VESSEL INTERNALS

The following codes were used in the analysis of the reactor vessel internals.

MRI/STARDYNE

a. Description and Assumptions

The program uses the finite-element method for the static and dynamic analyses of two and three dimensional solid structures subjected to any arbitrary static or dynamic loading or base acceleration. In addition, initial displacements and velocities may be considered. The physical structure to be analyzed is modeled with finite elements which are interconnected by nodes. Each element is constrained to deform in accordance with an assumed displacement field that is required to satisfy continuity across element interfaces. The displacement shapes are evaluated at nodal points. The equations relating the nodal point displacements and their associated forces are called the element stiffness relations and are a function of the element geometry and its mechanical properties. The stiffness relations for an element are developed on the basis of the theorem of minimum potential energy. Masses and external forces are assigned to the nodes. The general solution procedure of the program is to formulate the total assemblage stiffness matrix [K] and apply it to either of the following equations:

$$[k]\ (\delta) = (p) \qquad (1)$$

$$W^2[m](q) - [k](q) = 0 \qquad (2)$$

where:

(δ) = the nodal displacement vector

(p) = the applied nodal forces

[m] = the max matrix

w = the natural frequencies

(q) = the normal modes

Equation (1) applies during a static analysis which yields the nodal displacements and finite elements internal forces. Equation (2) applies during an eigenvalue/eigenvector analysis, which yields the natural frequencies and normal modes of the structural system. Using the natural frequencies and normal modes together with related mass and stiffness characteristics of the structure, appropriate equations of motion may be evaluated to determine structural response to a prescribed dynamic load.

The finite element used to date in CE analyses is the elastic beam member. The assumptions governing its use are as follows: small deformation, linear-elastic behavior, plane sections remain plane, no coupling of axial, torque and bending, geometric and elastic properties constant along length of element.

b. Extent of Program's Applications

The program is used to obtain the response of the reactor vessel internals due to prescribed seismic excitation. The structural components are modeled with beam elements. The geometric and elastic properties of these elements are calculated such that they are dynamically equivalent to the original structures. The response analysis is then conducted using both model response spectra and modal time history techniques. Both methods are compatible with the program.

c. Reference

"MRI/STARDYNE-Static and Dynamic Structural Analysis System: User Information Model", Control Data Corporation, June 1, 1970.

ASHSD

a. Description and Assumptions

The program uses a finite-element technique for the dynamic analysis of complex axisymmetric structures subjected to any arbitrary static or dynamic loading or base acceleration. The three-dimensional axisymmetric continuum is represented either as an axisymmetric thin shell or as a solid of revolution, or as a combination of both. The axisymmetric shell is discreted as a series of frustums of cones and the solid revolution as triangular or quadrilateral "toroids" connected at their nodal circles.

Hamilton’s variational principle is used to derive the equations of motion for these discrete structures. This leads to a mass matrix, stiffness matrix and load vectors which are all consistent with the assumed displacement field. To minimize computer storage and execution time, the non-diagonal “consistent” mass matrix is diagonalized by adding off-diagonal terms to the appropriate diagonal terms. These equations of motion are solved numerically in time domain by a direct step-by-step integration procedure.

The assumptions governing the axisymmetric thin shell finite element representation of the structure are those consistent with linear orthotropic thin elastic shell theory.

b. Extent of Program’s Applications

ASHSD is used to obtain the dynamic response of the CSB/TS system due to a LOCA. An axisymmetric thin shell model of the structure is developed. The spatial Fourier series components of the time varying LOCA loads are applied to the modeled structure. The program yields the dynamic shell and beam mode response of the structural system.

c. Reference

Ghosh, S., Wilson, E. L., “Dynamic Stress Analysis of Axisymmetric Structures Under Arbitrary Loading”, Report Number EERC 69-10, U. of California, Berkeley, September, 1969.

ICES/STRUDL - II

a. Description and Assumptions

The ICES/STRUDL-II computer program provides the ability to specify characteristics of problems - framed structures and three-dimensional solid structures, perform analyses - static and dynamic, and reduce and combine results.

Analytic procedures in the pertinent portions of ICES/STRUDL-II apply to framed structures. Framed structures are two or three dimensional structures composed of slender, linear members which can be represented by properties along a centroidal axis. Such a structure is modeled with joints, including support joints, and members connecting the joints. A variety of force conditions on members or joints can be specified. The member stiffness matrix is computed from beam theory. The total stiffness matrix of the modeled structure is obtained by appropriately combining the individual member stiffnesses.

The stiffness analysis method of solution treats the joint displacements as unknowns. The solution procedure provides results for joints and members. Joint results include displacements and reactions and joint loads as calculated from member end forces. Member results are member end forces and distortions. The assumptions governing the beam element representation of the structure are as follows: linear, elastic, homogeneous, and isotropic behavior, small deformations, plane sections remain plane, and no coupling of axial, torque and bending.

b. Extent of Program’s Applications

The program is used to obtain stiffness properties of lower support structure and upper guide structure grid beams due to transverse loads. The results of the analyses are incorporated into overall reactor vessel internals’ models, which calculate the dynamic response due to seismic and LOCA conditions. These latter results, at a given time, are fed back into the grid beam models to yield dynamic stresses.

c. Reference

“ICES/STRUDL-II, The Structural Design Language: Engineering User's Manual, Volume I”, Structures Division and Civil Engineering’s Systems Laboratory, Department of Civil Engineering, MIT, Second Edition, June, 1970.

SHOCK

a. Description and Assumptions

The computer program SHOCK solves for the response of structures which can be represented by lumped-mass and spring systems and are subjected to a variety of arbitrary type loadings. This is done by numerically solving the differential equations of motion for an n^{th} degree of freedom system using the Runge-Kutta-Gill or a specialized Newmark integration technique. The equations of motion can represent an axially responding system or a laterally responding system; i.e., an axial motion, or a coupled lateral and rotational motion. The program is designed to handle a large number of options for describing load environments and includes such transient conditions as time-dependent forces and moments, initial displacements and rotations, and initial velocities. Options are also available for describing steady-state loads, preloads, accelerations, gaps, and nonlinear elements.

The output from the code consists of minimum and maximum values of translational and angular accelerations, forces, shears and moments for the problem time range. In addition, the above quantities are presented for all printout times requested. Plots can also be obtained for displacements, velocities and accelerations as desired.

b. Extent of Program’s Applications

The program is used to obtain the transient response of the reactor vessel internals due to LOCA loads. Lateral and vertical lumped-mass and spring models of the internals are formulated.

Various types of springs - linear, compression only, tension only, or nonlinear springs - are used to represent the structural components. Thus, judicious use of load-deflection characteristics enables effects of components impacting to be predicted. Transient loading appropriate to the horizontal and vertical directions is applied at mass points and a dynamic response - displacements and internals forces - is obtained.

c. Reference

Gabrielson, V. K., "SHOCK - A computer Code For Solving Lumped-Mass Dynamic Systems", SCL-DR-65-34, January, 1966.

SAMMSOR-DYNASOR

a. Description and Assumptions

SAMMSOR-DYNASOR provides the ability to perform nonlinear dynamic analyses of shell structures represented by axisymmetric finite-elements and subjected to arbitrarily varying load configurations.

The program employs the matrix displacement method of structural analysis utilizing a curved shell element. Geometrically nonlinear dynamic analyses can be conducted using this code.

Stiffness and mass matrices for shells of revolution are generated utilizing the SAMMSOR part of this code. This program accepts a description of the structure in terms of the coordinates and slopes of the nodes and the properties of the elements joining the nodes. Utilizing the element properties, the structural stiffness and mass matrices are generated for as many as twenty harmonics and stored on magnetic tape. The DYNASOR portion of the program utilizes the output tape generated by SAMMSOR as input data for the respective analyses.

The equations of motion of the shell are solved in DYNASOR using Houbolt's numerical procedure with the nonlinear terms being moved to the right-hand side of the equilibrium equations and treated as generalized pseudo-loads. The displacements and stress resultants can be determined for both symmetrical and asymmetrical loading conditions. Asymmetrical dynamic buckling can be investigated using this program. Solutions can be obtained for highly nonlinear problems utilizing as many as five circumferential Fourier harmonics.

b. Extent of Program's Application

This program is used to analyze the dynamic buckling characteristics of the core support barrel during a LOCA hot-leg break. The program's nonlinear characteristics provide this capability.

A finite element model of the CSB is formulated which is consistent with the computer program. Taking into account the initial deviation of the structure and the shell mode which is most likely to give the minimum critical pressure, the time-dependent pressure load is applied to the barrel. The maximum displacement occurring in the barrel is obtained. This result is used as a basis for an overload analysis to satisfy ASME Code requirements.

c. References

Tillerson, J. R. and Haisler, W. E., “SAMMSOR II - A Finite Element Program to Determine Stiffness and Mass Matrices of Shells of Revolution”, Texas A&M University, TEES-RPT-70-18, October, l970.

Tillerson, J. R. and Haisler, W. E., “DYNASOR II - A Finite Element Program for the Dynamic Non-Linear Analysis of Shells or Revolution”, Texas A&M University, TEES-RPT-70-19, October, 1970.

SAAS

a. Description and Assumptions

The program performs finite element static analyses of axisymmetric solids. The continuous body to be analyzed is replaced by a system of ring elements with triangular or quadrilateral cross sections. The elements are interconnected at their apexes which are referred to as nodes. The displacement method of finite element analysis is used to derive the element stiffness matrix. This method proceeds by selecting a displacement expansion over the element, consistent with elemental boundary conditions, and assuming displacements in the interior of the element depend only on nodal quantities. The elemental stiffness matrices are computed and combined to yield the total stiffness matrix of the modeled structure. The principle of minimum potential energy is then applied to yield displacements and elemental forces (stresses). Since these elements are of relative arbitrary shape, the procedure can be applied to bodies of complex geometry. The program performs static analyses due to both boundary forces and thermal loads by converting these effects into equivalent nodal quantities.

Assumptions governing the use of the aforementioned finite elements are those consistent with linear elasticity theory of solid structures.

b. Extent of Program’s Applications and Assumptions

The program is employed to determine the stiffness properties of flanged regions as related to axisymmetric loads. Specifically, the CSB upper and lower flanges with connecting cylinders and the UGSP flange with connecting cylinder were analyzed. Displacements due to a known external load were determined. The resulting stiffnesses were incorporated into an overall model of the reactor vessel internals, which were employed in determining the dynamic response during vertical seismic and LOCA excitation. The results of the dynamic analyses were fed back into the flange analyses to determine their maximum stresses and deformations.

Instrumentation flanges, which are used as primary pressure boundaries were analyzed with the subject program to determine their acceptability with respect to ASME Section III criteria.

c. Reference

Wilson, E. L., Jones, R. M., “Finite Element Stress Analysis of Orthotropic, Temperature-Dependent Axisymmetric Solids of Revolution”, Aerospace Report Number TR-0158 (S3816-22)-1, September, 1967.

NAOS

a. Description and Assumptions

The program applies the finite element analysis to axisymmetric solids subjected to arbitrary nonaxisymmetric loadings by expanding the various kinematic and forcing functions into Fourier series.

The continuous body to be analyzed is replaced by a system of ring elements with triangular or quadrilateral cross sections and/or thin conical shell elements. The elements are interconnected at their apexes and ends (for the case shell elements) which are referred to as nodes. The displacement method of finite element analysis is used to derive the element stiffness matrices. This method proceeds by selecting a displacement expansion over the elements, consistent with elemental boundary conditions and assuming displacements in the interior of the element depend only on nodal quantities. The elemental stiffness matrices are computed and then combined to yield the total stiffness matrix of the modeled structure. The principle of minimum potential energy is then applied to obtain displacements and element forces (stresses). The program performs static analyses due to both boundary forces and thermal loads by converting these effects into equivalent nodal quantities. Since the elements employed are of relatively arbitrary shape, the procedure can be applied to bodies of complex geometry.

Assumptions governing the aforementioned analyses are those consistent with linear elasticity theory of solids and thin shell structures.

b. Extent of Program’s Applications

The program is employed to determine the stiffness properties of flanged regions due to lateral loads. Specifically, the CSB upper and lower flanges with connecting cylinders and the UGSP flange with connecting cylinders were analyzed for lateral shear and bending moment loads. Displacements due to known magnitudes of these loads were determined. The resulting stiffnesses were incorporated into overall models of the reactor vessel internals which were employed in determining the dynamic response during horizontal seismic and LOCA excitation.

The results of the dynamic analysis were fed back into the flange analyses to determine maximum stresses and deformations.

c. Reference

Dunham, R. S., Nickell, R. E., et. al., “NAOS - Finite Element Analysis of Axisymmetric Solids with Arbitrary Loadings”, Structural Engineering Laboratory, University of California, Berkeley, California, June, 1967.

EAC/EASE

a. Description and Assumptions

The EAC/EASE computer program provides static structural analyses of linear, three-dimensional systems subjected to sets of arbitrarily prescribed mechanical and thermal loads and displacement boundary conditions. The analysis used in the program is an application of the direct stiffness method.

As the first step, the actual system is approximated by an assemblange of discrete structural elements interconnected at a finite number of points called nodes. The behavior of the “discretized structure” is an approximation to the response of the real system.

Next, each element is constrained to deform in accordance with an assumed displacement field that is required to satisfy continuity across element interfaces. The displacement shapes are evaluated at convenient locations within the element (usually at points on element boundaries), and their amplitudes are sometimes called “generalized coordinates”.

The equations relating generalized coordinates and their associated forces are called the element stiffness relations and are a function of the element geometry and its mechanical properties. The stiffness relations for an element are developed on the basis of a governing variational principle, the theorem of minimum potential energy.

The “complete stiffness” of the discretized structure is obtained in the following manner: The element equations are combined into the system equations using the requirement that the summation of all internal forces contributed from those elements common to a particular node must be equal to the externally applied load at that joint. The resulting set of equations (appropriately modified for displacement boundary conditions) represent the equilibrium equations for the discretized structure connected only at the joints.

The solution of the equilibrium equations results in six displacement components at each node arising from the loads applied to the system. Having defined the displacements at all nodes in the system, the internal stresses are calculated from displacements for each element in the structure.

The elements used to model structures are the triangular plate membrane and bending elements. The stiffness relations for the elements are developed according to the following assumptions: small deformations, linear-elastic isotropic behavior, uniform element thickness, negligible through-the-thickness stress, normals to the original mid-surface do not distort and remain normal to the deflected mid-surface (applies to plate bending element).

b. Extent of Program's Application

The program is used to perform thermal stress analysis of the core shroud. A symmetrical section of the core shroud is modeled with triangular plate membrane and bending elements. The thermal load is applied by specifying temperatures at each of the nodal points. The results of the analysis are compared to stress criteria defined in Section III of the ASME Boiler and Pressure Vessel Code.

c. Reference

"EAC/EASE - Elastic Analysis for Structural Engineering: User's Information Manual", Control Data Corporation, April, 1971.

III. REACTOR COOLANT SYSTEM

The computer programs that were used in the dynamic seismic analysis of the reactor coolant system components, as discussed in Section 4.A.2.3, Appendix 4.A, of the FSAR, include:

ICES/STRUDL-11

a. Description and Assumptions

The ICES/STRUDL-II computer program provides the ability to specify characteristics of problems - framed structures and three-dimensional solid structures, perform analyses - static and dynamic, and reduce and combine results.

Analytic procedures in the pertinent portions of ICES/STRUDL-II apply to framed structures. Framed structures are two or three dimensional structures composed of linear members which can be represented by properties along a centroidal axis. Such a structure is modeled with joints, including support joints, and members connecting the joints. A variety of force conditions on members or joints can be specified. The member stiffness matrix of the modeled structure is obtained by appropriately combining the individual member stiffnesses.

Masses may be specified for selected joint degrees-of-freedom represented in the total stiffness matrix. The total stiffness matrix is then "condensed" to yield a dynamic stiffness matrix in which only those joint degree-of-freedom for which mass is specified are retained. Using the (condensed) dynamic stiffness matrix and the associated diagonal mass matrix, an eigenvalue solution is performed by a diagonalization process (Jocobi's method) to yield the natural frequencies and mode shapes corresponding to the free vibrations of the structure.

Using the total stiffness matrix, the program will also solve the general statics problem to yield influence coefficients which relate member end forces and moments and support reactions to unit displacement imposed, in turn, at each of a designated group of joint degrees-of-freedom. In addition, the program will condense the total stiffness matrix to yield a set of stiffness coefficients which relate the forces corresponding to selected degrees of freedom of one group of designated joints to imposed displacements corresponding to selected degrees-of-freedom of a second group of designated joints.

The stiffness analysis method of solution used in STRUDL treats the joint displacements as unknowns. The solution procedure provides results for joints and members. Joint results include displacements and reactions and joint loads as calculated from member end forces. Member results are member end loads and distortions. The assumptions governing the beam element representation of the structure are as follows: linear, elastic, homogeneous, and isotropic behavior, small deformations, plane sections remain plane, and no coupling of axial, torque and bending.

b. Extent of Program's Application

The program is used to define the dynamic characteristics of the structural models used in the dynamic seismic analyses of the reactor coolant system components. The natural frequencies and mode shapes of the structural models and the influence coefficients which relate member end forces and moments and support reactions to unit displacements are calculated. The influence coefficients are calculated for each dynamic degree-of-freedom of each mass point and for each degree-of-freedom of each support point at which relative motion is imposed. In addition, stiffness coefficients are calculated which relate the forces corresponding to those joint degree-of-freedom for which mass is specified to the imposed displacements corresponding to those (support) joint degrees-of-freedom at which relative motion will be specified during subsequent seismic response calculations. As appropriate, these data are stored for later use in response spectra or time-history seismic response calculations (see Appendix 4.A of the FSAR).

c. Reference

"ICES/STRUDL-II, The Structural Design Language: Engineering User's Manual, Volume I", Structures Division and Civil Engineering's Systems Laboratory, Department of Civil Engineering, MIT, Second Edition, June, 1970.

TMCALC

a. Description and Assumptions

Using normal mode theory and Newmark's Beta-Method, with Beta equal to 1/6, the C-E computer program TMCALC solves the differential equations of motion which represents the dynamic characteristics of a singly or multiply supported, multi-degree-of-freedom linear structural system subjected to seismic excitations. The program provides for separate, time-dependent, inputs at each support point at which relative motion produced by the seismic event may be imposed. In the step-by-step numerical integration process (Newmark's Beta-Method) employed by TMCALC, the time step selected is less than one-tenth of the period of the highest frequency mode.

Inputs to TMCALC consist of:

1. Output from STRUDL:

   a. frequencies,

   b. mode shapes,

   c. stiffness coefficients which relate mass point degree-of-freedom forces to support point degree-of-freedom (relative) displacements.

2. Digitized time histories which describe the seismic event in terms of time-dependent motion imposed at the support points of the structural system. These consist of time histories of absolute accelerations at the reference support point of the system and corresponding time histories of relative displacements at all nonreference support points at which relative motion is imposed (see Appendix 4.A of the FSAR).

   The output from TMCALC consists of complete time histories of absolute accelerations, relative velocities and relative displacements corresponding to each dynamic degree-of-freedom of each of the mass points of the structural system. These data are calculated at each point in time during the integration process for the entire duration of the seismic event.

   The formulation of the program assumes linear elastic behavior of the structure and a linear variation in accelerations over an integration time step.

b. Extent of Program's Application

The program is used to calculate the dynamic response of structural models used in the dynamic seismic analysis of the reactor coolant system components. These data include time histories of absolute accelerations, relative velocities and relative displacements corresponding to each dynamic degree-of- freedom of the structural system. The data are stored for use in subsequent seismic response calculations.

c. References

1. Przemieniecki, J. S., "Theory of Matrix Structural Analysis", Chapter 13, McGraw-Hill Book Company, New York, New York, 1968.

2. Hurty, W. C., and Rubinstein, M. F., "Dynamics of Structures", Chapter 8, Prentice Hall, Inc., Englewood Cliffs, New Jersey, 1964.

3. Newmark, N. M., "A Method of Computation for Structural Dynamics", Volume 3, Journal of Engineering Mechanics Division, A.S.C.E., July, 1959.

FORCE

a. Description and Assumptions

The formulation of the computer program FORCE assumes a linear elastic structural system modeled as a three-dimensional assemblage of joints, or modes, which are interconnected by elastic structural elements or members. The program calculates a discrete time history of the loads, forces and moments, at each designated member end and the reactions at each designated support joint, induced by specified seismic conditions (see Section 4.A.2.3.4, Appendix 4.A of the FSAR). The program selects the maximum absolute values of each component of load at each member end and at each support joint, and the times at which the maximum values occur, over the entire duration of the specified seismic event.

As input, the program FORCE uses a matrix of influence coefficients calculated by ICES/STRUDL-II, the dynamic response of the structure, i.e., the time history of relative displacements corresponding to each mass joint degree-of-freedom, as calculated by the program TMCALC, and the time histories of relative displacements prescribed for each support joint degree-of-freedom for the seismic event under consideration. With this input, the program FORCE forms appropriate linear combinations of the time-dependent relative displacements to yield a complete loads analysis of the deformed shape of the structure at each point in time over the entire duration of the seismic event.

b. Extent of Program’s Application

The program is used to calculate the time-dependent response of the reactor coolant system components to specified seismic conditions.

c. References

Przemieniecki, J. S., “Theory of Matrix Structural Analysis”, McGraw-Hill Book Company, New York, New York, l968.

SHAKE

a. Description and Assumptions

The computer program SHAKE performs a normal mode response spectrum analysis of a three-dimensional linear elastic structural system modeled as an assemblage of joints, or modes, which are interconnected by elastic structural elements, or members. In the formulation, mass is assumed “lumped” at selected joints, each of which may have up to three translational dynamic degrees-of-freedom.

Input to SHAKE consists of frequencies and mode shapes, corresponding to each normal mode of vibration of the structure, the relevant diagonal mass matrix, and the response spectrum value, acceleration, corresponding to the period of each normal mode. The output from SHAKE consists of the modal inertial loads, forces, corresponding to each mass joint dynamic degree-of-freedom of the structure, for each normal mode.

b. Extent of Program's Application

The program SHAKE is used to calculate the dynamic response, modal inertial loads, for the reactor coolant system components in those cases where spectrum analysis is applied (see Appendix 4.A of the FSAR). In turn, the modal inertial loads are applied to the structure, mode-by-mode, using the ICES/STRUDL-II program, which calculates the member end loads and support joint reactions for each mode, and combines the modal values to give the total response to the specified seismic event.

c. References

Hurty, W. C., and Rubinstein, M. F., "Dynamics of Structures", Prentice Hall, Inc., Englewood Cliffs, New Jersey, 1964.

IV. REACTOR COOLANT SYSTEM - STATIC STRUCTURAL ANALYSIS

MEC-21

a. Description and Assumptions

The program is designed to compute the reactions and stresses in complex piping systems, including closed-loop configurations, due to thermal expansion and contraction; pressure effects; concentrated loads such as valves, fittings and fixtures; and uniform loads such as weight. The computational method is a tensor analysis treatment of Castigliano's theorem and assumes linear elastic behavior.

b. Extent of Program's Application

The program is used extensively to calculate the loads which act on the components and supports of the reactor coolant system for the various operating conditions. The calculated loads are included in the equipment specifications and, subsequently, used in the design calculations performed for the individual components.

c. References

1. J. A. Olson and R. V. Cramer, "Pipe Flexibility Analysis Program MEL21", Report Number 10-66, Rev. 1, "A Modification of Program MEC21S", San Francisco Bay Naval Shipyard, Mare Island Site, Vallejo, California, August 26, 1969.

2. J. A. Olson and R. V. Cramer, "Pipe Flexibility Analysis Program MEC21S", Report Number 35-65, San Francisco Bay Naval Shipyard, Vallejo, California, November 22, 1965.

3. James Griffin, MEC21 7094, “A Piping Flexibility Analysis Program for the IBM-7090 and 7094”, Los Alamos Scientific Laboratory, Report LA-2929, Los Alamos, New Mexico, July 14, 1964.

V. <u>ASME SECTION III, CLASS I COMPONENTS</u>

The following programs were used to determine stresses in ASME Section III, Class I components.

SAAS

a. Description and Assumptions

The program performs finite element static analyses of axisymmetric solids. The continuous body to be analyzed is replaced by a system of ring elements with triangular or quadrilateral cross sections. The elements are interconnected at the apexes which are referred to as nodes. The displacement method of finite element analysis is used to derive the element stiffness matrix. This method proceeds by selecting a displacement expansion over the element, consistent with elemental boundary conditions, and assuming displacements in the interior of the element depend only on nodal quantities. The elemental stiffness matrices are computed and combined to yield the total stiffness matrix of the modeled structure. The principle of minimum potential energy is then applied to yield displacements and elemental forces (stresses). Since these elements are of relative arbitrary shape, the procedure can be applied to bodies of complex geometry. The program performs static analyses due to both boundary forces and thermal loads by converting these effects into equivalent nodal quantities.

Assumptions governing the use of the aforementioned finite elements are those consistent with linear elasticity theory of solid structures.

b. Extent of Program’s Application

The program is used to determine stresses in the primary head-tube sheet-secondary shell regions of the steam generators.

c. References

Wilson, E. L., Jones, R. M., “Finite Element Stress Analysis of Orthotropic, Temperature-Dependent Axisymmetric Solids of Revolution”, Aerospace Report Number TR-0158 (S3816-22)-1, September, 1967.

SEAL SHELL - 2

a. Description and Assumptions

The program uses the stiffness matrix method to solve the equations of thin elastic shells of revolution. The formulation assumes homogeneous, isotropic, elastic material properties. Thickness, radii of curvature, applied surface loads, temperature and material properties may vary along the generating curve of the shell, but not around its circumference. Circumferential uniformly distributed, line-loads and moments can be applied. In the formulation, the stiffness matrix is calculated from strain energy considerations by use of the principal of virtual work. Thick shell effects are included by using known results from the theory of beams to determine the appropriate contribution to the strain energy from the normal stress and shear deflections.

b. Extent of Program's Applications

The program is used to determine stresses and deformations in various axially-symmetric regions of Class I components.

c. References

"Seal-Shell-2-A Computer Program for the Stress Analysis of a Thick Shell of Revolution with Axisymmetric Pressures, Temperatures, and Distributed Loads", by C. M. Friedrich, WAPD-TM-398, UC-38: Engineering and Equipment, TID-4500, 24th Edition.

ANALYSIS OF AXISYMMETRIC SOLIDS

a. Description and Assumptions

The finite element method is applied to the determination of stresses and displacements in axisymmetric solids of arbitrary geometry subjected to thermal and mechanical loadings. The formulation is based upon energy principals and assumes linear elastic, isotropic materials.

b. Extent of Program's Application

The program is applied in the analysis of the regions of reactor vessel-to-vessel head bolted closure, and other axisymmetric regions of Class I components.

c. References

E. L. Wilson, "Analysis of Axisymmetric Solids", University of California, February, 1967.

WIN - 12100

a. Description and Assumptions

The CE computer program WIN–12100 is a general purpose thermal analysis program which determines transient and steady-state temperature distributions in physical systems.

The physical system, or structures analyzed may have irregular geometries, may be composed of several different materials and may be subjected to varying, time dependent, temperature transients at the boundaries. The formulation of the program is based upon the general finite difference equation of heat flow which is solved by relaxation techniques. Provisions are included to accommodate radiation, convection and conduction modes of heat transfer and to include internal heat generation.

b. Extent of Program's Application

The program is used extensively to determine the temperature distributions which are subsequently used in stress analysis of Class I components.

c. References

Computer Program Number WIN - 12100, “Heat Transfer by Relaxation”, T. R. McCormack, Combustion Engineering, Inc., Windsor, Connecticut, 1968.

Stanley Hellman, George Habetler and Harold Babrov, “Use of Numerical Analysis in Transient Solution of Two-Dimensional Heat Transfer Problems with Natural and Forced Convection”, ASME Transactions, 1956.

TAP

a. Description and Assumptions

The program uses finite element techniques in obtaining numerical solutions to the general, three-dimensional, time-dependent differential equation of heat conduction. The formulation is general with respect to geometry, material properties and boundary conditions, permitting the solution of complex transient and steady-state heat transfer problems.

b. Extent of Program’s Application

The program is used to obtain temperature distributions in the reactor coolant pump case-cover assembly for subsequent use in stress analysis.

c. References

Peterson, F. E., Hui, H., “Three Dimensional Transient Heat Transfer Using a Finite Element Procedure”, Theoretical Basis and Sample Solutions, Engineering/Analysis Corporation, 1611 South Pacific Coast Highway, Redondo Beach, California, August, 1971.

SOLIDS II

a. Description and Assumptions

The program is a finite element formulation for the analysis of axisymmetric and plane solids with variable orthotropic, temperature-dependent material properties. The program accommodates both axisymmetric and non-axisymmetric loadings. The formulation assumes linear elastic behavior of materials.

b. Extent of Program's Application

The program is used to determine both thermal and mechanical stresses in the axisymmetric regions of the reactor coolant pumps.

c. References

Crose, J. C. and Jones, R. M. "Finite Element Stress Analysis of Axisymmetric and Plane Solids with Different Orthotropic, Temperature-Dependent Material Properties in Tension and Compression", TR-0059(56816-53)-1, The Aerospace Corporation, San Bernardino, California, June 1971.

BJS

a. Description and Assumptions

The program is a general purpose, three-dimensional, finite element formulation for the analysis of elastic structures of arbitrary shape sustaining arbitrary loadings and temperature distributions. The program BJS was adapted from the program SAP (September 1970 version) by the Engineering/Analysis Corporation, 1611 South Pacific Coast Highway, Redondo Beach, California for the Byron Jackson Pump Division, Borg-Warner Corporation, P. O. Box 2017, Terminal Annex, Los Angeles, California. The program assumes linear elastic behavior of materials and runs on the CDC-6600 Computer System.

b. Extent of Program's Application

The program is used for stress analysis of the reactor coolant pump casings.

c. References

"BJS A First Generation Three Dimensional Finite Element Computer Program for the Analysis of Structures of Arbitrary Geometry", Byron Jackson Pump Division, Borg-Warner Corporation, P. O. Box 2017, Terminal Annex, Los Angeles, California.

Wilson, E. L., "SAP A General Structural Analysis Program", Structural Engineering Laboratory, University of California, Berkley, California (September 1970).

VI. CLASS I PIPING SYSTEM

The following codes are used in the analysis of the Class I piping systems.

MRI/STARDYNE

a. Description and Assumptions

Refer to Section II, Reactor Vessel Internals, of this question.

b. Extent of Program's Applications

The program is used to obtain the responses of Class I piping system using three-dimensional model. The model response spectrum technique is used to compute forces, moments and displacements at each point specified in the piping system.

ADLPIPE

a. Description and Assumption

The program provides an elastic analysis of redundant piping systems subjected to thermal, static and dynamic loads. The system may contain a number of sections, a section being defined as a sequence of straight and/or curved members lying between two points.

The basic approach to be used in computing the response of piping systems to ground spectra consists of generating the dynamic properties of the system and applying the modal super-position method or normal mode method to obtain the structural response.

The two eigenvalve routines used in ADLPIPE are the Jocobi rotation scheme and the Givens-Householder scheme; the later has been modified to incorporate a suggestion made by Wilkinson.

b. Extent of Program's Application

The program is used to obtain the modal responses of Class I piping by the response spectrum technique.

c. References

1. Greenstadt, J., "The Determination Characteristic Roots of a Matrix by the Jocobi Method", John Wiley, New York, 1959.

2. Wilkinson, "The Algebraic Eigenvalve Problem".

NUPPIE II

a. Description and Assumptions

The program provides an elastic of redundant piping systems subjected to thermal, static and dynamic loads. The system may contain a number of sections, a section being defined as a sequence of straight and/or curved members lying between two points.

The basic approach to be used in computing the response of piping systems to ground spectra consists of generating the dynamic properties of the system and applying the modal super-position method or normal mode method to obtain the structural response.

b. Extent of Program's Application

The program is used to structurally analyze Class I piping systems. It performs static analysis as well as obtains modal responses by the modal analysis response spectra method and dynamic response to time history loads using the time history modal superposition method.

VII. CLASS I STRUCTURES

The following codes were used in the dynamic analysis of the Class I Structures.

CE 309 STRESS

a. Description and Assumption

STRESS (Structural Engineering Systems Solver) is a programming system for the solution of structural engineering problems. It was originally developed at M.I.T. in 1964 and intended for IBM-7094 application. In 1968 Bechtel implemented the program into GE-600 Series. The program is capable of executing a large variety of structural problems in two or three dimensional structures with different joint conditions. The program is primarily used to obtain the stiffness characteristics of a structure. Stiffness matrix of the structure obtained is used in the dynamic analysis.

b. Extent of Program's Applications

The program is used to obtain the stiffness matrix of the containment structure and containment internal structures.

c. Reference

Fenves, S. J., Logcher, R. D., and Mauch, S. P., "STRESS Reference Manual", The M.I.T. Press, Cambridge, Massachusetts, 1964.

CE 611 TIME-HISTORY ANALYSIS

a. Description and Assumptions

This program performs the time history analysis of a structure subjected to an earthquake motion. The analytical technique is based on modal synthesis. The general solution of the program is to formulate the equation of motion of the structure in terms of its mode shapes, frequencies and mass distribution as follows:

$$q_j + 2B_jw_jq_j + W_j^2q_j = -F_jU_g \ (j=1,2....N)$$

where:

N = member of modes

q = generalized coordinates

B = modal damping

w = frequency

$F_j = \frac{[\varnothing]T}{M_j}[M]$

[Ø] = mode shape matrix

[M] = mass matrix

M = mass

U_g = input acceleration

The above equation is solved using Runge-Kutta method.

b. Extent of Program's Application

The program is used to generate acceleration time history at all Class I equipment locations in the containment, auxiliary building (including warehouse portion), turbine building and intake structure.

c. References

Hildebrand, F. B., "Introduction to Numerical Analysis," McGraw-Hill Book Company, 1956.

Kuo, S. S., "Numerical Methods and Computers," Addition Wesley Publishing Company, 1965.

Hurty, W. C., Rubinstein, M. F., "Dynamics of Structures," Prentice Hall, Inc., 1964.

Biggs, J. M., "Introduction to Structural Dynamics," McGraw-Hill Book Company, 1964.

CE 617 MODES AND FREQUENCIES EXTRACTION

a. Description and Assumptions

The program provides a means for obtaining the natural frequencies, w, and modes shapes, Ø, of structural models. The structural input consists of the models lumped masses and either the stiffness or flexibility matrix. If the flexibility matrix is entered, the program provides an option for automatic inversion to a stiffness matrix.

The program uses the method of diagonalization by successive rotations. A detailed description of the method can be found in Engineering Analyses, A Survey of Numerical Procedures, by S. Crandall, published by the McGraw Hill Book Company, New York, 1966.

The modes shapes can be printed out on an optional basis for normalization such that either $[Ø]^T$ [m\] [Ø] = [1] or with respect to a unit relative deflection of some arbitrary point. It is recommended to use the first scheme for complex structures where it is not known which mass point will show the largest activity. Otherwise the second scheme may result in the selection of a nodal point with virtually no activity. For purposes of plotting the modes and to provide a quick inspection of modal behavior, a third scheme is automatically provided to normalize each mode with respect to its largest relative value. These modes should not be further utilized.

As a last step an orthogonality check is provided by forming the product $[Ø]^T$ [m\] [Ø]. The resulting product should show the off diagonal terms virtually zero in comparison to the main diagonal. This automatic check should always be reviewed.

b. Extent of Program's Applications

The program is used to obtain the mode shapes and frequencies containment structure and containment internal structures.

c. Reference

S. Crandall, S., "Engineering Analyses, A Survey of Numerical Procedures," McGraw Hill, 1966.

CE 641 RESPONSE SPECTRUM TECHNIQUE

a. Description and Assumption

The program assumes that the structure has been previously analyzed and that the natural frequencies, w, and mode shapes Ø, for the structure has been obtained. The earthquake input is described in terms of response spectrum curves associated with different damping values. Acceleration values are obtained from the curve for each mode corresponding to its natural frequency and damping value.

Fore each mode j, a modal inertial force at each mass point is calculated as follows:

$$F_{ij} = \frac{F_j \phi_{ij} M_i}{\sum^{\eta} \phi_{ij} M_i}$$

$$F_j = \frac{\left(\sum^{\eta} \phi_{ij} M_i\right)^2}{\sum^{\eta} \phi^2_{ij} M_i} A_j$$

where:

$$F_j = \frac{\left(\sum^{\eta} \phi_{ij} M_i\right)^2}{\sum^{\eta} \phi^2_{ij} M_i} A_j$$

A_j = special acceleration for mode j

Then static analysis is used to obtain modal shear, moment and displacement by applying the inertial force at each mass center. Model responses are combined on an absolute sum basis.

b. Extent of Program's Application

The program is used to obtain the model responses of the containment structure and containment internal structures by the response spectrum technique.

CE 784 RESPONSE SPECTRUM TECHNIQUE

a. Description and Assumption

This program combines the previously described programs, CE 309, CE 617 and CE 641, and calculates the dynamic responses of a structure. The program utilizes the mathematical model of the structure which is represented by its cross sectional properties and lumped masses. The program then forms the structure stiffness matrix. By using the modified Jocobi method of symmetric matrix diagonalization, natural frequencies and normal modes of the structure are obtained. Using the natural frequencies and normal modes together with input acceleration, modal responses of the structure are calculated.

b. Extent of Program's Applications

The program is used to obtain the modal responses of the auxiliary buildings (including warehouse portion) turbine building and intake building.

c. References

1. Gere and Weaver, "Analysis of Framed Structures," Van Nostrand, 1968.

2. Weaver, W. J., "Computer Programs for Structural Analysis," Van Nostrand, 1967.

3. Biggs, J. M. "Introduction to Structural Dynamics," McGraw Hill, 1964.

4. Ralston, A., Wilf, H. S., "Mathematical Methods for Digital Computers," John Wiley & Sons, 1962.

CE 792 RESPONSE SPECTRUM CALCULATION

a. Description and Assumptions

This program computes the response spectra for specified acceleration time histories which are generated by CE 611 Time History Analysis. The input acceleration time histories is digitalized at equal time intervals. The numerical method used for integration is based on the exact solution to the governing differential equation, assuming that the input acceleration time history varies linearly between consecutive data points.

b. Extent of Program's Applications

This program is used to generate response spectra at all equipment locations in the containment, auxiliary building (including warehouse portion), turbine building and intake structure.

c. References

Nigam, N. C., Jennings, P.C., "Digital Calculation of Response Spectra from Strong-Motion Earthquake Records," CIT, 1968.

VIII. <u>DELETED</u>

IX. <u>CLASS I STRUCTURES</u>

CE 316-4 Finite Element Stress Analysis (FINEL)

a. Description and Assumptions

This program does static analysis of plane or axisymmetric structures using the finite element method. The finite element library contains orthotropic quadrilateral reinforcement elements, and isotropic triangles and quadrilaterals. Element stresses and joint displacements are solved due to applied loads or temperature distributions. Applied loads can be concentrated, distributed or inertial and must be axisymmetric for axisymmetric structures. The total load can be applied in small increments and the solution is iterated within each increment if necessary to establish equilibrium. The program will consider up to eight different materials and allows for material properties changing with temperature. Materials can have bilinear stress-strain curves to model elasto-plastic behavior. Prestress forces are simulated by using appropriate concentrated forces.

b. Extent of Program's Applications

The program is used to obtain stresses in the containment structure due to thermal and pressure loads.

CE 779 Structural Analysis Program (SAP)

a. Description and Assumptions

This program performs the static and dynamic analysis of linear elastic three-dimensional structures using the finite element method. The finite element library contains truss and beam elements, plane and solid elements, plate and shell elements, axisymmetric (torus) elements, and special boundary (spring) elements.

Element stresses and displacements are solved due to either applied loads or temperature distributions. Concentrated loads, pressures or gravity loads can be applied. Temperature distributions are assigned as an appropriate uniform temperature change in each element. Prestressing can be simulated by using artificial temperature change on rod elements.

The available dynamic response routines will solve for arbitrary dynamic loads or seismic excitations using either modal superposition or direct integration. The program also does response spectrum analysis.

b. Extent of Program's Applications

The program is used to obtain stresses in the concrete shell which is designed to protect the condensate storage tank from missiles.

5.E.1 COMPUTER PROGRAM APPLICABILITY AND VALIDATION

I. BLOWDOWN LOADS

The computer programs WATERHAMMER and CEFLASH-4, described in Appendix 5.E were derived from programs in the public domain. Changes have been made to each to increase its utility and improve its treatment of the blowdown problem.[1, 2]

The WATERHAMMER [3] code is recognized for its applicability to the analysis of the subcooled decompression. The code manual (Reference 3) demonstrates the program's validity through comparison of its predictions to LOFT Semiscale experimental results.

CEFLASH-4 is the C-E modified version of the FLASH-4 code [4]. The C-E modifications are discussed in Reference 1. CEFLASH-4 has been accepted by the AEC via the Interim Acceptance Criteria of December 1971 [5].

The FLASH-4 program was written in FORTRAN IV for use on the CDC-6600 computer. It has been converted, at C-E such that it may also be run on the CDC-7600 computer.

Verification of this conversion was obtained by running test cases when this change was made.

a. References

1. CENPD-26, “Description of Loss-of-Coolant Calculational Procedures,” August 1971 (Proprietary).

2. CENPD-42, “Topical Report on Dynamic Analysis of Reactor Vessel Accident Conditions With Application of Analysis to CE 800 MWe Class Reactors,” August 1971 (Proprietary).

3. 65-28-RA, “Early Blowdown (WATERHAMMER) Analysis For Loss-of-Fluid Test Facility,” by Stanislav Fabic, dated June 1965 and revised April 1967.

4. WAPD-TM-840, “FLASH-4: A Fully Implicit FORTRAN IV Program for the Digital Simulation of Transient In a Reactor Plant,” by T. A. Porsching, J. H. Murphy, J. A. Redfield, and V. C. Davis, March 1969.

5. “Criteria for Emergency Core Cooling Systems for Light-Water Power Reactors,” Federal Register, pp 24083, Volume 36, Number 244, Saturday, December 18, 1971.

II. REACTOR VESSEL INTERNALS

The following programs are in the public domain and have had sufficient use to justify their applicability and validity.

MRI/STARDYNE June 1, 1970 Version

The program was developed by Mechanics Research, Inc., for use on the CDC-6600 Computer System. The program is run on the CDC-6600 Computer System located at the Boston, Massachusetts data center.

ICES/STRUDL Version 1.4

This version of the program was developed by the McDonnell Automation Company/Engineering Computer International. This version was purchased by Combustion Engineering and is run on the IBM-360 computer system.

EAC/EASE March 1970 Version

The program was developed by Engineering/Analysis Corporation for use on the CDC-6600 Computer System. The program is run on the CDC-6600 Computer System located at the Boston, Massachusetts data center.

SAAS Version I

The program was developed by E. L. Wilson and R. M. Jones and is documented in the following reference:

> Wilson, E. L., Jones, R. M., "Finite Element Stress Analysis of Orthotropic, Temperature-Dependent Axisymmetric Solids of Revolution," Aerospace Report Number TR-0158 (S3816022)-1, September 1967.

The theoretical basis of the program was developed in the following reference:

> Wilson, E. L., "Structural Analysis of Axisymmetric Solids," AIAA Journal, Volume 3, Number 12, December, 1965, pp 2269-2274.

The program is compatible with the CDC-6600 Computer System. It is run on the CDC-6600 Computer System located at Combustion Engineering's Windsor, Connecticut data center.

III. REACTOR COOLANT SYSTEM - DYNAMIC ANALYSIS

The following program is in the public domain and has had sufficient use to justify its applicability and validity.

ICES/STRUDL Version 1.4

This version of the program was developed by the McDonnell Automation Company/Engineering Computer International. This version was purchased by Combustion Engineering and is run on the IBM-360 computer system. (See, also, Appendix 5.E.3)

IV. REACTOR COOLANT SYSTEM - STATIC STRUCTURAL ANALYSIS

MEC-21 June 1970 Version

The program was developed by the Mare Island Naval Shipyard and is run on the CDC 6600/7600 computers.

V. ASME SECTION III, CLASS I COMPONENTS

The following programs are in the public domain and have had sufficient use to justify their applicability and validity.

SAAS Version I

The program was developed by E. L. Wilson and R. M. Jones and is documented in the following reference:

> Wilson, E. L., Jones, R. M., "Finite Element Stress Analysis of Orthotropic, Temperature-Dependent Axisymmetric Solids of Revolution," Aerospace Report Number TR-0158 (S3816-22)-1, September 1967.

The theoretical basis of the program was developed in the following reference:

> Wilson, E. L., "Structural Analysis of Axisymmetric Solids," AIAA Journal, Volume 3, Number 12, December 1965, pp 2269-2274.

The program is compatible with the CDC-6600 Computer System. It is run on the CDC-6600 Computer System located at Combustion Engineering's Windsor, Connecticut data center.

SEAL-SHELL-2

The program was developed by C. M. Friedrich and is documented in the following reference:

> "Seal-Shell-2-A Computer Program for the Stress Analysis of a Thick Shell of Revolution with Axisymmetric Pressures, Temperatures, and Distributed Loads," by C. M. Friedrich, WAPD-TM-398, UC-38: Engineering and Equipment, TID-4500, 24th Edition.

The program is run on the CDC-6600 Computer Systems, located at Combustion Engineering's Windsor, Connecticut data center.

Analysis of Axisymmetric Solids - 1967 Version

The program was developed by E. L. Wilson, University of California. The theoretical basis of the program is given in the following reference:

> Wilson, E. L., "Structural Analysis of Axisymmetric Solids," AIAA Journal, Volume 3, Number 12, December 1965, pp 2269-2274.

The program is run on the CDC-6600 Computer System located at Combustion Engineering's Windsor, Connecticut data center.

SOLIDS II - June 1971 Version

The program was developed by the Aerospace Corporation and runs on the CDC-6600 Computer System. The program is described in the following reference:

> Crose, J. C. and Jones, R. M., "Finite Element Stress Analysis of Axisymmetric and Plane Solids with Different Orthotropic, Temperature-Dependent Material Properties in Tension and Compression," TR-0059(56816-53)-1, The Aerospace Corporation San Bernardino, California (June 1971).

VI. CLASS I PIPING SYSTEMS

The following programs are in the public domain and have had sufficient use to justify the applicability and validity.

MRI/STARDYNE September 1, 1972 Version

The program was developed by Mechanics Research, Inc., for use on the CDC-6600 Computer System.

ADLPIPE

The program was developed by Arthur D. Little, Inc., Cambridge, Massachusetts for use on the CDC-6600 and UNIVAC 1108 Computer Systems.

VII. CLASS I STRUCTURES

None

5.E.2 COMPUTER PROGRAM TEST PROBLEM SOLUTIONS

I. BLOWDOWN LOADS

Additional demonstration of the WATERHAMMER's validity is given in CENPD-42 [1] which presents comparisons of the code's predictions to LOFT Semiscale and Battelle Northwest CSE experimental results.

CEFLASH-4 has also been tested against blowdown test data from the LOFT Semiscale and CSE experiments. The results of these comparisons are published in CENPD-26 and CENPD-42. These comparisons support the validity of the CEFLASH-4 method of analysis.

a. Reference

1. CENPD-42, "Topical Report on Dynamic Analysis of Reactor Vessel Accident Conditions With Application of Analysis to CE 800 MWe Class Reactors," August 1971 (Proprietary).

II. REACTOR VESSEL INTERNALS

The following programs' solutions to a series of test problems have been demonstrated to be substantially identical to those obtained by computer program, SABOR-5-DRASTIC, developed at the Aeronautics at the Massachusetts Institute of Technology:

ASHSD

SAMMSOR/DYNASOR

The comparison appears in "Topical Report on Dynamic Analysis of Reactor Vessel Internals under Loss-of-Coolant Accident Conditions with Application of Analysis to CE 800 MWe Class Reactors," Combustion Engineering Report CENPD-42, Combustion Engineering, Inc., Nuclear Power Department, Combustion Division, Windsor, Connecticut, Appendix D.

III. REACTOR COOLANT SYSTEM - DYNAMIC ANALYSIS

None

IV. REACTOR COOLANT SYSTEM - STATIC STRUCTURAL ANALYSIS

None

V. ASME SECTION III, CLASS I COMPONENTS

The following program's solutions to a series of test problems have been demonstrated to be substantially identical to those obtained by:

a. closed form solutions

b. EASE, a computer program developed by the Engineering/Analysis Corporation

c. SOLIDS II, TR-0059 (56816-53)-1, the Aerospace Corporation, San Bernardino, California

d. SAP, Structural Engineering Laboratory, University of California, Berkeley, California

BJS

The following program's solutions to a series of test problems have been demonstrated to be substantially identical to those obtained by computer program MARC-HEAT, described in MARC-CDC, User Information Manual, Volume I, CDC Publication Number 17309500.

WIN-12100

VI. <u>CLASS I PIPING SYSTEMS</u>

None

VII. <u>CLASS I STRUCTURES</u>

CE 309 STRESS

This program has been demonstrated to be substantially identical to those obtained by the original program, STRESS, developed at the Massachusetts Institute of Technology. The traceability of this program can be obtained at the Pacific International Computer Corporation.

5.E.3 COMPUTER PROGRAM TEST PROBLEM SIMILARITIES

I. BLOWDOWN LOADS

Minor modifications were made to the WATERHAMMER code to increase its utility such as the addition of a plot routine, revision of the output edit format, and increase of the number of legs which may be employed.

WATERHAMMER was written in FORTRAN IV for use on the CDC-3600 computer. It has been converted, at C-E, for operation on both the CDC-6600 and CDC-7600 machines. A test case, given in the manual, was reproduced with the code and by hand techniques. Agreement was excellent.

II. REACTOR VESSEL INTERNALS

The following programs' solutions have been demonstrated to be substantially identical to those obtained by hand calculations or from accepted experimental test or analytical results. The references below each program indicate where details of these comparisons can be found.

SHOCK

1. "Topical Report on Dynamic Analysis of Reactor Vessel Internals Under Loss-of-Coolant Accident Conditions with Application of Analysis to CE 800 MWe Class Reactors," Combustion Engineering Report CENPD-42, Combustion Engineering, Inc., Nuclear Power Department, Combustion Division, Windsor, Connecticut, Appendix B.

2. Gabrielson, V. K., "SHOCK - A Computer Code for Solving Lumped-Mass Dynamic Systems," SCL-DR-65-34, January 1966, pp 58-79.

ASHSD

1. "Topical Report on Dynamic Analysis of Reactor Vessel Internals Under Loss-of-Coolant Accident Conditions with Application of Analysis to CE 800 MWe Class Reactors," Combustion Engineering Report CENPD-42, Combustion Engineering, Inc., Nuclear Power Dept., Combustion Division, Windsor, Connecticut, Appendix A.

2. Ghosh, S., Wilson, E. L., "Dynamic Stress Analysis of Axisymmetric Structures under Arbitrary Loading," Report Number EERC 69-10, University of California, Berkeley, September 1969, pp 69-81.

SAMMSOR/DYNASOR

1. "Topical Report on Dynamic Analysis of Reactor Vessel Internals Under Loss-of-Coolant Accident Conditions with Application of Analysis to CE 800 MWe Class

Reactors," Combustion Engineering Report CENPD-42, Combustion Engineering, Inc., Nuclear Power Department, Combustion Division, Windsor, Connecticut, Appendix C.

NAOS

1. Dunham, R. S., Nickell, R. E., et al., "NAOS - Finite Element Analysis of Axisymmetric Solids with Arbitrary Loadings," Structural Engineering Laboratory, University of California, Berkeley, California, June 1967, pp 32-39.

III. <u>REACTOR COOLANT SYSTEM - DYNAMIC ANALYSIS</u>

ICES/STRUDL-II

The basic version of this code, Version 1.4, is described under 5.31.1, above. To facilitate dynamic analysis, additional Input/Output options and an eigenvalue analysis routine have been incorporated by Combustion Engineering. The validity of these additions have been carefully verified by appropriate test problems.

TMCALC

The program was developed by Combustion Engineering, Inc., and its validity has been carefully confirmed. The various matrix manipulations employed by the program were confirmed by hand calculations and the numerical integration procedure, Newmark's Beta-Method, used in the analysis of the reactor coolant system components for Millstone Nuclear Power Station, Unit Number 2, was confirmed by an alternate, independent, integration procedure which was, subsequently, incorporated into the program.

The alternate integration procedure employs a closed-form solution of the modal equations of motion over each time step of the integration process.

Input excitation is provided in digitized form and varies linearly between input points. The validity of the closed-form solution over a time increment was verified by hand calculations. After incorporation of the alternate integration procedure, TMCALC was again used to calculate the dynamic response of the reactor coolant system model shown in Figure 4.A–1, Appendix 4.A of the FSAR. The results obtained, using the two, independent, integration routines are substantially identical.

FORCE

The program was developed by Combustion Engineering, Inc., and its validity has been carefully verified by a series of test problems which were confirmed by hand calculations.

SHAKE

The program was developed by Combustion Engineering, Inc., and its validity has been carefully verified by a series of test problems which were confirmed by hand calculations. The test problems include those presented in the paper, “Response of Structural Systems to Ground Shock,” by Dana Young, presented at the Annual Meeting of the ASME, November 30, 1960.

IV. REACTOR COOLANT SYSTEM - STATIC STRUCTURAL ANALYSIS

None

V. ASME SECTION III, CLASS I COMPONENTS

None

VI. CLASS I PIPING SYSTEMS

None

VII. CLASS I STRUCTURES

The following programs’ solutions have been demonstrated to be substantially identical to those obtained by hand calculations from accepted experimental test on analytical results. The documents traceability of the following programs can be obtained at the Bechtel Power Corporation.

CE 611 TIME-HISTORY ANALYSIS

CE 617 MODES AND FREQUENCIES EXTRACTION

CE 641 RESPONSE SPECTRUM TECHNIQUE

CE 784 RESPONSE SPECTRUM TECHNIQUE

CE 792 RESPONSE SPECTRUM CALCULATION

5.F CONTAINMENT WATER INTRUSION INTO TENDON GALLERY DURING CONSTRUCTION [1]

Due to the recurrent appearances of water in some of the tendon sheaths and on the gallery floor, a formal inspection was made by Engineering personnel to determine a method to stop the water leakage. This inspection was performed on October 28, 1971.

The vertical tendon sheaths and trumpets were examined from the tendon access gallery, where water was observed running down the interior surfaces of the vertical tendon sheaths and dripping to the floor from the bearing plates. The number of the trumpet/bearing plate assemblies affected was 70 out of 124.

There were indications of light rusting on the interior surfaces of the trumpets and around the rims of the holes in the bearing plates, which had been painted; and in some trumpets, deposits of minerals were building up inside and around the edges of the holes. However, the galvanized surfaces of the inside of the tendon sheaths appeared to be free of rust and deposits.

Mineral deposits and damp areas were also noted in various locations along the gallery walls at construction joints.

A sample of the dripping water was taken by the Site personnel and submitted laboratory analysis.

From the report it was observed that:

1. The water is very alkaline, with a pH = 11.68.
2. The water is very hard.
3. The iron content is very low.

Two possible sources of water were investigated:

1. Curing water sprayed on the exterior concrete containment wall.
2. Ground water seeping into the concrete from below grade.

No tendons have been affected by water leakage since no tendons were installed in any sheaths while leaking water was present. Extreme care had been exercised to ensure that all water leakage was stopped in each tendon sheathing before installation of the tendon.

After consulting with American Drilling and Boring Company, they provided a proposed method of stopping the ground water via letter of April 4, 1972. After review and approval of procedures, work started on May 15, 1972.

NOTE: [1] From response to NRC Question Number 5.57.

The first scheme utilized was that of drillings upward through the access gallery ceiling, at elevation (-)32 feet 6 inches, to the construction joint between the Containment Mat and the Containment Wall at approximate elevation (-)26 feet 0 inches, and injecting chemical grout into these holes under a maximum pressure of 30 psi. It was theorized that the grout would travel along the same path as the water and seal the joint against leaking. This scheme worked around many of the sheathings, but in other instances the grout would travel to and leak from a sheathing near the drilled hole to the extent that no pressure could be built up during the grouting operation. Without a pressure build-up, the grout would not seal off the water. Approximately 40 of the 70 sheathings, which were leaking, were sealed by this method. On July 5, 1972, a second scheme for sealing off the water was initiated. This scheme involved putting a plug in the sheathing above and below the first sheathing joint, at elevation (-)22 feet 6 inches, and pumping the grout out through the joint. This scheme proved very successful and by July 29, 1972, the water leaking into the remaining 30 tendon sheathings was essentially stopped. However, the very humid weather during the weeks of July 30 and August 6, 1972, caused considerable condensation on the access gallery ceiling and the grouting operation was delayed until additional ventilation fans were installed to supplement the two fans then in use.

On August 14, 1972, with the use of one additional fan and with the arrival of less humid weather, the access gallery ceiling had dried up enough for grouting to proceed. By August 25, 1972, the leaks had been stopped in all tendon sheaths except 31V36, 31V31, 31V20, and 23V26. Several further attempts, utilizing scheme two, failed to completely stop the water seepage into these four tendon sheathings. However, the water seepage was in all cases under no pressure and in negligible quantities.

In early December 1972, a third scheme for sealing the tendon sheathing was initiated. This scheme involved the installation of a plug in the tendon sheath just above the bottom vertical trumpet. The sheath was then filled with chemical grout to approximately elevation 14 feet 6 inches. The top end of the sheath was then sealed with another plug and the air in the sheath was pressurized to approximately 40 psi. This pressure was held for 20 minutes to allow the grout to gel. The plugs were then removed and the grout in the sheath was expelled using air pressure. All remaining grout in the sheath was removed with a high pressure (6000 psi) water jet. Visual inspection of the sheathing showed it to be free of grout.

As of February 1973, there was only one location at which water was entering the access gallery. This location is near the construction opening and the water leakage was through holes drilled to the construction joint, through the gallery ceiling. These holes were not initially grouted in order to be able to observe this leakage during the post-tensioning of the containment. On July 11, 1973, the remaining holes were grouted.

As of July 16, 1973, vertical tendon installation, stressing, and greasing were completed. At all locations where water was indicated inside the sheathings, the grouting operation completely arrested all water seepage into the sheathings. Examination inside the sheathings prior to tendon installation and greasing showed a dry condition. Therefore, all tendons are installed in a non-corrosive environment.

A pressurized grease system connected to the tendon filler caps was installed after the fourth tendon surveillance inspection in 1986. As monitored in the sixth surveillance inspection report in 1996, the number of tendons in which water was found has been greatly reduced. No significant corrosion was identified beyond that noted at installation and reported during the fifth surveillance.

See FSAR Section 5.9.3.3.4, "Corrosion Protection", for additional data to prevent ground water intrusion.