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QUESTION 5J.1 Reinforcing Steel

Considering the critical nature of the structure, a material specification on splicing in conformance with ACI 318-63 does not provide adequate assurance of structural ductility. Revise your material performance criteria in this regard and provide more explicit information with regard to the type of cadweld splicing intended.

Indicate the extent to which splice stagger will be achieved.

Indicate the location of and extent to which splicing or tacking or reinforcing steel will be made by welding.

Discuss in detail the extent to which NDT requirements will be imposed on the reinforcing steel. Also, indicate how quality control will be exercised to ensure that these requirements are achieved. (If no requirements are imposed justify the omission.) Discuss similar requirements for the prestressing wire and anchorage hardware.

ANSWER Refer to 5.1.3.2

ACI 318-63 establishes the basic criteria for splicing and anchorage of reinforcement. In general, lapped splices will not be made in regions of high tension. If such a splice is made in this region, the problem will be considered to be one of anchorage rather than splicing. The anchorage requirements of ACI 318-63 are such that the yield strength of the bar will be developed.

Normally, no more than 50 percent of the tensile reinforcement will be lapped spliced within a length of 40 bar diameters where the spacing is less than 12 bar diameters.

In general welded splices will not be used, if welding is required it will be as specified in Paragraph 5.1.3.2. Criteria for cadweld splicing is covered in Appendix 5C.

The nil-ductility transition temperature criteria (as defined by the Charpy Impact Test) does not apply to the reinforcing bars and prestress tendon because it is not indicative of the type of loading encountered by the containment. The membrane strength of the structure is dependent upon numerous independent ligaments which preclude propagation of a crack initiated in a single ligament.

A further consideration is that prestressed concrete has been used successfully for bridges in cold weather climates for a number of years. The environment and dynamic loading of such a structure are more severe than that of a containment structure.

QUESTION 5J.2 — Describe the pressure/thermal load in the liner.

ANSWER — Figure 5J.2-1 shows the results of a preliminary analysis of reactor building pressure and liner temperature as a function of time after 14.1 ft^2 hot-leg pipe rupture. This analysis was performed using the digital computer code COPATTA.

QUESTION 5J.3 — Discuss the provisions which will be made to ensure that cranes cannot be displaced from the track.

ANSWER — The reactor building crane and fuel handling cranes including all supports are considered as Class 1 systems and equipment as outlined in Appendix 5A of the Preliminary Safety Analysis Report.

Refer to Appendix 5A

The stability of the crane systems will be assured in the form of complete tie-down of the bridge to the rail and the rail to the runway girder during operating conditions.

The reactor building crane support will consist of embedded WF sections adequately anchored to the containment wall to safely resist the eccentric loads induced by the crane rail loads. Braces will be provided as secondary support and will be anchored to a thickened portion of the liner plate. Additional anchor bolts will provide for the transfer of loads into the concrete. preliminary details are shown in Figure 5.1-1

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QUESTION 5J.4 (DRL 4.2) SEISMIC DESIGN

5J.4.1 (DRL 4.2.1) For all Class 1 systems and components provide the design basis load combinations and the proposed stress and deformation limits for each combination.

ANSWER The basic criteria for Class I systems and components are outlined in Appendix 5A which has been modified to include load combinations and stress and deformation limits.

5J.4.2 (DRL 4.2.2) Supply criteria or specific information on the interaction forces, deformation and stresses connected with the relative motions between the reactor vessel, steam generators or other large components. Indicate how these relative motions will be controlled by snubbers or other means, and what reaction forces (and corresponding stresses) will be transmitted to the pipes.

ANSWER Type of Loads

Reactor Vessel and Internals - The reactor and internals are subjected to two fundamentally different types of loading due to LOCA. The first is horizontal shaking due to the side thrust resulting from a rupture that occurs in a radial pipe leg. The majority of the thrust is applied directly to the vessel and is transmitted to the internals through the upper flange of the core support shield. The reactor respons are a complex mass vibrating on a spring composed mainly of the vessel support skirt. Since the peak load duration is much shorter than the natural period of the reactor, the response is typical of a structure subjected to a suddenly applied load. Initially, the reactor deflects twice as far as it would under an equal but slowly applied load, and then vibrates between this double deflection and its initial position, at its natural frequency. (This description is somewhat over-simplified, since the actual motion will be modified by reduction of thrust load with time, and damping.) Inertia loads result throughout the reactor. Also, since all components do not respond in phase, some impacting may occur between adjacent parts.

The second type of LOCA loading is that resulting from transient pressure differentials which occur across various components within the reactor. The pressure differentials are cyclic, generally having the appearance of a damped sine wave plotted against time. When the period of the pressure-time history is less than or of the same order as the natural period of the pertinent structure, dynamic effects predominate in the response. On the other hand, when the period of the pressure-time history is larger than that of the perti-

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nent structure, the response is predominantly static. The pressure-time histories of interest within the reactor vessel have finite rise times and, therefore, the internals are not subjected to the 2:1 suddenly applied load factor discussed above in the section on response of the reactor vessel to side thrust.

Similar loads result from an earthquake. Horizontal ground motion produces shaking of the reactor, differing only in magnitude and in that the excitation is applied at the junction of the vessel support skirt and the foundation. The vertical ground motion produces vertical intertial loading within the reactor which has an effect similar to vertical pressure differentials.

Pressurizer - Qualitatively, the pressurizer loads are similar to those on the reactor vessel. The LOCA loads are much smaller because of the smaller pipe size involved.

Steam Generator - Although the loads are similar, the response of the steam generator to a LOCA is greatly limited (reduced) by a lateral support at the top.

Methods of Analysis to be Employed for Reactor Internals and Core

All reactor internals and core components (including control rods) will be analyzed separately for stresses and deflections resulting from accidents and earthquakes.

Static or dynamic analyses will be used as appropriate. In general, dynamic analysis will be used for the subcooled portion of the LOCA and earthquakes, and static analysis will be used for the relatively steady state portion of the LOCA.

Dynamic analysis will include the response of the entire system (as applicable in each case) to the various excitations. For LOCA, the excitation will be applied at the appropriate nozzle or internals component. Where appropriate, the response of the reactor vessel on its support skirt will be used as input to the internals. The response of the internals will then be used as input to the core. Seismic excitation will be handled in a similar manner, except that the ground motion will be input at the junction of the support skirt and the vessel foundation. Lumped parameter simulation will be used generally.

For LOCA, predicted pressure-time histories will be used as input. For earthquake, actual earthquake records, normalized to the appropriate ground motion, will be used as input. Output will be in the form of internal's motions (displacements,

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velocities and accelerations), motions of individual fuel assemblies, impact loads between adjacent fuel assemblies and impact loads between peripheral fuel assemblies and the core shroud.

Seismic analysis will also be performed using the response spectra approach.

The relative timing of the various aspects of a given LOCA will be considered only as indicated by the various local time histories associated with that particular accident, although sensitivity to the time duration of the pulses and other calculated input will be investigated.

Where simultaneous occurrence of LOCA and the MHE is considered, it is intended that both excitations will be input to the system simultaneously. Relative starting times will be changed until maximum structural motions, indicating maximum stresses, are obtained. Alternately, the maximum stress from one component of the combination will be added to the square root of the sum of the squares of the other components.

Stress Limits

The basic loading combinations and the corresponding design stress criteria for the internals, vessel, supports, and piping are in section 5A. Each of the four cases of loading combinations will be discussed separately, and an explanation for its stress limits will be given with respect to the primary system.

Case I - Design Loads Plus Design Earthquake Loads - For this, the reactor must be capable of continued operation; therefore, all components including piping are designed to Section III of the ASME Code for Reactor Vessel[1]. This code's applicability and conservatism for these requirements are well known and need no elaboration.

Case II - Design Loads Plus Maximum Hypothetical Earthquake Loads

and

Case III - Design Loads Plus Pipe Rupture Loads - In establishing stress levels for these two cases, a "no-loss-of-function" criterion applies, and higher stress values than in Case I can be allowed. The multiplying factor of (1.2) has been selected in order to increase the code-based stress limits and still insure that for the primary structural materials; i.e., 304 SST, 316 SST, SA302B, and SA106C, an

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acceptable Margin of Safety will always exist. To illustrate this point, two of the primary materials, 304 SST and SA302B, have been selected, and the Margin of Safety has been calculated for each. These two materials are fairly representative of the others, since one is a stainless steel and the other a carbon steel.

The Margin of Safety (MS) between the design stress limit (S_d) and the ultimate stress (S_u) is defined as

$$MS = \frac{S_u - S_d}{S_d} \times 100\%$$

For 304 stainless steel at 600 F

$$S_u = 2.75\ S_y{}^{(2)}$$

Where S_y = minimum yield strength[1]

For the stated design limits

$$S_d = 1.2\ (1.5 \times S_m)$$

Where $S_m = 0.9\ S_y$

Therefore,

$$S_d = 1.2 \times 1.5 \times 0.9\ S_y = 1.62\ S_y$$

and

$$MS = \frac{2.75 - 1.62}{1.62} \times 100\% = 70\%$$

For SA302B at 600 F

$$S_u = 1.43\ S_y{}^{(2)}$$

$$S_m = 1/3\ S_u = 1/3\ (1.43)\ S_y = 0.48\ S_y$$

Therefore,

$$S_d = 1.2\ (1.5\ S_m) = (1.2 \times 1.5 \times 0.48)\ S_y = 0.86\ S_y$$

and

$$MS = \frac{1.43 - 0.86}{0.86} \times 100\% = 67\%$$

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It is shown below that the 67% margin calculated above will apply for all materials whose yield strength equals or exceeds 50% of the ultimate strength. Since the yield strength of most "code" carbon steels exceeds 50% of the ultimate, the margin calculated for SA302 grade B may indeed by considered typical.

For $S_y \geq 50\% \; S_u$,

$$S_m = \frac{S_u}{3}$$

$$S_{d_{max.}} = 1.2\ (1.5\ S_m)$$

$$MS = \left(\frac{S_u - S_d}{S_d}\right) = \left(\frac{3S_m - 1.8\ S_m}{1.8\ S_m}\right) 100\%$$

$$MS = \frac{1.2}{1.8} = 67\%$$

Where $S_y < 50\% \; S_u$, the value of S_m will equal $2/3\ S_y$, and the margin of safety will exceed 67%, as indicated by the calculation for 304 SST above, where $S_y \simeq 36\% \; S_u$, and the margin of safety is 70%.

Margins of safety are required to cover uncertainties in load, structural performance, and material properties. In view of the detailed and extensive engineering practices and analyses used on these components, and the use of minimum values of material strength properties, per ASME Section III, a margin of safety of 50% provides adequate conservatism. (This margin will be used for Case IV, below, covering simultaneous occurrence of accident and earthquake.) When considering Cases II and III, however, since only one of these two severe accident conditions is applied, a slightly higher margin on maximum stress ($P_m + P_b$) is achievable without penalty. From preliminary work already completed, it is apparent that a much greater margin on membrane stress is possible without penalty, since these stresses are not high. Taking advantage of these conditions permits representation of the Case II and Case III limits in a familiar form, simply 120% of the code stress allowables. This latter form has been adopted for present purposes. Future work is expected to justify much higher stresses.

<u>Case IV - Design Loads Plus Maximum Hypothetical Earthquake Loads Plus Pipe Rupture Loads</u> - As in Cases II and III, the "no-loss-of-function" criterion applies. Also, in the dis-

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cussion of Cases II and III, it was stated that a margin of safety of 50% was adequate. To insure that for this case the margin of safety will always be greater than or equal to 50%, the design stress level has been selected so that

$$S_d = 2/3\ S_u$$

with a resulting margin of safety:

$$MS = \frac{3/3 - 2/3}{2/3}\ (S_\mu) \times 100\% = 50\%$$

In Cases II, III, and IV, secondary stresses are neglected, since they are self-limiting. Design stress limits in most cases are in the plastic region, and local yielding has occurred. Thus, the conditions that caused the stresses can be assumed to have been satisfied.

It should also be mentioned that in applying this criterion, elastic equations are used for calculating all stresses. In the case of plastic bending, the maximum normal stress calculated by the elastic equation

$$S_b = \frac{MxC}{I}$$

where

S_b = maximum normal stress

M = applied moment

C = distance from neutral axis of S_b

I = moment of inertia of cross section about neutral axis

will always yield a normal bending stress greater or equal to the true maximum plastic stress. Therefore, ACTUAL MARGINS OF SAFETY will always be greater than or equal to the CALCULATED Margins of Safety. Where bending stresses are significant, the conservatism of elastic formulas is considerable.

Deformation Limits of Reactor Internals - Two primary safety considerations govern the deformation limits of the internals. Deformation shall not prevent the flow of coolant to the core. The specific deformation limits given below represent the limiting deflection of each component listed. Other considerations were not included, since the values given

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represent the deflection at which a safety limit is first reached. The "no-loss-of-function" deformations could cause a safety problem. The "allowable" deformations are those that are used as design limits.

Modes of Deformation of Reactor Internals

Components Required for Safe Shutdown of Reactor	Safety Implication	Allowable	No Loss of Function
Core Support Shield	Mode 1--Outward deflection of the shell will reduce the effectiveness of the internals vent valves, resulting in increased probability of uncovering the core during blowdown.		
	(a) Uniform radial expansion of the shell at the vent valve.	1/4"	3/8"
	(b) Outward local radial displacement of two valves (elliptical deformation of the shell).	1/2"	1"
	Mode 2--Inward deflection of the shell at the outward nozzles to prevent contact with guide assemblies.	1"	1-15/16"
	Mode 3--Deformation limit of the upper flange to insure that the core support assembly does not drop.		
	Uniform decrease in diameter.	1"	1-7/8"
	Mode 4--Axial elongation of the core support shield shall be limited to insure engagement of the fuel assemblies in the grid plates.	See Note 1	
Core Barrel	Mode 1--Decrease in diameter (local or average) to prevent distortion of fuel assembly spacer grids.	3/4"	15/16"
	Mode 2--Axial elongation of the core barrel shall be limited to insure engagement of the fuel assemblies in the grid plates.	See Note 1	

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Components Required for Safe Shutdown of Reactor	Safety Implication	Allowable	No Loss of Function
Upper Plenum Assembly	Mode 1--Limit radial expansion to maintain clearance between the shell and the internals vent valve wedge ring to insure valve operation.	1.5"	3"
	Mode 2--Limit radial compression to maintain clearance between the shell and upper guide tube structure.	2.4"	4.8"
	Mode 3--Bending of the cover as a plate shall be limited to insure engagement of the upper grid plate and the fuel assemblies.	See Note 1	
Control Road Guide	Mode 1--Limit axial deflection to insure engagement of the fuel assemblies in the upper grid plate.	See Note 1	
	Mode 2--Bending as a beam--contact between the guide tube and the control rod resulting from deflection of the guide tube must be limited so that the resultant frictional drag on the control rod will be small enough to permit control rod insertion.	(Later)	
	Mode 3--Cross-sectional distortion of individual tubes shall be limited to maintain clearance between the guide tubes and the control pins.	0.014"	0.029"
Fuel Assembly Guide Tube	Mode 1--The cross-sectional distortion of guide tubes shall be limited to maintain clearange between the tube and the control pin.	0.013"	0.027"

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Components Required for Safe Shutdown of Reactor	Safety Implication	Allowable	No Loss of Function
Lower Grid Plate Assembly	Mode 1--Downward deflection as a plate shall be limited to insure engagement of the fuel assemblies in the grid plates.	See Note 1	
Thermal Shield	No safety implication.		
Flow Distributor	No safety implication.		
Note 1	The combined axial displacement of the lower grid plate, the core barrel, the core support shield, the upper plenum assembly and the control rod guide assembly shall be limited to prevent disengagement of the fuel assemblies from the grid plates.	1"	1-3/4"

Margins of Safety

Margins of safety are, in general, selected such that design allowable deformations are approximately half of the deformations which cause a loss of function, providing a margin of safety of about 100%.

For the uniform radial expansion of the core support shield, a 50% margin of safety is specified because the "no loss of function" deformation of 3/8" is such a small percentage of the shell diameter. However, the loads are not great, and the margin between actual deformation and the "no loss of function deformation" is expected to be greater than 50%.

For the decrease in diameter of the core barrel a 25% margin is currently specified. This is because the specified deformations are small compared to the diameter of the core barrel, and because the 15/16" estimated "no loss of function" deformation is considered to be a very conservative estimate, which can probably be increased on further investigation. However, the 25% margin is considered adequate. All deformations are for the worst combinations of loads - which are expected to occur under Case IV combination.

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Snubbers and Dampers

The present dynamic analysis of the piping system indicates that shock repressers, if needed, will only be required at the location of the primary pumps and that no dampers or snubbers will be required.

REFERENCES

1. ASME Boiler and Pressure Vessel Code Section III, Nuclear Vessels, The American Society of Mechanical Engineers, 1965.

2. R. Wiesemann, R. Tome, and R. Salvatori, Ultimate Strength Criteria to Ensure No Loss of Function of Piping and Vessels Under Earthquake Loading, Westinghouse Electric Corporation, WCAP-5890 Revision 1, 1967.

3. W. Stokey, D. Peterson, and R. Wunder, "Limit Loads for Tubes Under Internal Pressure, Bending Moment, Axial Force and Torsion," "Nuclear Engineering and Design," Vol. 4, North Holland Publishing Company, Amsterdam, 1966.

4. P. Hodge, Jr., Plastic Analysis of Structures, McGraw-Hill Book Company, Inc., 1959, p. 201

QUESTION 5J.4.3 (DRL 4.2.3) Identify specific reactor internals which must maintain their functional performance capabilities to assure safe shutdown of the reactor. Provide calculated (or estimated) maximum limits of deformation or stress, at which inability to function occurs, for each component identified. Also, supply the calculated (or estimated) maximum design limit value, and the expected deformation or stress. In all cases identify the applicable loading combination and state the proposed margin of safety.

ANSWER See question 5J.4.2.

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QUESTION 5J.4.4 (DRL 4.2.4)

For reactor internals provide information that will permit evaluation of the effect of irradiation on the material properties and on the proposed deformation limits.

ANSWER

Evaluation of this area, covering flux levels in the structural components, changes in material properties due to irradiation, and the implications of these changes, is not yet complete. Data to date indicates that little irradiation damage occurs at fluences less than 10^{19} (>1 Mev). Of the major structural components, only the core barrel, the thermal shield, the flanges bolted to those shells, and the upper half of the lower grid plate heavy grillage will be subjected to fluences in excess of 10^{19}. To date there is no indication that severe or unacceptable damage will occur in any component. The results of this study are expected to be available in June, 1968.

QUESTION 5J.5 (DRL 16.5)

Discuss the effect of blowdown forces on reactor internals by identifying appropriate load combinations and deformation limits.

ANSWER

See question 5J.4.2.

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QUESTION 5J.6 (DRL 5.1)

METHODS AND CRITERIA

5J.6.1 (DRL 5.1.1)

Provide complete lists of all:

(a) Class I Structures,
(b) Class I Components,
(c) Class II Structures,
(d) Class II Components,
(e) Combined structures, i.e., structures consisting simultaneously of Class I and Class II elements, and
(f) Class I equipment housed in or adjacent to, or supported by, Class II structures and components.

ANSWER

(a)(b) The lists presented in Section 2.0 in Appendix 5A, as modified by Amendments 1, 2 and 3, are complete.

(c)(d) The use of sub-classification has been found convenient for Class II structures.

a. Sub Class IIA:

These are components of moderate importance whose limited damage would not result in a release of radioactivity but whose action under failure could interrupt power output while affecting a controlled plant shutdown. These are

1. Turbine building, including pedestal.
2. External electrical power system.
3. Secondary coolant system.
4. Demineralized water system.
5. Plant air system.
6. Lube oil and hydrogen seal oil system.
7. Main feedwater pumps.
8. Chemical addition system.
9. Auxiliary power system (other than emergency).
10. Circulating water system, including cooling towers.

b. Sub Class IIB:

These are structures, systems and equipment whose failure could inconvenience normal plant operation but which are not essential to generation of power, safe shutdown, or reactor maintenance and safety. In essence, this classification encompasses those structures, systems and equipment excluded from the above classification.

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(e) There are no combined structures.

(f) Plant layout is undertaken with the intention of minimizing the dependence of Class I Systems on Class II Structures, however, it is required to some extent at the following:

a. Main steam lines to the control valve at the high pressure turbine.

b. Isolated runs of emergency bus duct.

5J.6.2 (DRL 5.1.2) Describe the protection which will be provided to Class I equipment which are not located in, or supported by, Class I structures.

ANSWER Attention will be given to a conservative approach to protect Class I equipment which is not supported by or located within Class I structures. Appropriate consideration will be given to the design of that area to provide a degree of safety in design consistent with the functional requirements of the Class I equipment. In particular, the main steam line will be routed and supported to prevent any damage while maintaining its functional requirements. The turbine building including the pedestal can withstand the maximum hypothetical earthquake due to the important nature of the turbine elements and as a result will be compatible with the attached steam piping and safeguards bus ducts.

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5J.6.3 (DRL 5.1.3) State how the earthquake loads for these Class I components will be established, since they are not supported by Class I structures.

ANSWER A general description of the design approach for these isolated conditions is given in the answer to question 5J.6.2 including a discussion of the inherent strength under seismic conditions because of the special deformation restrictions dictated by normal turbine operation. Specific attention will be given to areas supporting Class I equipment.

4 The response of the Class II structures to the earthquakes for Class I structures will be used to determine the input accelerations and displacements to the supported component. The components will be analyzed using the appropriate degrees of freedom to adequately simulate the response of the component. Input to the component will consist of forcing functions consisting of the building accelerations amplified at the level of support and ground accelerations at or near ground level.

These components are identified in the answer to Question 5J.6.1 part (f).

5J.6.4 (DRL 5.1.4) Describe the design methods used for combined (Class I and Class II) structures.

ANSWER There are no combined (Class I and Class II) structures. The structures possessing combined characteristics have been designated as Class I and will be designed accordingly.

5J.6.5 (DRL 5.1.5) For plant structures and equipment rated other than Class I, indicate in detail the design criteria for seismic loading.

ANSWER The design criteria for structures and equipment other than Class I is divided into sub classes as outlined in the answer to question 5J.6.1, c and d, for Class II A. The analysis will be made using an equivalent static loading of 0.10g or the Uniform Building Code, Zone 2, requirements, whichever is greater. The allowable working stress range of the materials involved will not be increased for seismic loading on structures and equipments in this classification.

Class II B structures will be designed using the Uniform Building code, Zone 2, with the normal allowable stress increase.

5J.6.6 (DRL 5.1.6) It appears from the PSAR that the foundations of the containment and other structures will rest on layers of sands, gravels, silts, and silty clays. It is not stated whether these materials are insensitive to accelerated weathering, or whether they expand when exposed to the atmosphere, during construction. Provide information on:

(a) The extent to which the above are true;
(b) The construction procedure that will be used to avoid damage to these materials during the time interval between excavating and installing of foundations; how they will be protected;
(c) What the shape of the excavation will be; how the excavation will be drained;
(d) The provisions that will be made to accommodate differential settlements during earthquakes.

ANSWER

(a) The foundation soils are characterized as dense-to-very-dense sandy silts, silty clays with some gravel with a general increase in density with depth, based on the results of the soil and foundation investigation program. The upper soils have a moderately high potential for erosion or weathering as evidenced by local ground surface weathering and topographic erosional features and qualitive results of the laboratory testing program. The soils are less sensitive to weathering with depth, and foundation excavation should be relatively stable and insensitive to weathering during construction for a significant length of time.

(b) As stated in the PSAR, the soils increase in density with depth and are, therefore, less sensitive to weathering. All grading and site preparation will be under continual guidance and inspection of a soil engineer and excavated surfaces will be appropriately graded to prevent ponding of water during construction.

(c) The rough excavation will be a plane at elevation 164 ft± sloping away from the major structures at from 0 to 2 percent to the plant storm drain systems. Overland flow will be intercepted at the perimeter of the excavation and carried around the plant in surface ditches and through the plant in buried pipe. The excavation for the containment, turbine building, and auxiliary building will be lower than the surrounding natural drainage channels. The soil surface runoff resulting from rainfall over the excavated areas for the containment, turbine building and auxiliary building will have to be pumped out of these areas by the constructor of the storm drain system. Due to the configuration of the excavation for these structures, sumps will probably be required in the containment area, the turbine building area, and the radwaste area. The preliminary plant excavation plan is shown in Figure 5J.6.6-1.

(d) No significant differential settlement is anticipated during earthquakes (see answer to question 5J.13).

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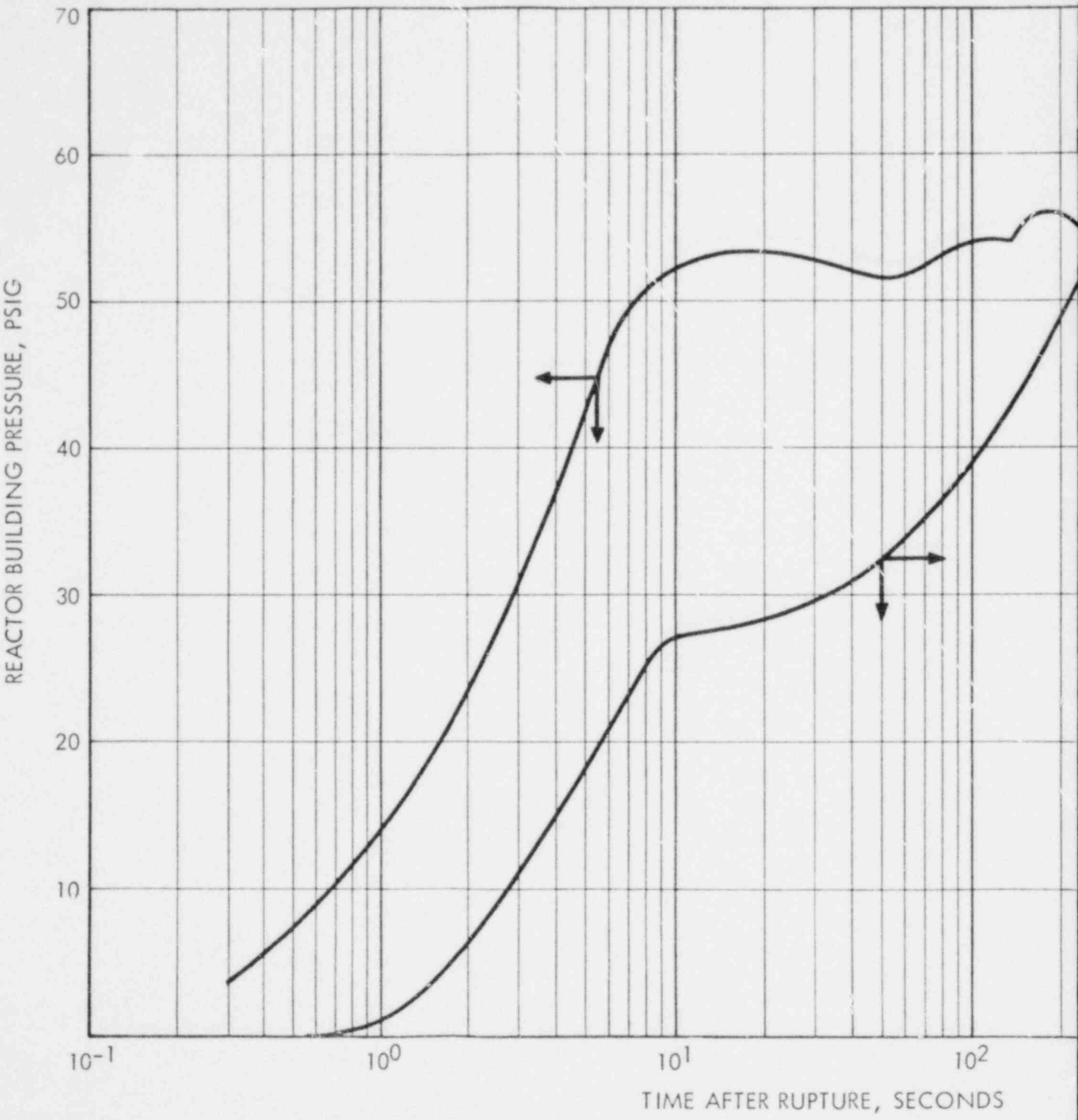

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REACTOR BUILDING PRESSURE, PSIG
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10^{2}
TIME AFTER RUPTURE, SECONDS

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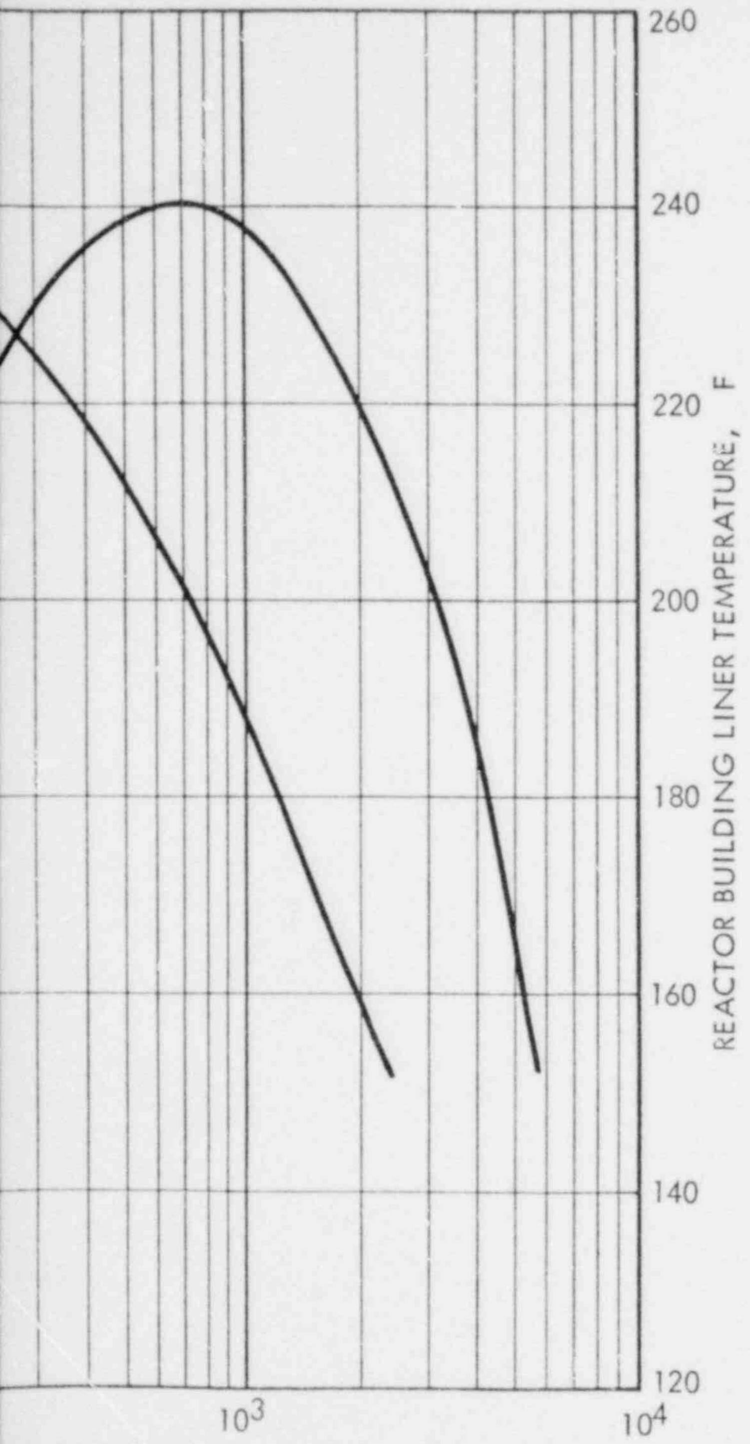

FIGURE 5J.2-1
REACTOR BUILDING PRESSURE AND LINER TEMPERATURE AS A FUNCTION OF TIME AFTER 14.1 FT^2 HOT - LEG PIPE RUPTURE

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Amendment 1

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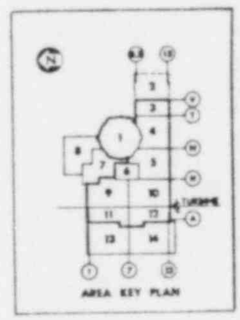

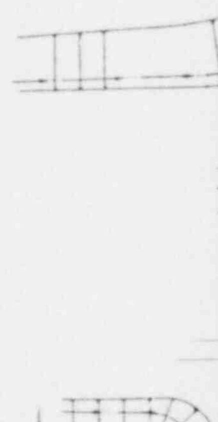

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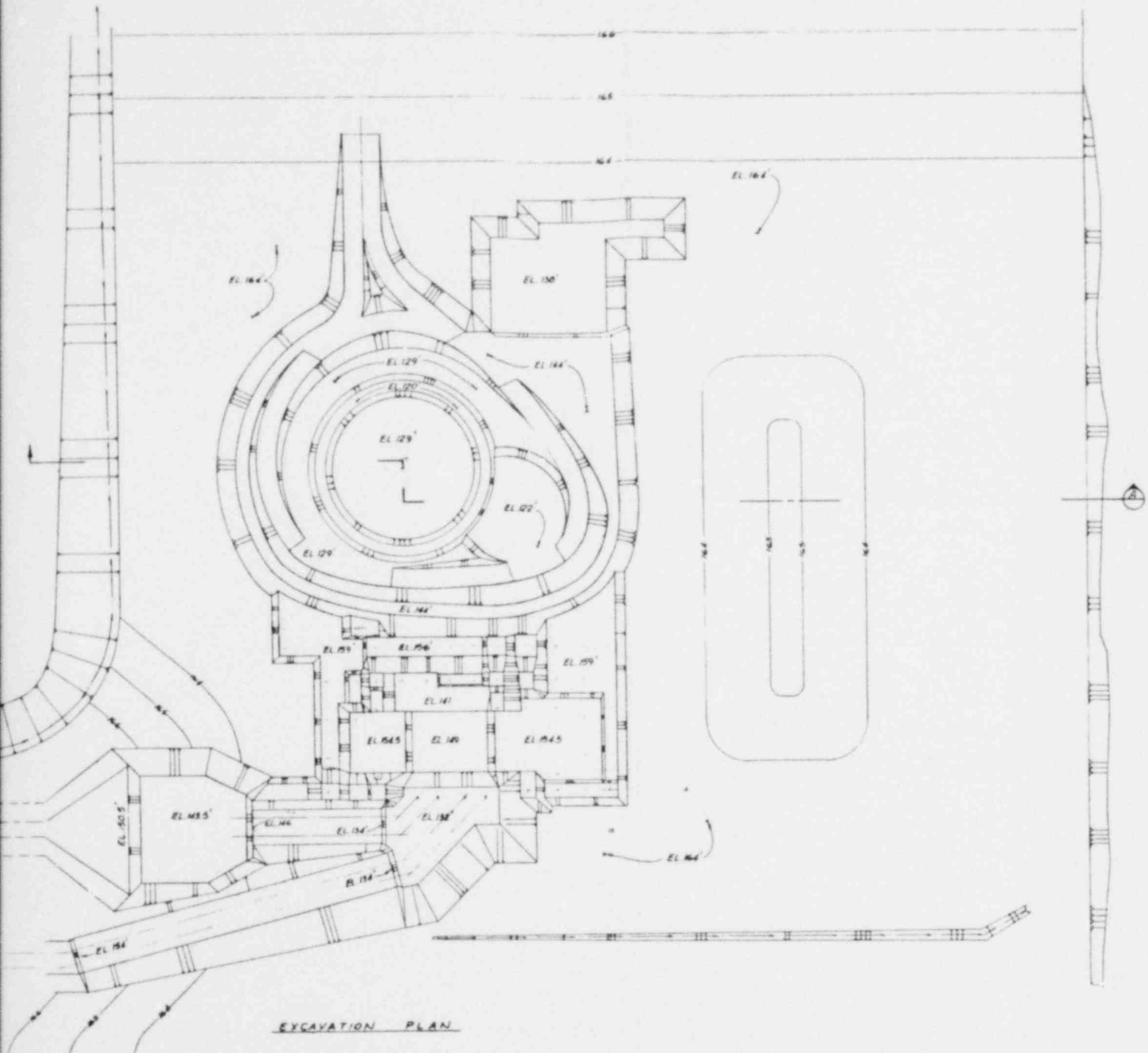

FIGURE 5J.6.6-1
PLANT EXCAVATION PLAN

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SACRAMENTO MUNICIPAL UTILITY DISTRICT

AMENDMENT 3

5J.6.7 (DRL 5.1.7) Justify the load factors used for Class I structures other than the containment. Indicate design methods. Provide a list of codes, standards & specifications on which the design & construction will be based. Where applicable consider transient thermal gradients instead of steady state gradients.

ANSWER A complete discussion of load factors is presented in Appendix 5E of the PSAR. Particular attention in this section is directed toward the containment structure. However, the extent of application of these factors to structures other than the containment is indicated in Section 3.1.3, of Appendix 5A including the modifications specified in Amendment 1. It should be noted that a portion of the loading conditions indicated in Section 3.1.3 of Appendix 5A are maintained in the allowable stress range. This approach defines the upper limit of load factors as 1.0, considering the fact that the code values inherently contain factors of safety. In addition, normal allowable increases are neglected. The factors in the remaining equations are completely consistent with the predictability of the loads and the maintaining of a conservative approach to the following structures:

a. Reactor Building (Other Than Containment)

The reactor building will be designed to withstand all loads imposed upon it during normal operation and under those conditions which may rationally be expected in the unlikely event of an accident. In general, the primary and secondary shield walls will be sized to provide shielding. These walls will be designed to withstand all dead loads, live loads, thermal loads, and design earthquake loads at allowable stress levels with no stress increases as outlined in Section 3.1.3 of Appendix 5A and referenced below. In addition, the refueling canal will be designed to withstand the hydrodynamic effects of the design earthquake.

The final design of this structure will also consider a rational combination of loading conditions evolving from a DBA. Typical effects which will be considered are those originating from dead loads and appropriate live loads, loads from any one pipe reaction, thermal loads, transient pressure buildup, and maximum hypothetical earthquake loads.

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The loads will be combined as outlined in Section 3.1.3 of Appendix 5A using the yield capacity of the structure as the appropriate stress limit. Localized transient thermal effects will be evaluated with full consideration given to the time history. Localized concrete yielding will be permitted only 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 will be given to the time increments associated with these postulated failure conditions, and recommended yield capacities will be appropriately increased when a transient analysis demonstrates that the rapid strain rate justifies this approach (See References). The walls will also be designed to provide adequate protection for potential missile generation which could damage the containment liner.

b. Fuel Storage Building

The Fuel Storage Building superstructure is a steel and concrete block structure. Lateral loads in the east-west direction are resisted by rigid steel frames. Lateral loads in the north-south direction are resisted by concrete block shear walls. Lateral loads and displacements will be determined, using the response spectrum approach based on foundation acceleration. The foundation for this building will be the Spent Fuel Storage Pool, which has been found to be sufficiently rigid to be considered as subjected to a maximum acceleration equal to the maximum ground acceleration.

Substantial attention will be given to the action of the new fuel storage racks due to the value of the stored fuel elements, as well as overall plant safety.

Complete structural separation will be provided between the Fuel Storage Building and adjacent buildings to allow for maximum anticipated lateral displacements during seismic conditions.

The Spent Fuel Storage Pool is designed as a rectangular concrete tank with the top edges free and the bottom edges fixed. Loads in addition to normal operating loads include:

1. Lateral loads due to earthquake motion, including hydrodynamic pressure, and

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2. Thermal stresses due to possible fuel pool cooling system loss of function, as well as other associated thermal transient conditions.

The entire structure will be designed at stress levels outlined in Section 3.1.3 of Appendix 5A and referenced below.

c. Auxiliary Building

The Auxiliary Building is a three story reinforced concrete structure with a partial basement. The building will have reinforced concrete roof, floor slabs, beams, and columns. Exterior walls will be primarily concrete and interior wall masonry. Laterial loads will be resisted by concrete and masonry shear walls. Miscellaneous platforms, equipment supports, and stairs will be of steel construction. In addition, particular attention will be given to the deflections induced by design loading conditions, particularly seismic forces, so as to insure predictable operation of all instrumentation.

Reference is made to the stress levels outlined in Section 3.1.3 of Appendix 5A and those referenced below.

d. Spray Ponds

The Spray Ponds will be constructed of reinforced concrete and will be embedded in the ground. The vertical walls will be designed to resist lateral loads caused by earth pressure when the ponds are empty. 4

Under conditions when the ponds are full, the vertical walls shall be designed to resist all lateral loads caused by hydrostatic and hydrodynamic effects of the contents during seismic conditions, as well as lateral soil pressures.

e. Storage Reservoir

The design of the storage reservoir is discussed in Appendix 29 and the answer to Question 5J.25.

The design and construction of the Class I structures other than the containment will be accomplished using the following:

1. Applicable Construction Codes and Specifications

Reference is made to the construction codes of practice referenced in considerable detail in Section 5.4. This listing will apply to all structures on the project.

2. Applicable Design Codes, Specifications, and References:

   (a) Conventional Codes

      (1) Uniform Building Code, 1967 Edition

      (2) A.C.I. Building Code (ACI 318-63), 1963 Edition

      (3) A.I.S.C. Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, 1963 Edition

      (4) State of California General Safety Orders

      (5) ASME Boiler and Pressure Vessel Code, Section III, Nuclear Vessels, Section VIII, Unfired Pressure Vessels, Section IX, Welding Qualifications, Latest Editions

   (b) Related Specifications

      (1) A.W.S. Standard Code for Welding in Buildings Construction

      (2) A.W.S. Standard Specifications for Welded Highway and Railway Bridges

      (3) A.I.S.C. Commentary on the Specification for the Design Fabrication and Erection of Structural Steel for Buildings

      (4) A.I.S.C. Code of Standard Practice

      (5) A.I.S.C. Specification for Structural Joints Using ASTM A325 Bolts

      (6) Australian Standard No. CA.2, SAA Code for Concrete in Buildings, Standards Association of Australia, Sydney, 1958

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(c) Material Specifications

(1) A.S.T.M. Specifications for Structural Steel

a. A7-61T Miscellaneous Steel

b. A36-63T Structural Steel

c. A441-64T High Strength Low Alloy Structural Steel

d. A514 (Grades B and F) Specialized Steel

e. A53-64 Welded and Seamless Steel Pipe

f. A120 (Grade B) Miscellaneous Pipe

(2) A.S.T.M. Specifications for Fasteners

a. A307-64 Low Carbon Miscellaneous Connections

b. A325-64 High Strength Connections

c. A490-64 Specialized Connections

(3) A.S.T.M. Miscellaneous

a. A307, Grade A Anchor Bolts

b. A193, Grade B7 High Strength Anchor Bolts

c. A233 Mild Steel Arc-Welding; Electrodes

d. A123 Galvanizing

(4) A.S.T.M. Specifications For Reinforcing Steel

a. A-15 Billet Steel (Intermediate Grade)

b. A-408 Billet Steel (Intermediate Grade)

c. A-432 Billet Steel (High Strength)

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d. A-431 Billet Steel (High Strength)

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3. Specialized Reference Material

   (a) U.S. Reactor Containment Technology - Vol. I, II - Oak Ridge National Laboratory and Nuclear Safety Information Center

   (b) Nuclear Reactors and Earthquakes - TID7024 - United States Atomic Energy Commission Division of Technical Information

   (c) Design of Structures to Resist Nuclear Weapons Effects - ASCE - Manual of Engineering Practice (No. 42) 1961

   (d) Rectangular Concrete Tanks - Portland Cement Association - Structural Bureau (ST 63)

   (e) Design of Multistory Reinforced Concrete Buildings For Earthquake Motion - Portland Cement Association by Blume, Newmark, and Corning 1961.

   (f) Wind Forces On Structures - ASCE Paper No. 3269 (1961)

4. Project Reports

   (a) Report to Sacramento Municipal Utilities on Seismic Hazard at the Clay Site - P. Byerly

   (b) Geology & Seismology - Bechtel Corporation

   (c) Soils and Foundations Investigation Report - Bechtel Corporation

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QUESTION 5J.7 (DRL 5.2) — CONTAINMENT STRUCTURAL DESIGN

5J.7.1 (DRL 5.2.1)

In certain circumstances the containment structure base may be located below water level. It appears that no layer of porous concrete and no membrane water-proofing exists between the soil and the containment. Consider the possibility of cracking of the concrete in the base mat, in the cylindrical wall and in the prestressing gallery. Ground water may reach the liner and the prestressing tendon anchors. The effect on the stability of the liner and possible corrosion of liner and tendons should be investigated. Explain drainage provisions at the containment lower section.

ANSWER

The normal groundwater is approximately 95 feet below the bottom of the containment and under no circumstances is it expected to rise to the elevation of the base slab. Figure 2.4-2 shows the relationship of the site to the surrounding area. The containment will be completely surrounded by paved areas or other buildings. The paving slopes away from the containment and will be detailed to prevent water seepage down the exterior walls.

5J.7.2 (DRL 5.2.2)

Provide the following:

(a) A preliminary design drawing of the containment, presenting details of the base slab, dome ring beam, cylinder-slab juncture, vertical buttresses and inspection gallery; showing reinforcing, prestressing, and liner features, including liner anchors;

(b) Scaled load plots for moment, shear, deflection, longitudinal force, and hoop tension, in order that an appraisal can be made of the significance of the various loadings which influence the containment design; Provide these plots as a function of containment height for prestress, dead, pressure, design earthquake wind, liner thermal (normal and accident) and concrete thermal (normal and accident) loading; 4

(c) The normal operating transient and steady state thermal gradients to be used in the design of the containment for typical winter and typical summer day;

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(d) The transient and steady state thermal gradients through the containment envelope during the design basis accident for typical winter and typical summer day.

ANSWER

(a) The preliminary design details of the containment structure are shown in Figure 5.1-1, 5.1-2, and 5.1-3 of Section 5 in the PSAR. These figures have been modified to include the requested preliminary information which is available.

(b) Preliminary scaled load plots for moment, shears, forces, and deflections are included in Figures 5J.7.2-1 through 5J.7.2-6 for all loading conditions. A plot of the thermal loading of the liner plate has been excluded due to the nature of its membrane action under prestress and thermal conditions. However, a description of this action is discussed in Questions 5J.7.4, 5J.7.18, 5J.7.19 and 5J.7.20 as well as Sections 5.1.3.4, 5.1.4.6 and 5.1.4.9 of the PSAR.

(c) (d) The steady state and transient thermal gradients through the containment walls are shown in Figure 5.1-4 as submitted in Amendment 1. To provide futher clarification this figure has been modified to demonstrate the effects of the maximum range of temperature fluction which are possible at the site. This modification is shown in the Figures 5J.7.2-7 and 5J.7.2-8. In addition to the gradients shown thermal stress differences due to varying concrete thicknesses (i.e., through the ring girder) will be considered for all conditions including startup.

Two items require consideration:

(1) The temperature range indicated in these figures represents extreme conditions. Normal fluctuations are anticipated to be substantially less severe. Refer to Section 2.3.3.6 and Appendix 2A.

(2) It should be noted that these fluctuations even under extreme conditions have little influence on the thermal gradients generated by postulated accident conditions.

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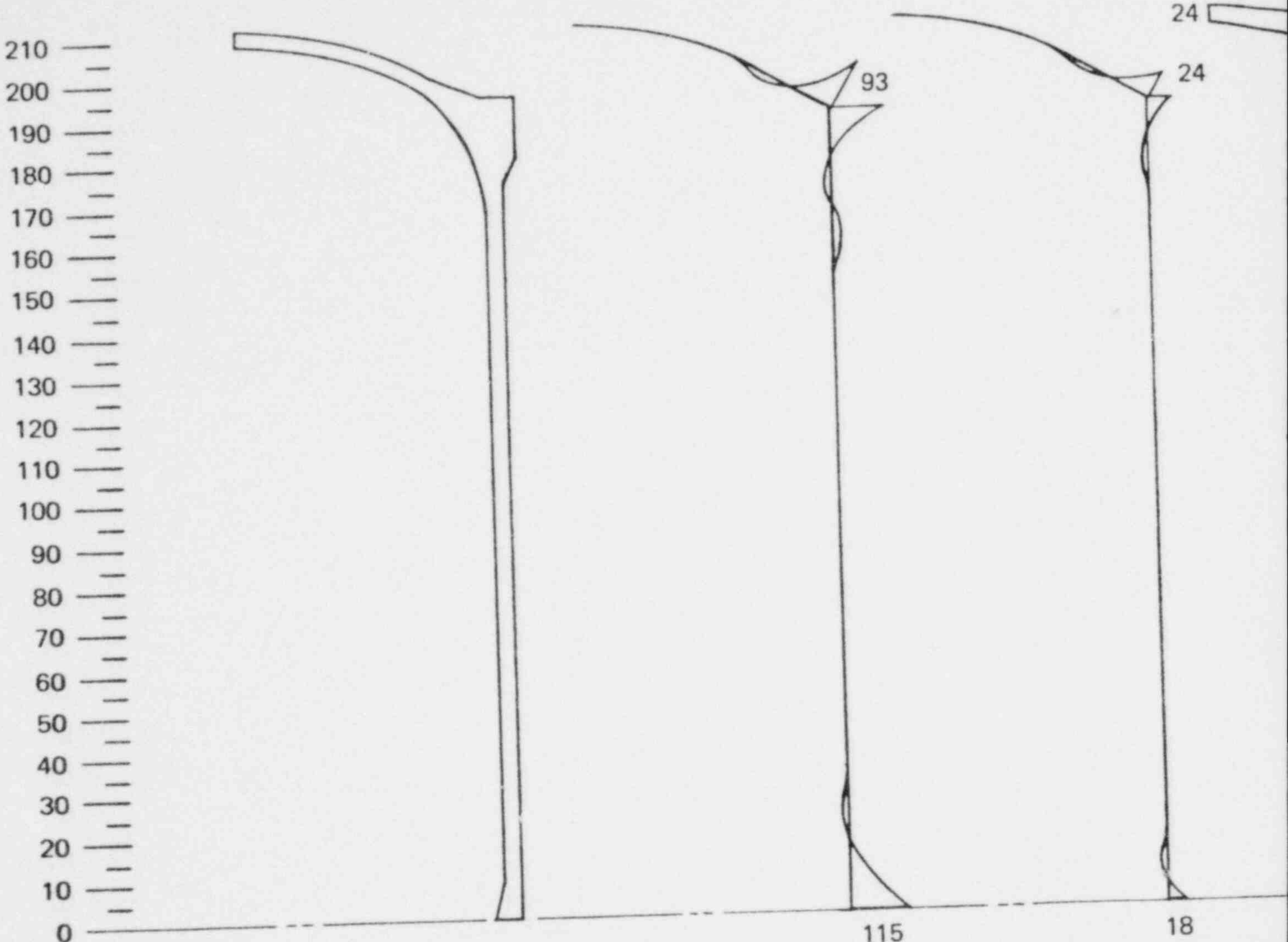

MERIDIONAL

MOMENT $\left(\frac{Ft\ K}{Ft}\right)$

HOOP

MOMENT $\left(\frac{Ft\ K}{Ft}\right)$

NOTE:
MOMENTS SHOWN ON TENSION SIDE.

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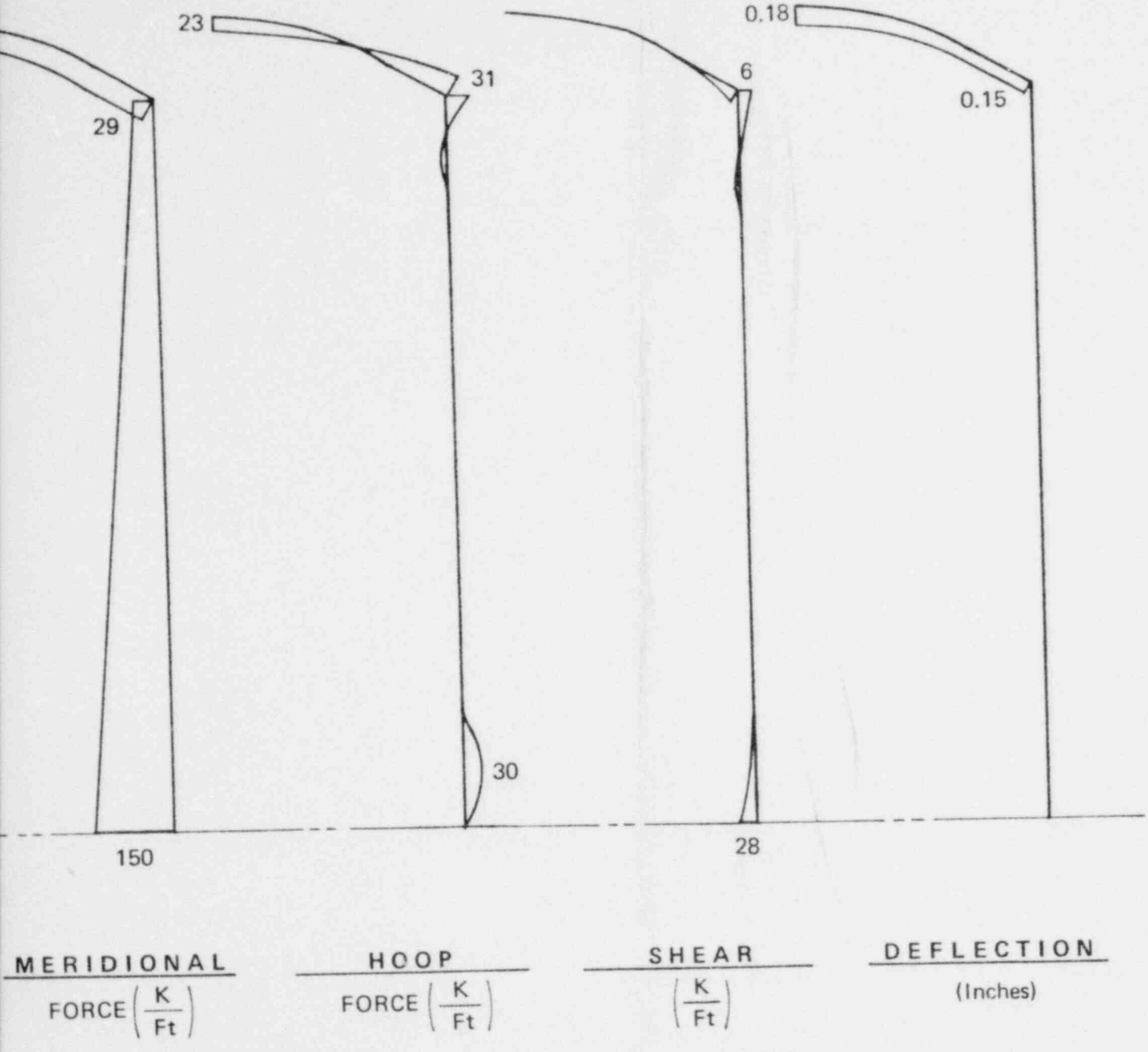

FIGURE 5J.7.2-1
CONTAINMENT STRUCTURE STRESS ANALYSIS
PRELIMINARY MANUAL CALCULATIONS
DEAD LOAD (D)

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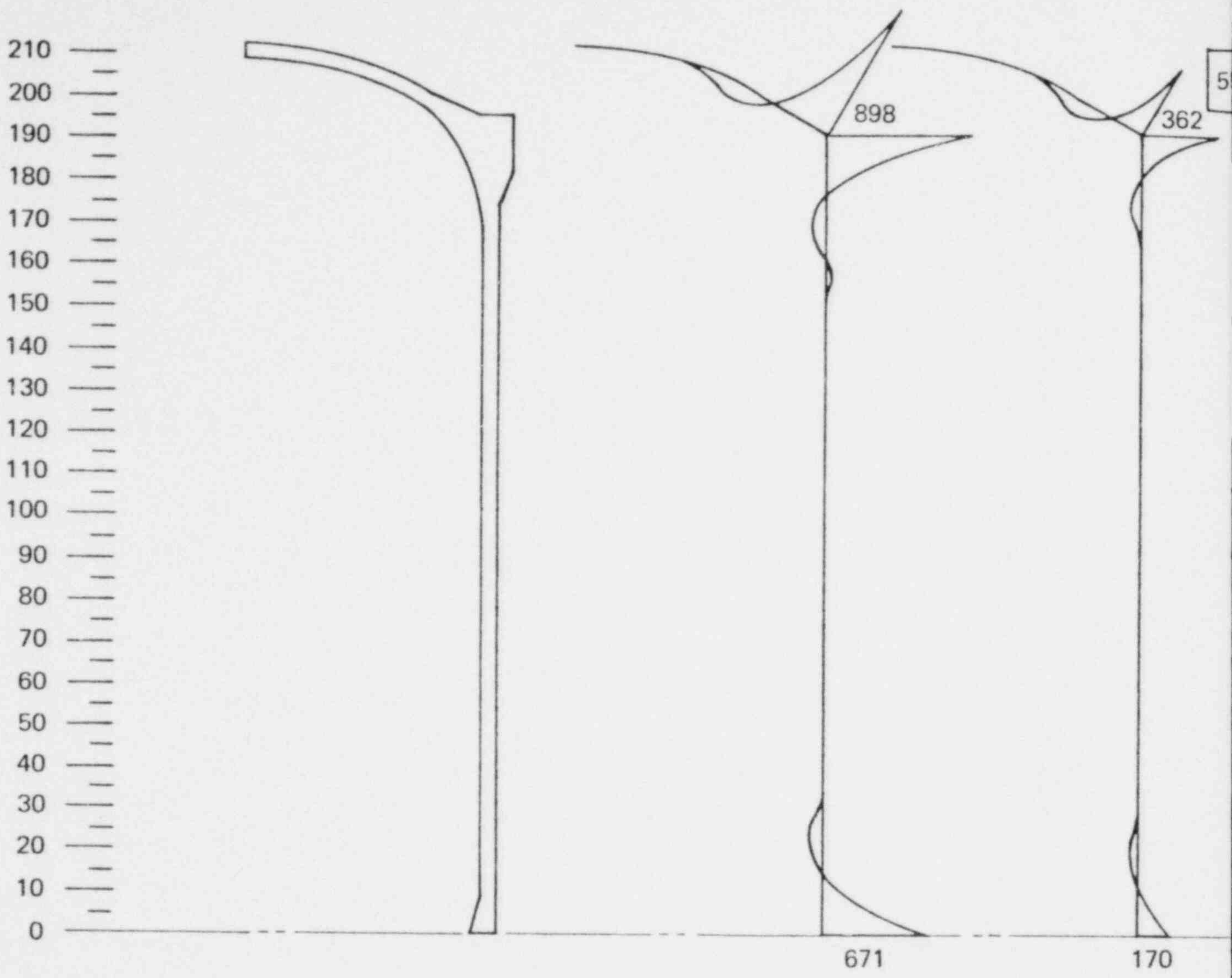
Amendment 3



MERIDIONAL

MOMENT $\left(\frac{\text{Ft K}}{\text{Ft}}\right)$

HOOP

MOMENT $\left(\frac{\text{Ft K}}{\text{Ft}}\right)$

NOTE:
MOMENTS SHOWN ON TENSION SIDE.

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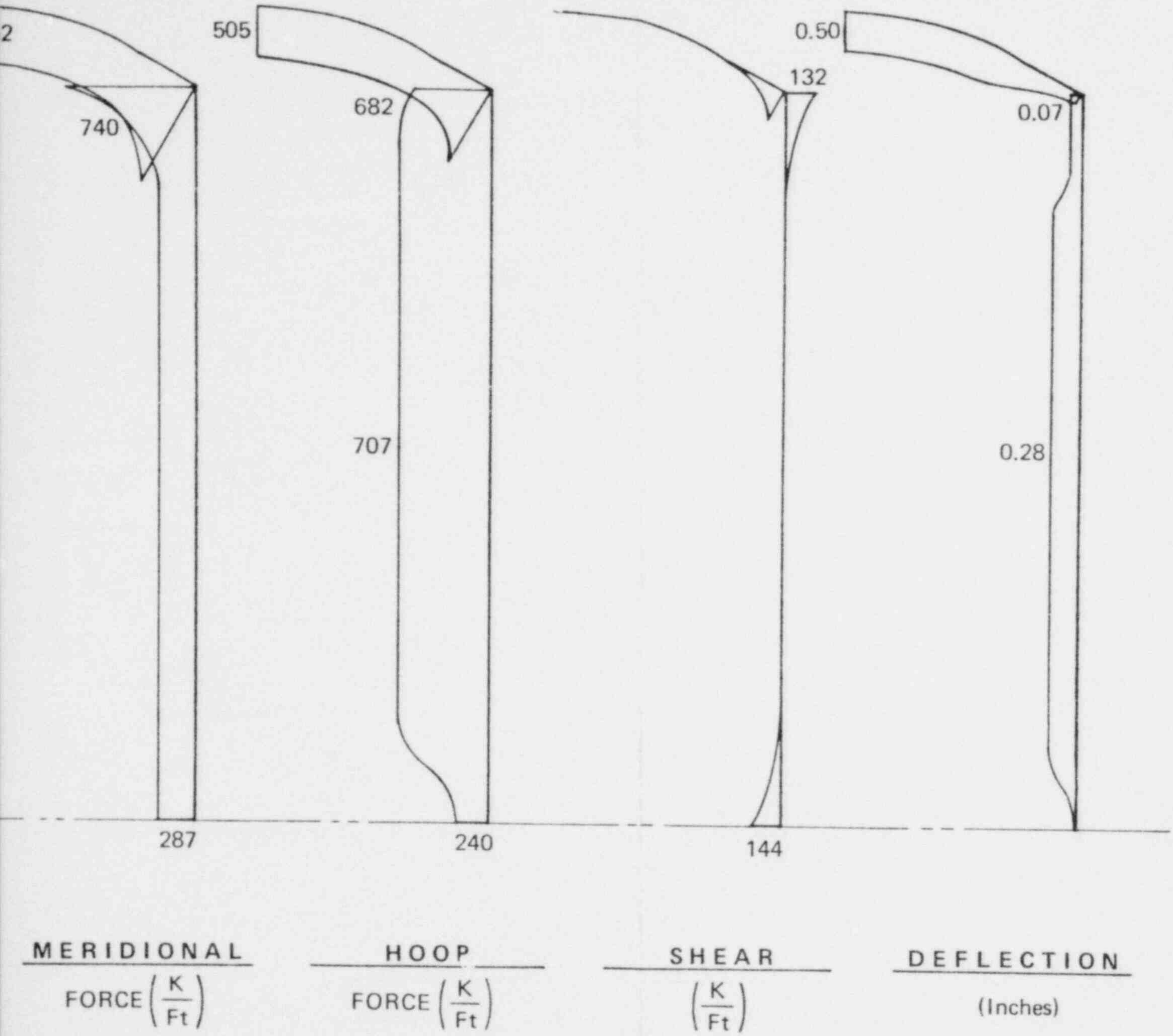
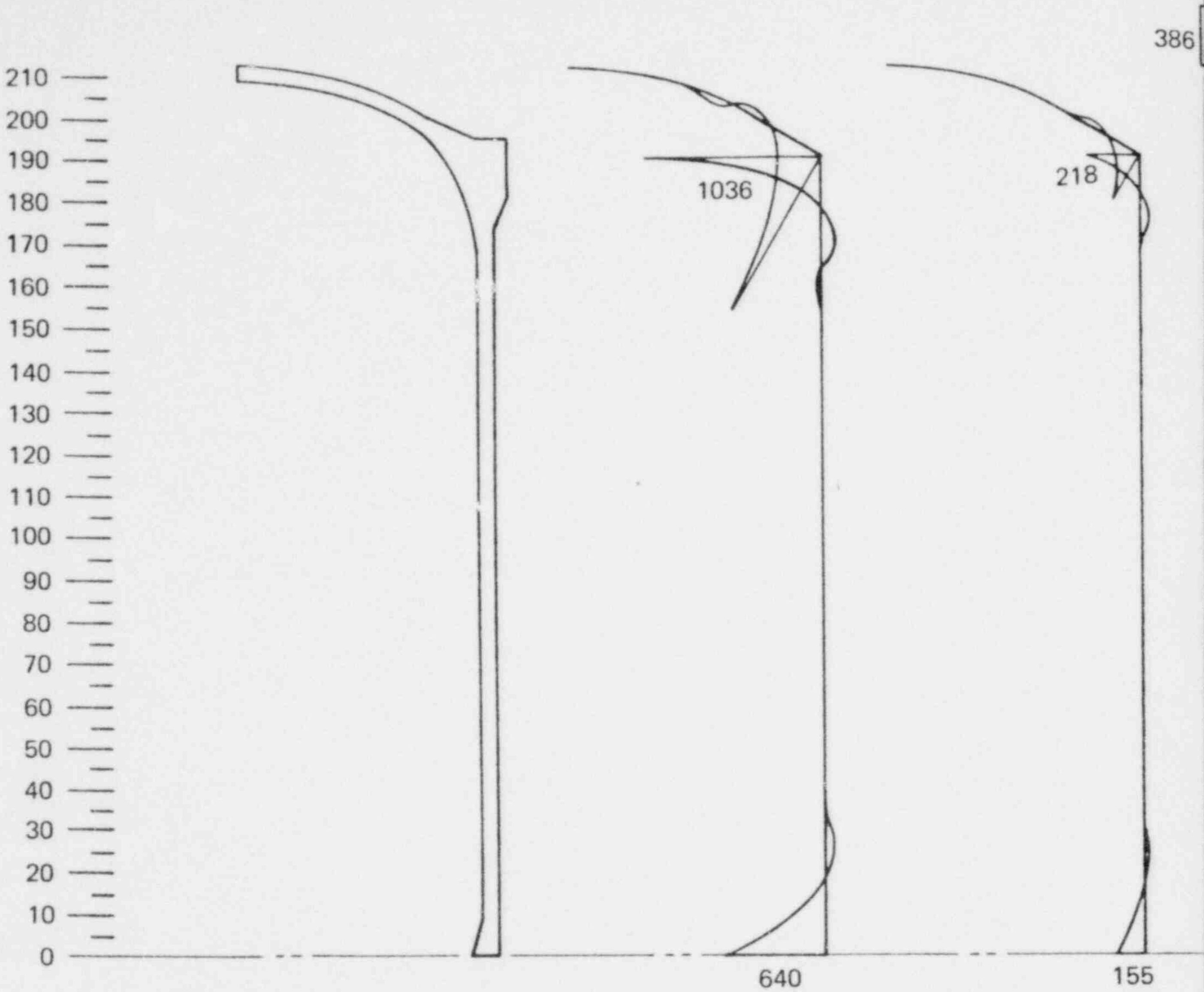


FIGURE 5J.7.2-2
CONTAINMENT STRUCTURE STRESS ANALYSIS
PRELIMINARY MANUAL CALCULATIONS
PRESTRESS LOAD (F)



MERIDIONAL

MOMENT $\left(\frac{\text{Ft K}}{\text{Ft}}\right)$

HOOP

MOMENT $\left(\frac{\text{Ft K}}{\text{Ft}}\right)$

NOTE:
MOMENTS SHOWN ON TENSION SIDE.

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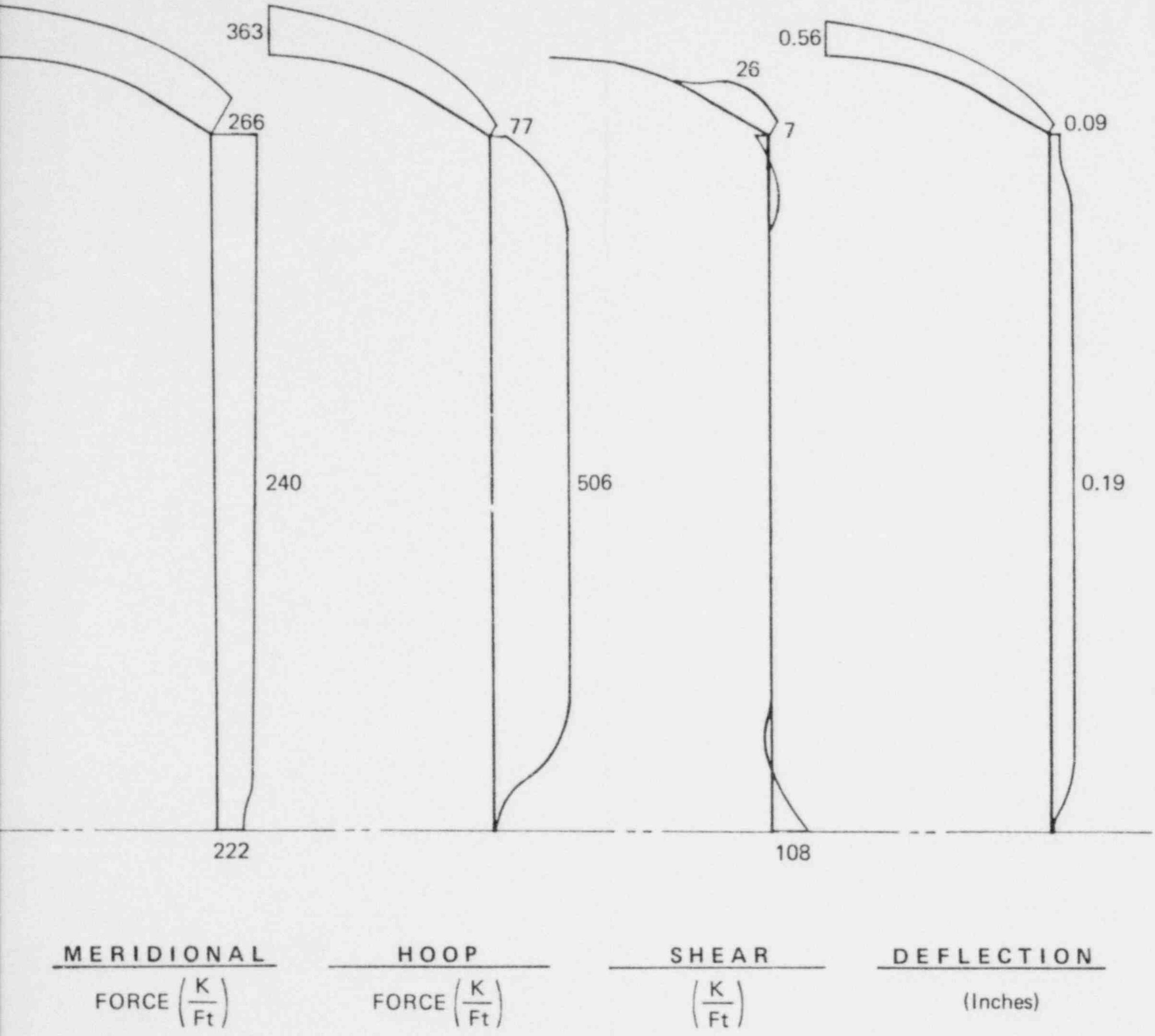

FIGURE 5J.7.2-3
CONTAINMENT STRUCTURE STRESS ANALYSIS
PRELIMINARY MANUAL CALCULATIONS
PRESSURE LOAD (P)

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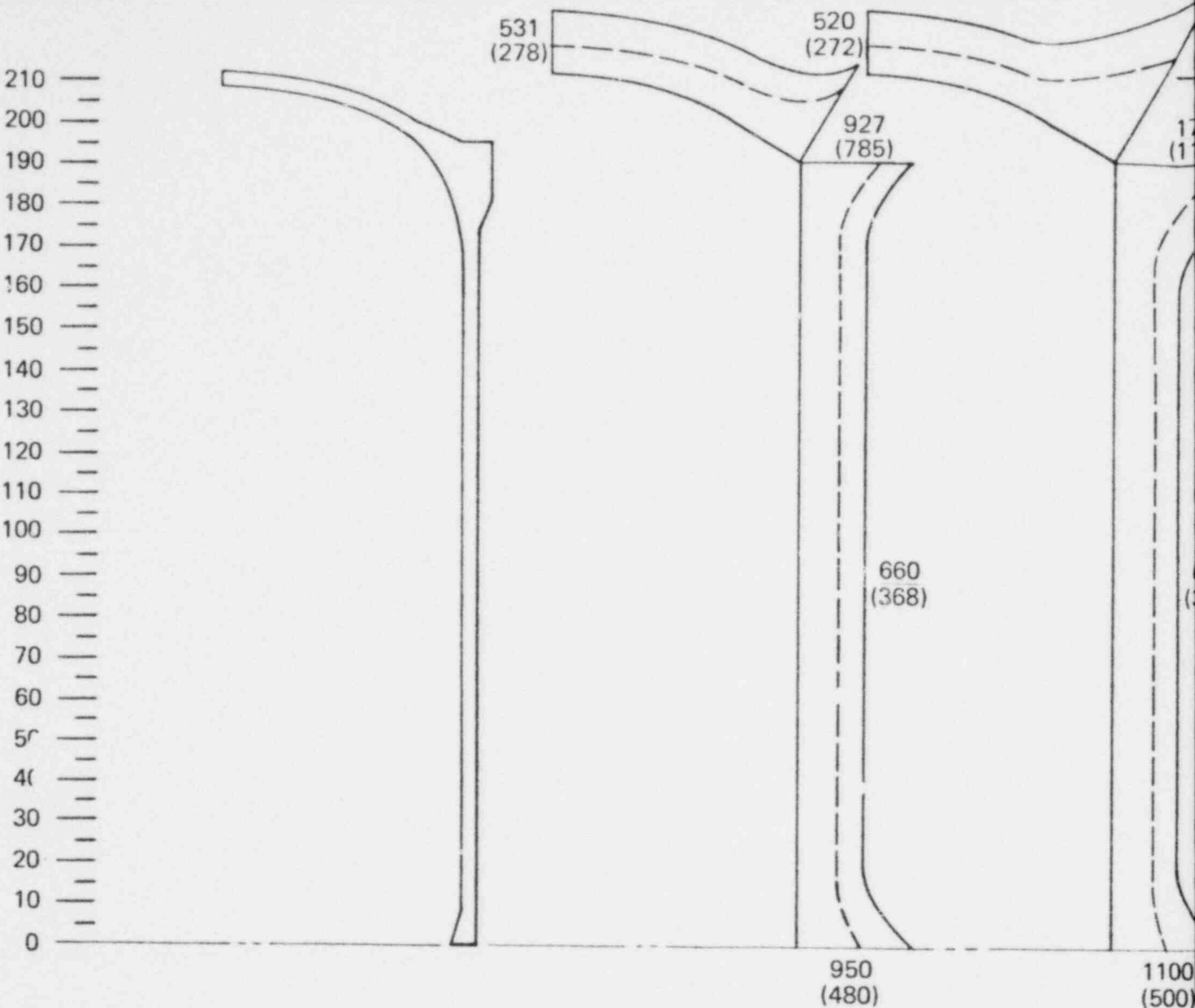
Amendment



MERIDIONAL

MOMENT $\left(\frac{Ft\ K}{Ft}\right)$

HOOP

MOMENT $\left(\frac{Ft\ K}{Ft}\right)$

NOTE:
MOMENTS SHOWN ON TENSION SIDE.

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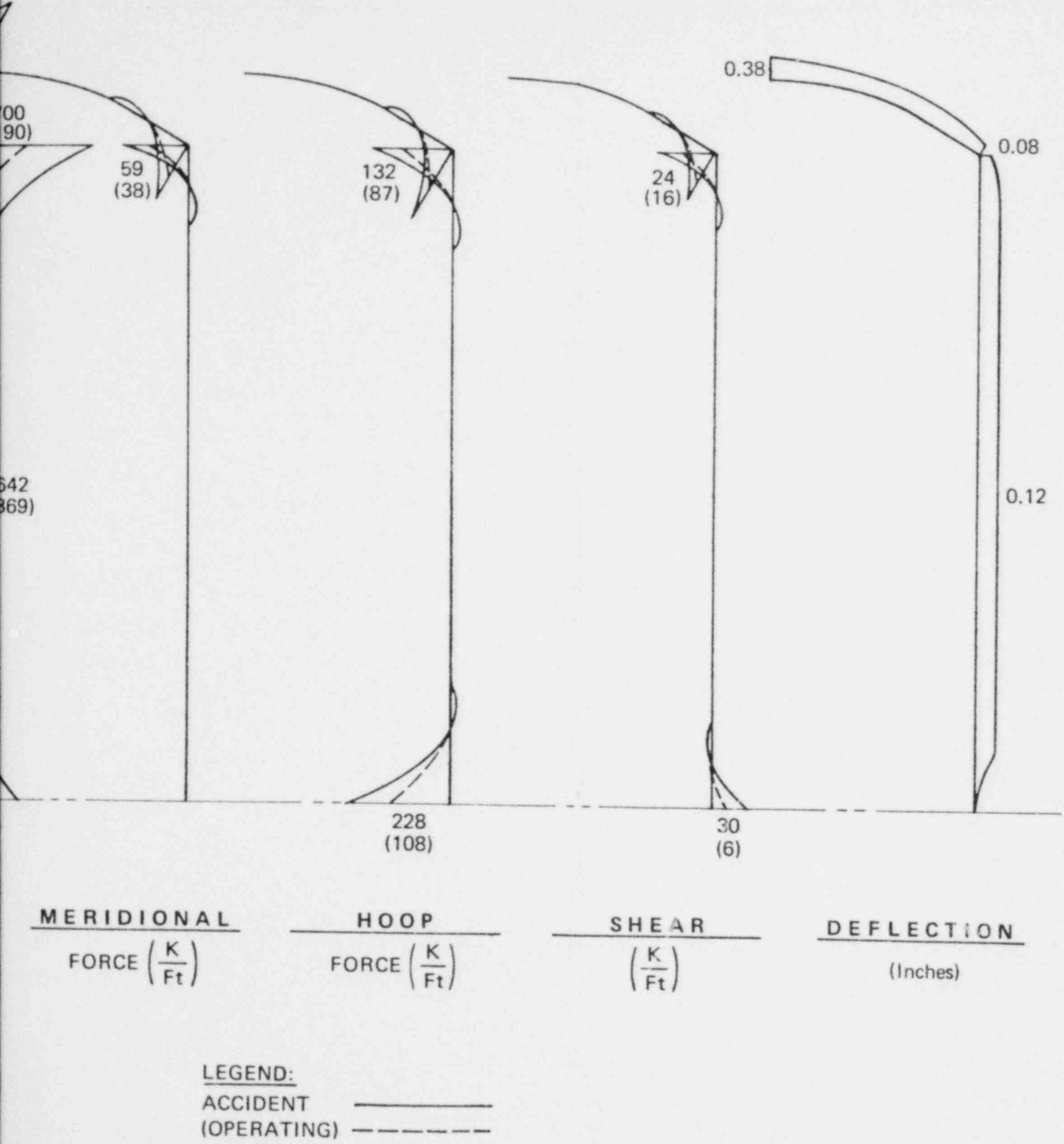

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FIGURE 5J.7.2-4
CONTAINMENT STRUCTURE STRESS ANALYSIS
PRELIMINARY MANUAL CALCULATIONS
THERMAL LOAD (T)

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MERIDIONAL
MOMENT $\left(\frac{Ft\ K}{Ft}\right)$

HOOP
MOMENT $\left(\frac{Ft\ K}{Ft}\right)$

NOTE:
MOMENTS SHOWN ON TENSION SIDE.

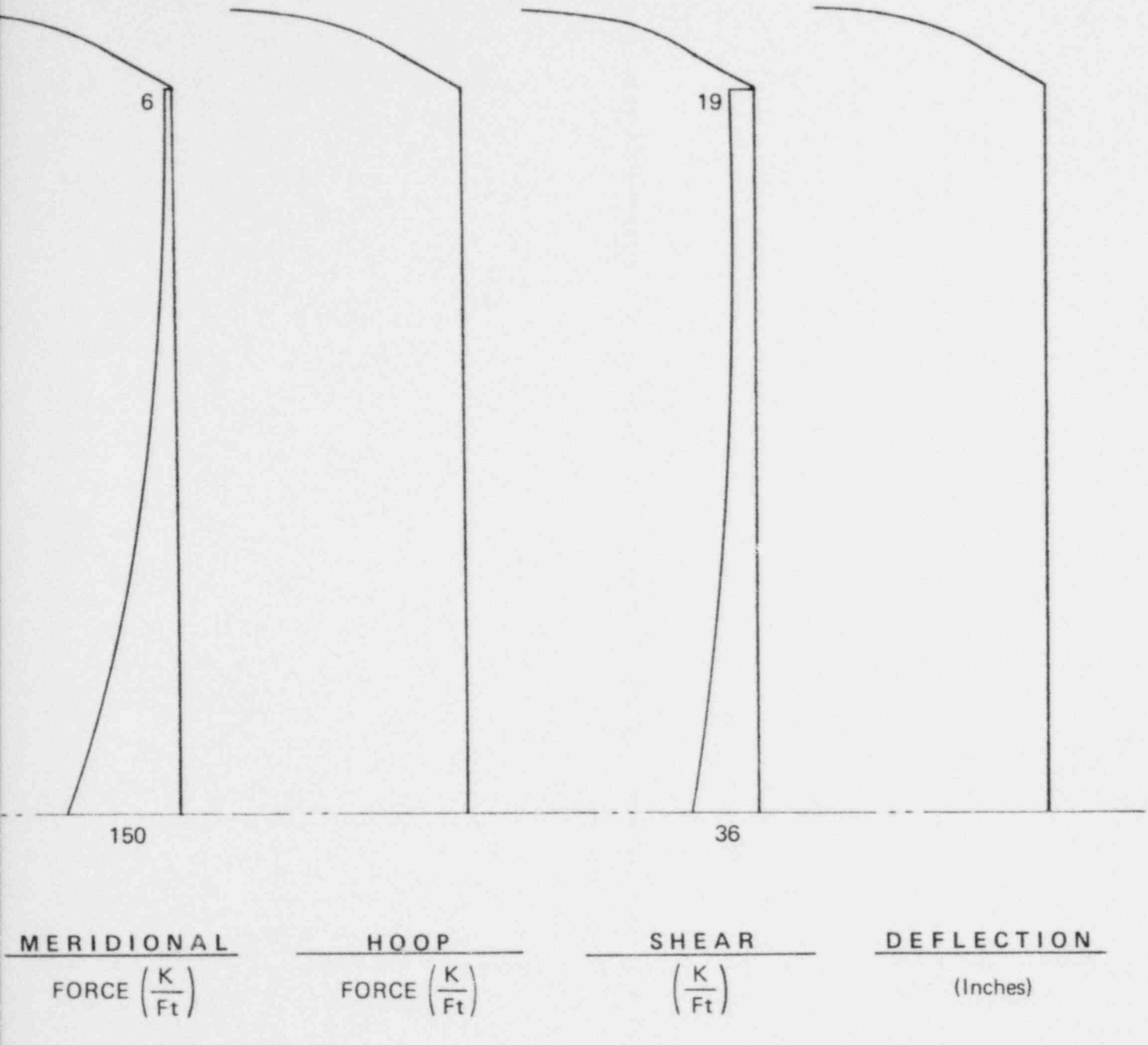

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FIGURE 5J.7.2-5
CONTAINMENT STRUCTURE STRESS ANALYSIS
PRELIMINARY MANUAL CALCULATIONS
EARTHQUAKE LOAD (E)

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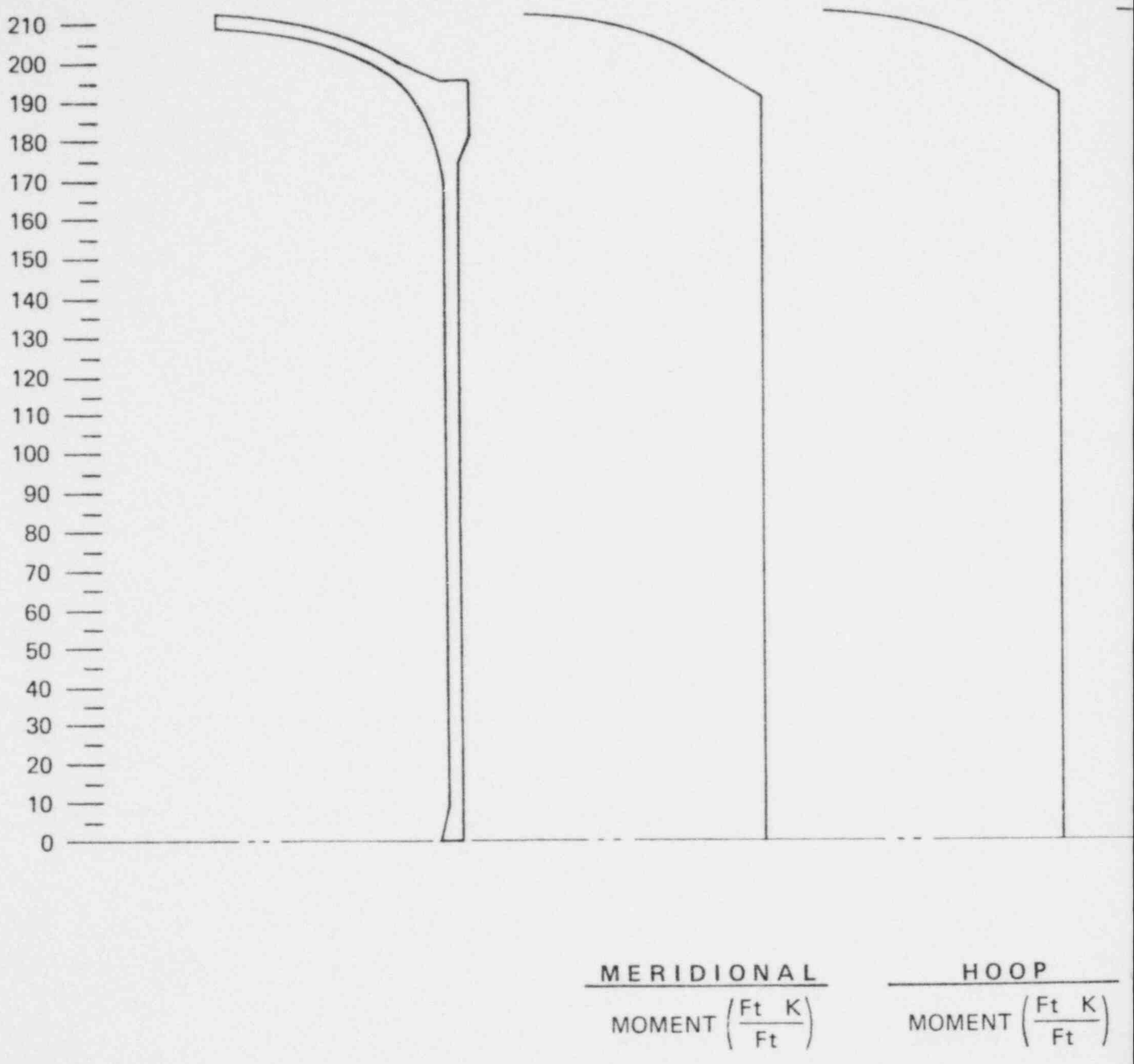

NOTE:
MOMENTS SHOWN ON TENSION SIDE.

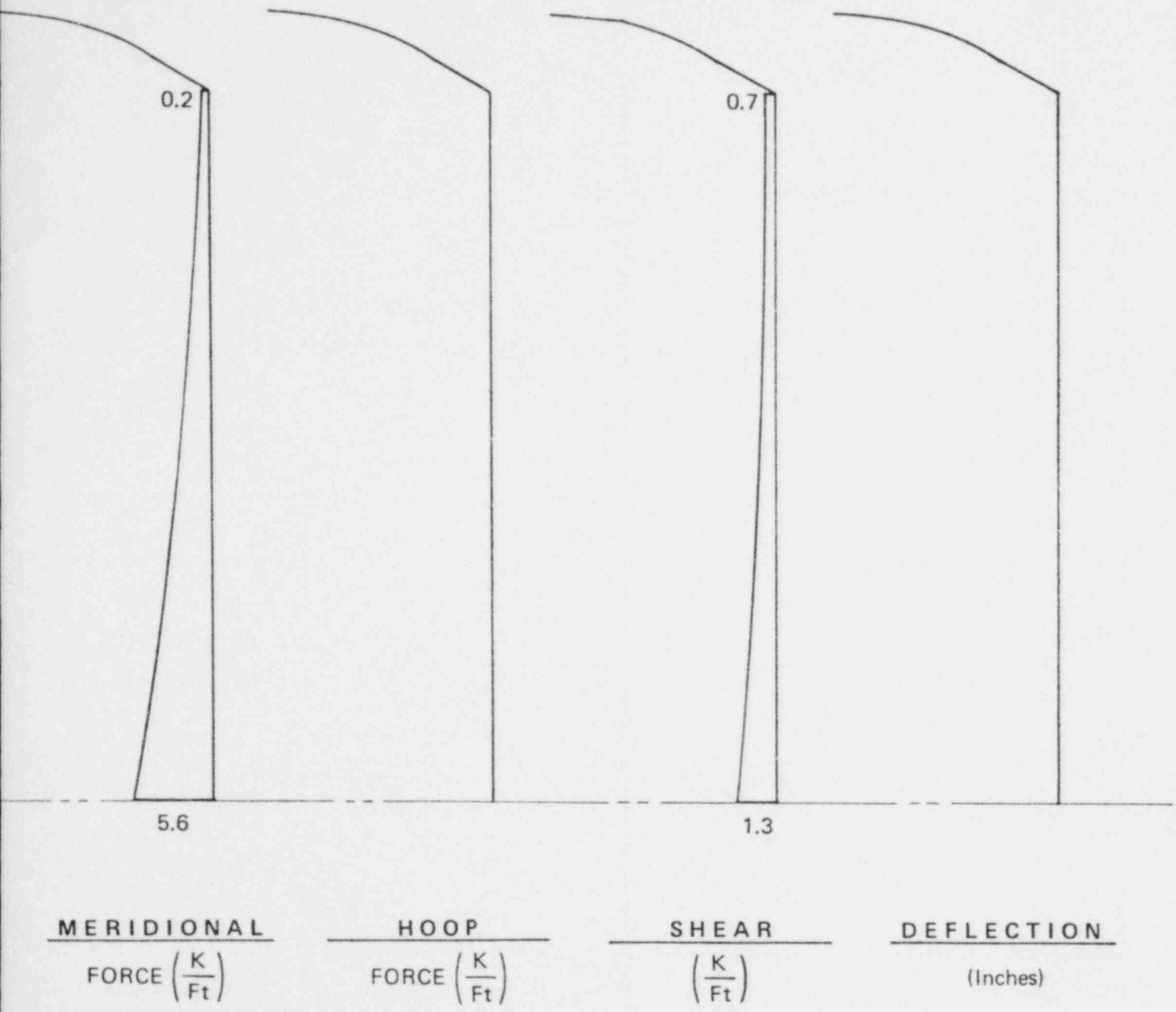

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FIGURE 5J.7.2-6
CONTAINMENT STRUCTURE STRESS ANALYSIS
PRELIMINARY MANUAL CALCULATIONS
WIND LOAD (W)

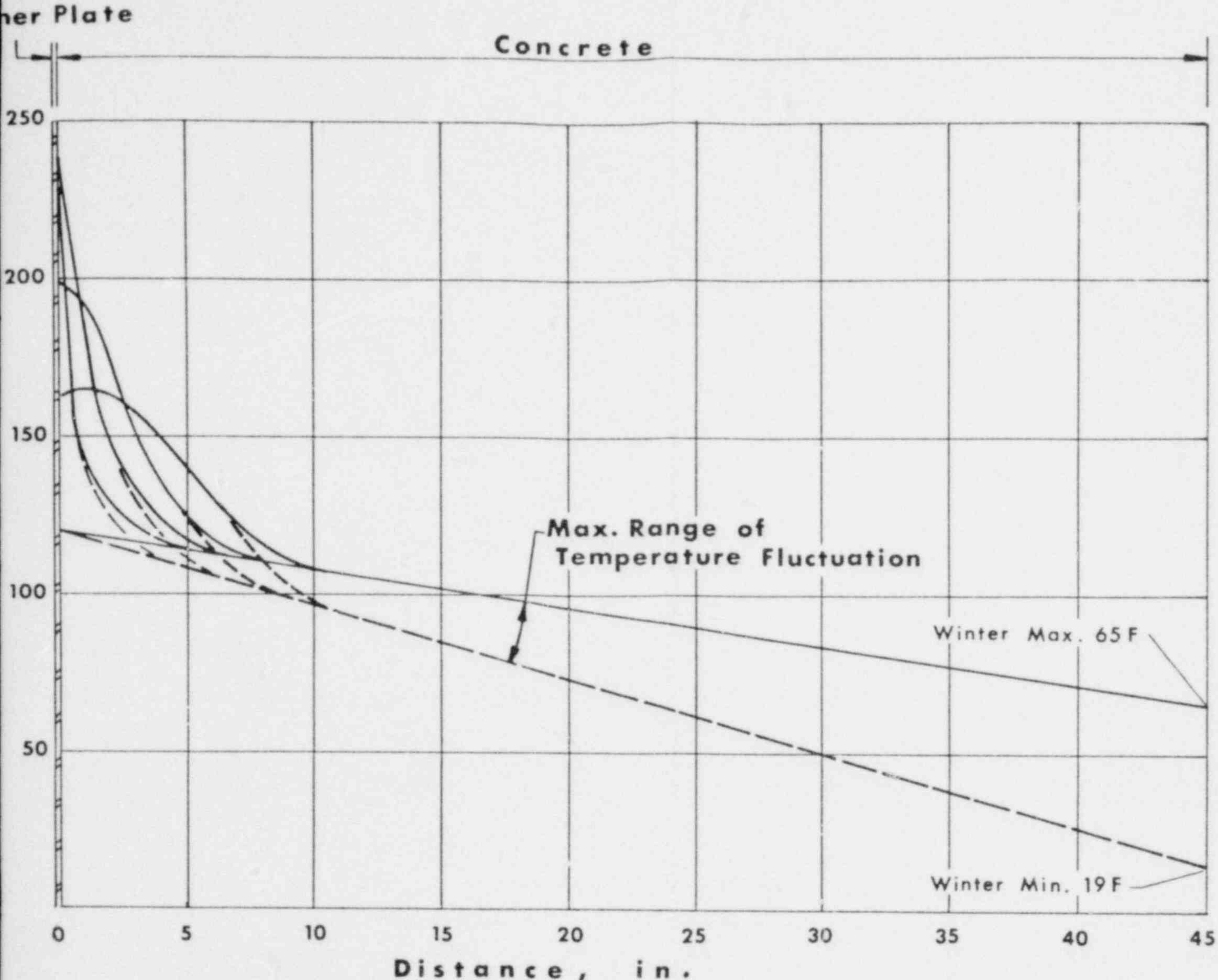

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FIGURE 5J.7.2-7
WINTER THERMAL GRADIENT
ACROSS CONTAINMENT WALL

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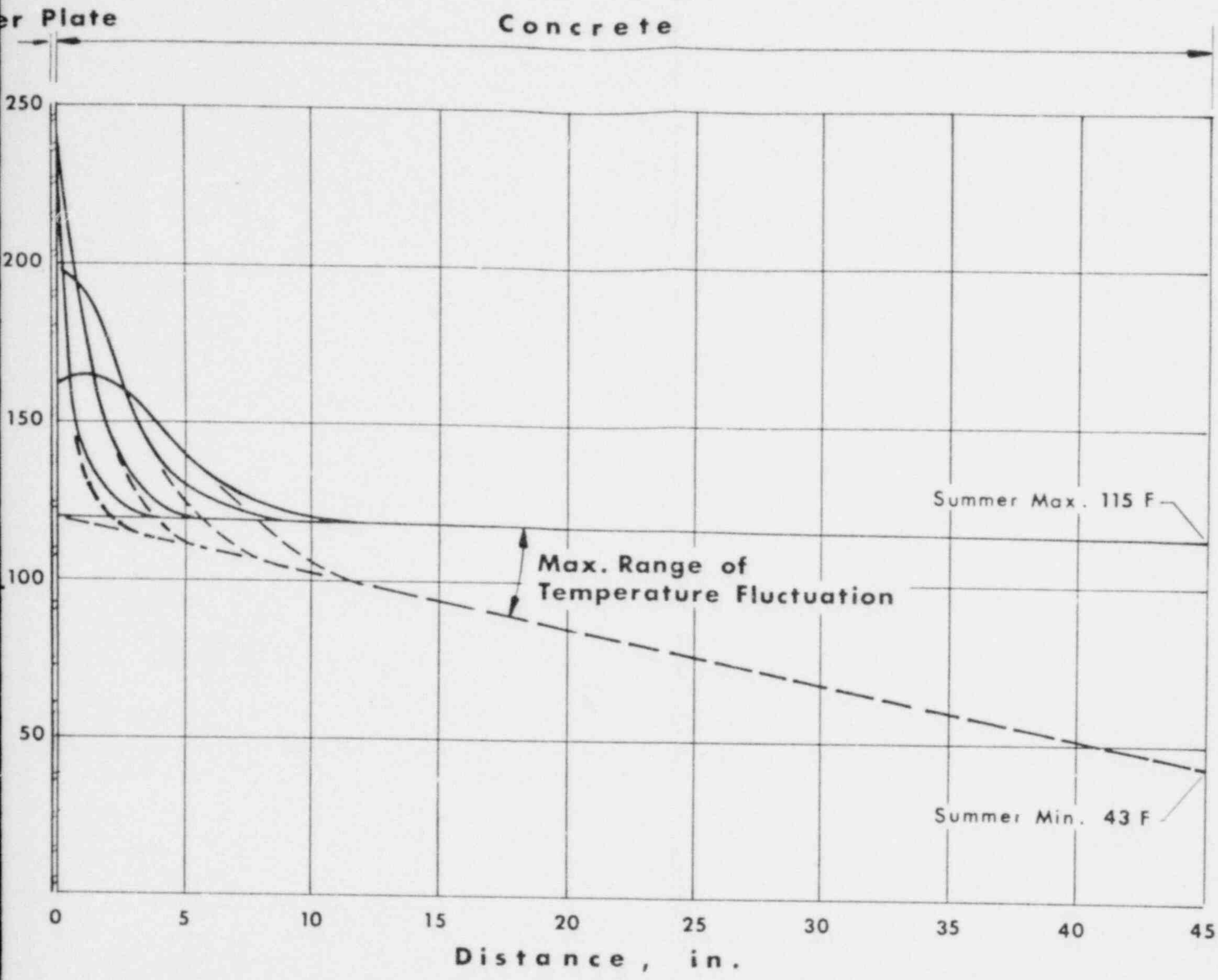

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FIGURE 5J.7.2-8
SUMMER THERMAL GRADIENT
ACROSS CONTAINMENT WALL

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AMENDMENT 3

5J.7.3 (DRL 5.2.3)

The thermal load from the liner is a function of the stiffness of the encasing concrete and its deformations. It is therefore necessary to define and to justify the values of the Young's modulus E_c and of the Poisson's ratio μ_c for cracked and uncracked reinforced concrete structure. List the values of E_c and μ_c for different elevations and explain their use in the design of the concrete shell and in thermal liner loading computations. Include the effect of shrinkage and creep.

ANSWER

Recognition is given to the importance of the effect of the Young's modulus E_c and of the Poisson's ratio μ_c in the prediction of thermal loads on the concrete from the liner plate. Various tests have been carried out to determine E_c and μ_c for uncracked concrete.

For operating condition analyses, using working stress design methods, the concrete is assumed uncracked. The assumed values of E_c and μ_c will be confirmed by laboratory tests of actual construction materials and mixes. |4
In arriving at the above mentioned value of E_c, the effect of creep is included by using the equation

$$E_{cs} = E_{ci} \frac{\epsilon_i}{\epsilon_s + \epsilon_i}$$

(where E_{cs} and E_{ci} are sustained and instantaneous modulii of elasticity of concrete respectively) for long term loads such as thermal load, dead load and prestress. Figure 5J.7.3-1 shows the relationship of instantaneous and sustained strain which is used to arrive at the appropriate E_c. When the effect of creep and shrinkage is included, the value of sustained modulus of elasticity of concrete is about one half the value of instantaneous modulus of elasticity. No modification is made of μ_c for instantaneous or sustained loading.

For accident condition analyses, using working stress design methods, cracking of concrete at the outside face is expected. It is very difficult to define E_c and μ_c for cracked concrete since it depends on factors like type, amount and location of cracking. However, the above mentioned value of elasticity of concrete, E_{cs}, will be used together with the method described in ACI Code 505-54 to find the stresses in concrete, reinforcing steel and liner plate from the predicted accident |4

For the yield stress design, essentially when all of the prestress is removed from the concrete, the thermal stresses in the liner plate and reinforcing steel are found assuming fully cracked section, using the methods of ACI-505. For this case, E_{cs} and $\mu_c = 0$.

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The same value of E_c and μ_c will be used throughout the cylinder wall and dome.

The minimum reinforcing provided for thermal stress cracking is 0.15 percent of the gross concrete area and it is located in two mutually perpendicular directions near the outer concrete surface.

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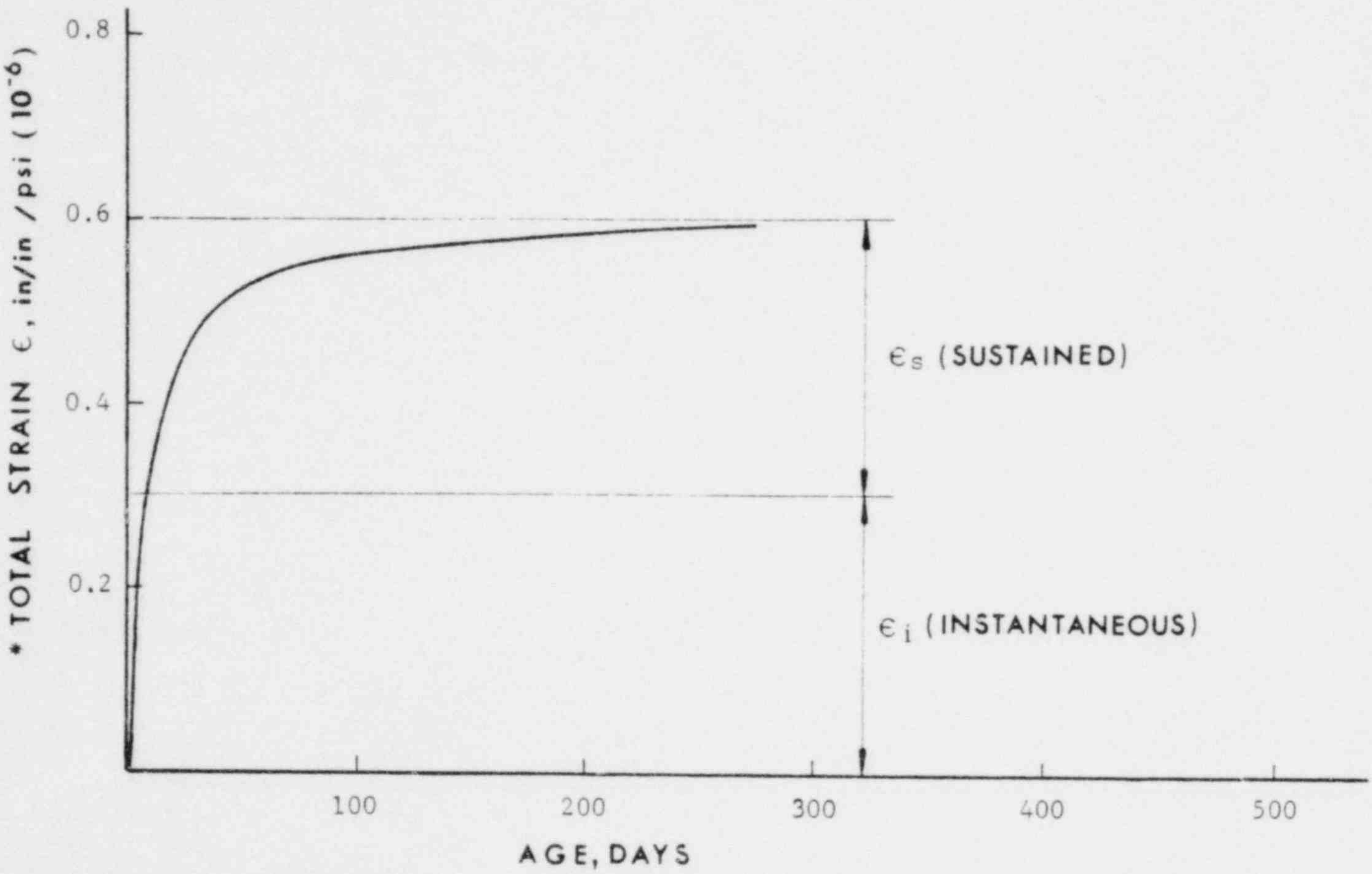

* AVERAGE VALUES ARE SHOWN. ACTUAL VALUES WILL BE DETERMINED BY TESTING

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FIGURE 5J.7.3-1
LONG TERM EFFECT
CREEP AND SHRINKAGE

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Amendment 3

5J.7.4 (DRL 5.2.4)

The thermal load from the liner is also a function of the thickness of the liner plates, and of the yield point of the liner steel. The thickness of the two adjacent liner plates may vary by as much as 10%. In addition, only the minimum yield point is indicated in PSAR, but not the maximum yield point, which may differ from the minimum by as much as 25% - 30%. Explain how the variations of thickness and yield point are considered in the design.

ANSWER

The most critical condition for a liner plate exists when the liner plate is in the condition illustrated by Figure 5J.7.4-1. In this condition Panel 1 and Panel 3 have outward initial curvature and Panel 2 has inward initial curvature. When a load is applied to the liner plate Panel 1 and 3 will bear against the concrete and Panel 2 deforms inward. If the load is primarily from concrete shrinkage, creep, prestress and thermal effects the membrane stress $(\frac{N}{t})$ in Panels 1 and 3 will tend to relax to a value of $(\frac{N-\Delta N}{t})$ in Panel 2. The anchors between the panels with inward and outward curvature must restrain a force of ΔN for static equilibrium. Due to inward deformation, flexural stress also exists in Panel 2 and the anchors are subjected to the moment (M).

Due to the fact that all of the significant compressive liner plate loads are self limiting i.e., as deformations occur, the loads tend to reduce, the liner may deform inward but can never get into an unstable condition provided it is sufficiently restrained.

The only significance of the mathematically calculated elastic buckling stress (σ_{CR}) is as follows: If a panel is perfectly straight and (σ_{CR}) is lower than the yield stress (σ_y) then when (σ_{CR}) is reached this panel will tend to deform inward if an internal pressure is not present. When a panel has initial inward curvature the rate of change of inward deflection with respect to membrane stress will increase after (σ_{CR}) is exceeded, but the panel will remain stable. If a perfectly straight panel has (σ_{CR}) higher than (σ_y) then the panel will deform inward when (σ_y) is reached.

The anchor detail has the capability of resisting the full force (ΔN) due to a theoretically fixed anchor, but in addition, it has sufficiently ductility to accept the .038" displacement without failure. The above displacement results from a uniform membrane strain of .0025 in/in distributed over a 15(in) anchor spacing. Various patterns of welds attaching the angle anchors to the liner plate have been tested for ductility and strength when subjected to a transverse shear load such as ΔN and are shown in Figure 5J.7.4-2.

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The following conditions and their effects will be considered in the liner plate design:

(1) Panels 1 and 3 are thicker than Panel 2, the force across the anchor will increase, the thickness variation for a plate can be obtained from "ASTM-Part 4 January 1967, Specification A-442-66 and Specification A-20-66" which states that the tolerance is +16% and -4% for 1/4" thick plate.

(2) An increase in yield strength of the liner plate material will increase the force across the anchor. The information will be obtained from the "Mill Test Reports." For preliminary design values a 25% increase should give adequate protection will result in the yield stress increasing from 32 ksi to 40 ksi.

(3) A variation of the modulus of elasticity and Poisson's ratio does not appreciably affect the design or the margins available due to the ductility of the anchorage system. The design will give adequate protection with respect to this variation.

Erection and fabrication inaccuracies are controlled by specified tolerances given in Section 5.4.3.5 including Amendment 1. These values are also considered in the design. By keeping the anchor spacing small the amount of force ΔN and the inward displacement may be controlled. A maximum initial inward displacement between the anchors of 1/8" is also specified and controlled in the field. This value is used in the design since it is important in controlling the amount of inward displacement and the amount of relaxation in Panel 2.

Offsets at liner plate seams are controlled in accordance with ASME Section III Code, which allows 1/16" misalignment for 1/4" plate. The flexural strains due to the moment (M) are added to calculate the total strain in the liner plate.

The liner plate will be anchored as shown in Figure 5.1-1 (Amendment 3) with anchorage in both the longitudinal and hoop direction, however, the anchor spacing will be set by the designer for the appropriate plant design basis.

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The load combinations given in Section 5.1.4 of the PSAR will be applicable to the liner plate and anchorage design together with the effects shrinkage and creep of concrete and vacuum loads. The penetration assemblies will be designed to accommodate all of the above loads where applicable and also the effects of stress concentrations.

The design details will be made available for review
4 when the design is completed, which will be by February 1969.

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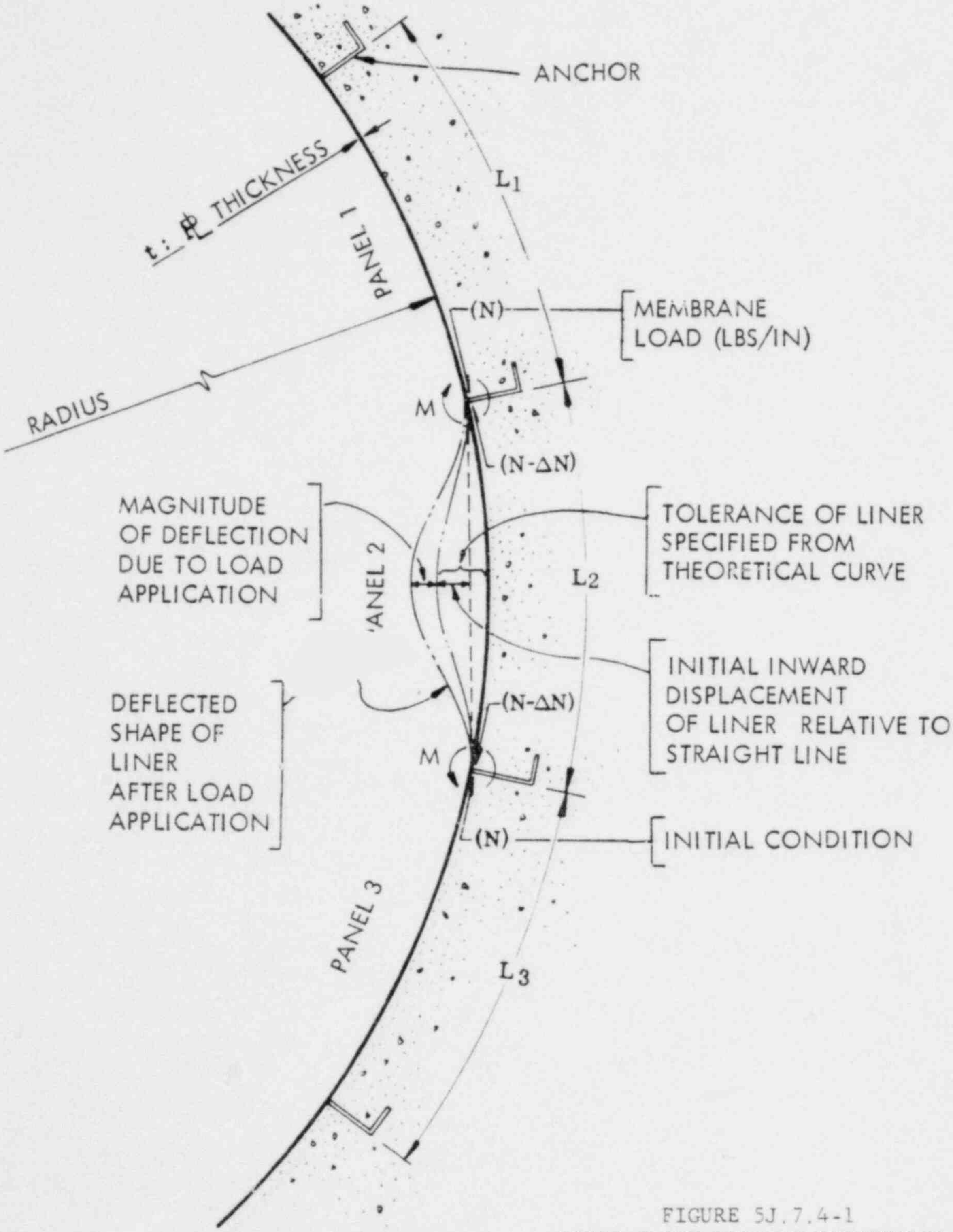
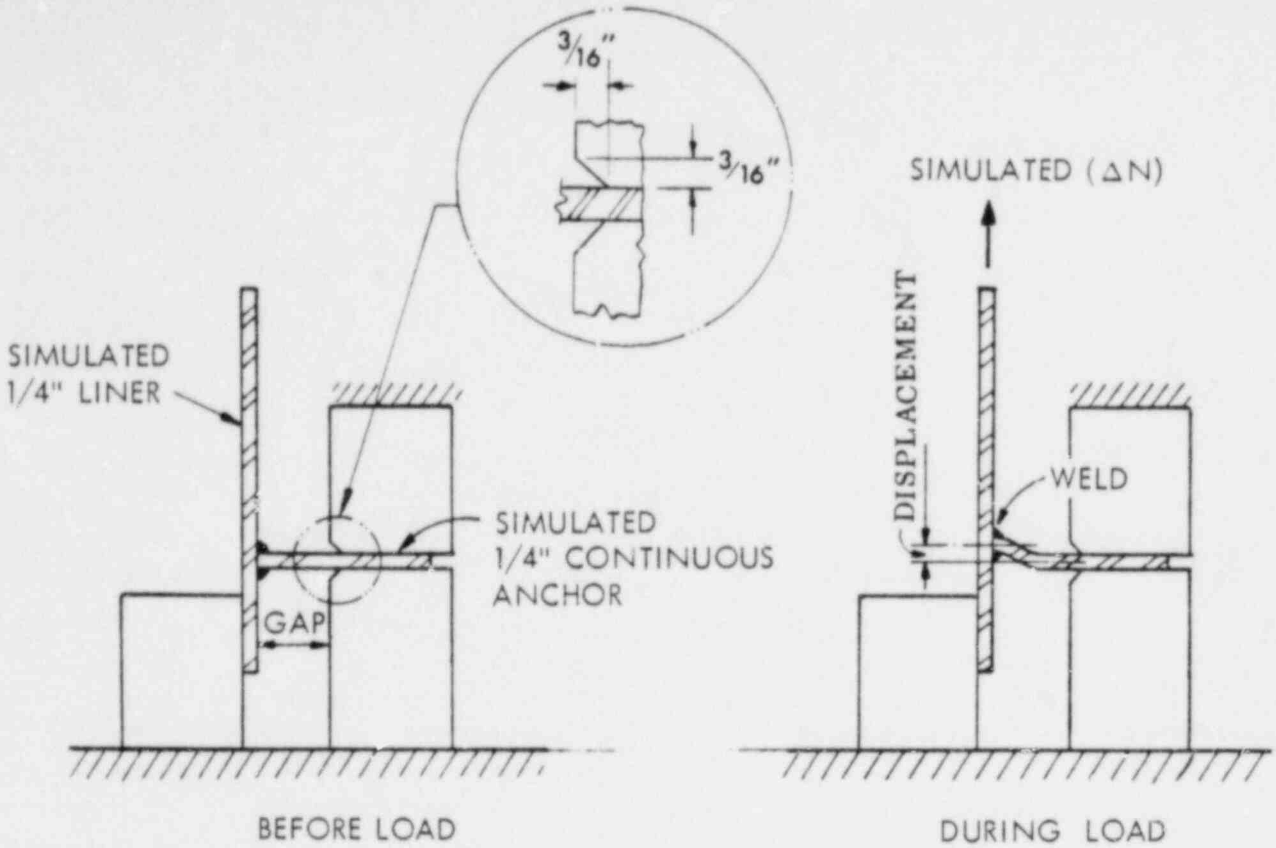


FIGURE 5J.7.4-1
LINER PLATE LOADING CONDITIONS



WELD CONFIGURATION	GAP (IN)	ULTIMATE LOAD (K/IN)	ULTIMATE DISPLACEMENT (IN)	LOCATION OF FAILURE
3/16	0	14.95	.14	LINER PLATE
3/16	5/8	5.56	.68	ANCHOR WELD
3/16 6-12	0	7.65	.18	ANCHOR WELD
3/16 6-12	5/8	2.93	.60	ANCHOR WELD
3/16 4-12	0	6.67	.18	ANCHOR WELD
3/16 4-12	5/8	2.46	.30	ANCHOR WELD

FIGURE 5J.7.4-2
ANGLE ANCHOR WELDS TEST RESULTS

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5J.7.5 (DRL 5.2.5) For the loadings of the containment structure wall and dome, describe:

(a) The analytical procedures used for arriving at the forces, shears and moments in the structural shell, considering that the structure is not axisymetric (buttresses);

(b) The considerations given to, and the analytical procedures for determining discontinuity stresses at the base, at the dome (ring girder), and at the buttresses; the assumptions with regard to structural stiffness that form the basis for these stress determinations; the variations of E_c and μ_c considered.

ANSWER

(a) An analysis is performed using a finite element computer program, assuming axisymmetric shape and loadings and elastic material properties that are determined by tests of specimens from the concrete mix to be used. The effects of the buttresses are not axisymmetric, however, the local effects are predicted by a two dimensional finite element program considering plane strain with loads acting in the plane of the coordinate system.

At each buttress the hoop tendons are alternately either continuous or spliced by being mutually anchored on the opposite faces of the buttress. Between the opposite anchorages the compressive force exerted by the spliced tendon is twice as much as elsewhere, therefore, this increased value added to the effects of the tendon which is not spliced, will be 1.5 times larger than the prestressing force acting outside of the buttresses. The cross sectional area of the buttress is about 1.5 times that of the wall, so the hoop stress as well as the hoop strains and radial displacements can be considered as being nearly constant all around the structure. An isostress plot made by the plain strain analysis, referred to Appendix 5G of the PSAR, is shown in Figures 5J.7.5-1, 5J.7.5-2, and 5J.7.5-3 and confirms the previous statement. The vertical stresses and strains, caused by the vertical post-tensioning become constant at a short distance away from the anchorages because of the large stiffness of the cylindrical shell. Since, as stated above, the stresses and strains remain nearly axisymmetric despite the presence of the buttresses, their effect on the overall analysis is negligible, when the structure is loaded with dead load or prestressing loads.

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When an increasing internal pressure acts upon the structure, combined with a thermal gradient such as at the design accident condition, the resultant forces being axisymmetric, the stiffness variation caused by the buttresses will be decreased as the concrete develops cracks. The structure will then tend to shape itself to even more closely follow the direction of the acting axisymmetric resultant forces.

The buttress effect is more axisymmetric at yield loads, which include factored pressure, than at design loads including pressure. This fact, combined with the redundancy of the pressure resisting structural elements, indicates that the buttresses will not reduce the margins of safety available in the structure.

(b) Discontinuity stresses at the base and at the dome ring girder are predicted by the axisymmetric finite element analysis for the axisymmetric loadings and they are independently determined for the earthquake loading. In this latter case the relative stiffness of the cylinder and the base slab is determined by the finite element program, then the base slab is analyzed as a circular plate. Moments are distributed to the slab and cylinder according to their relative stiffness. The consideration of variations of the E_c and μ_c is discussed in the answer to Question 5J.7.3.

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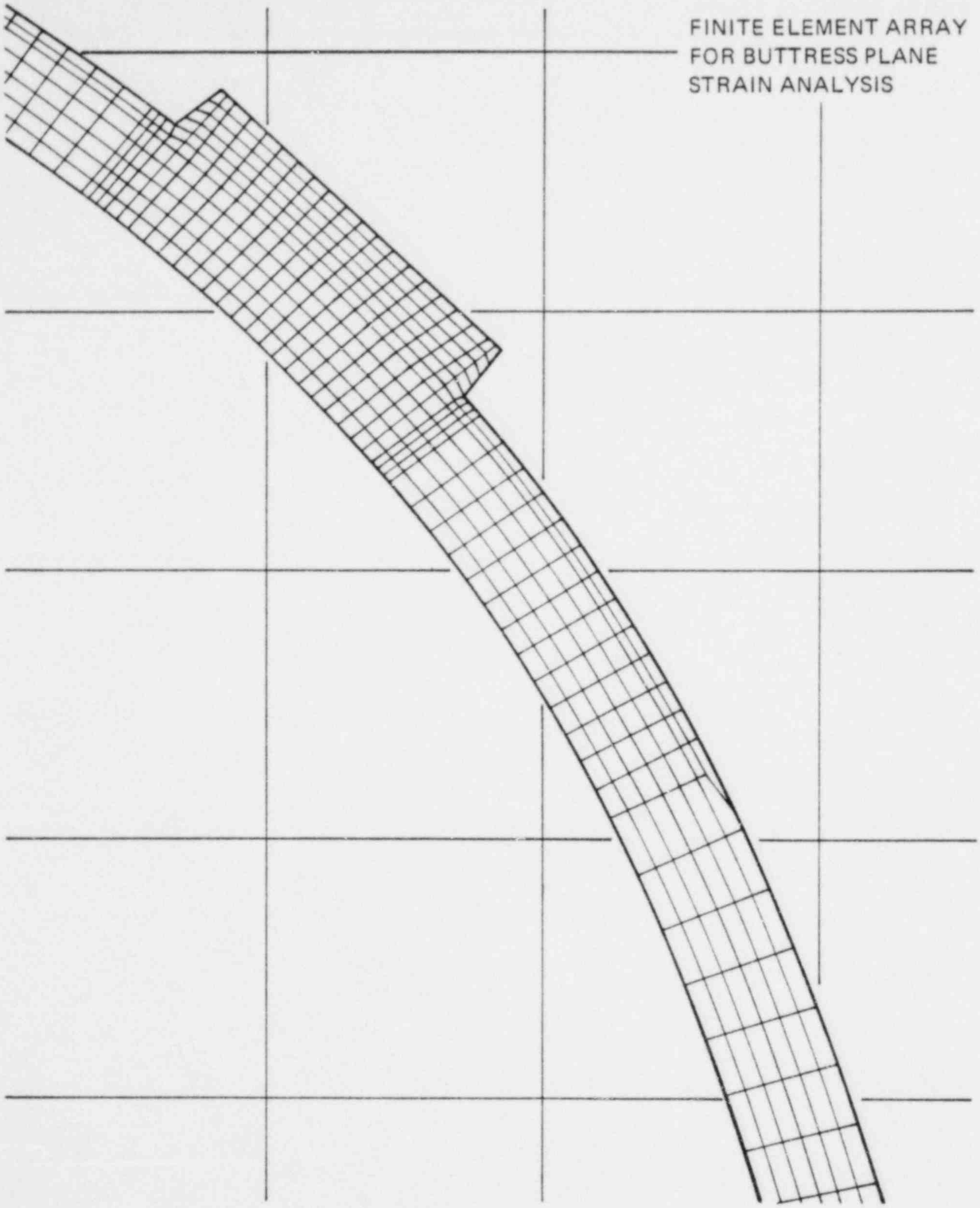

FIGURE 5J.7.5-1
STRAIN ANALYSIS

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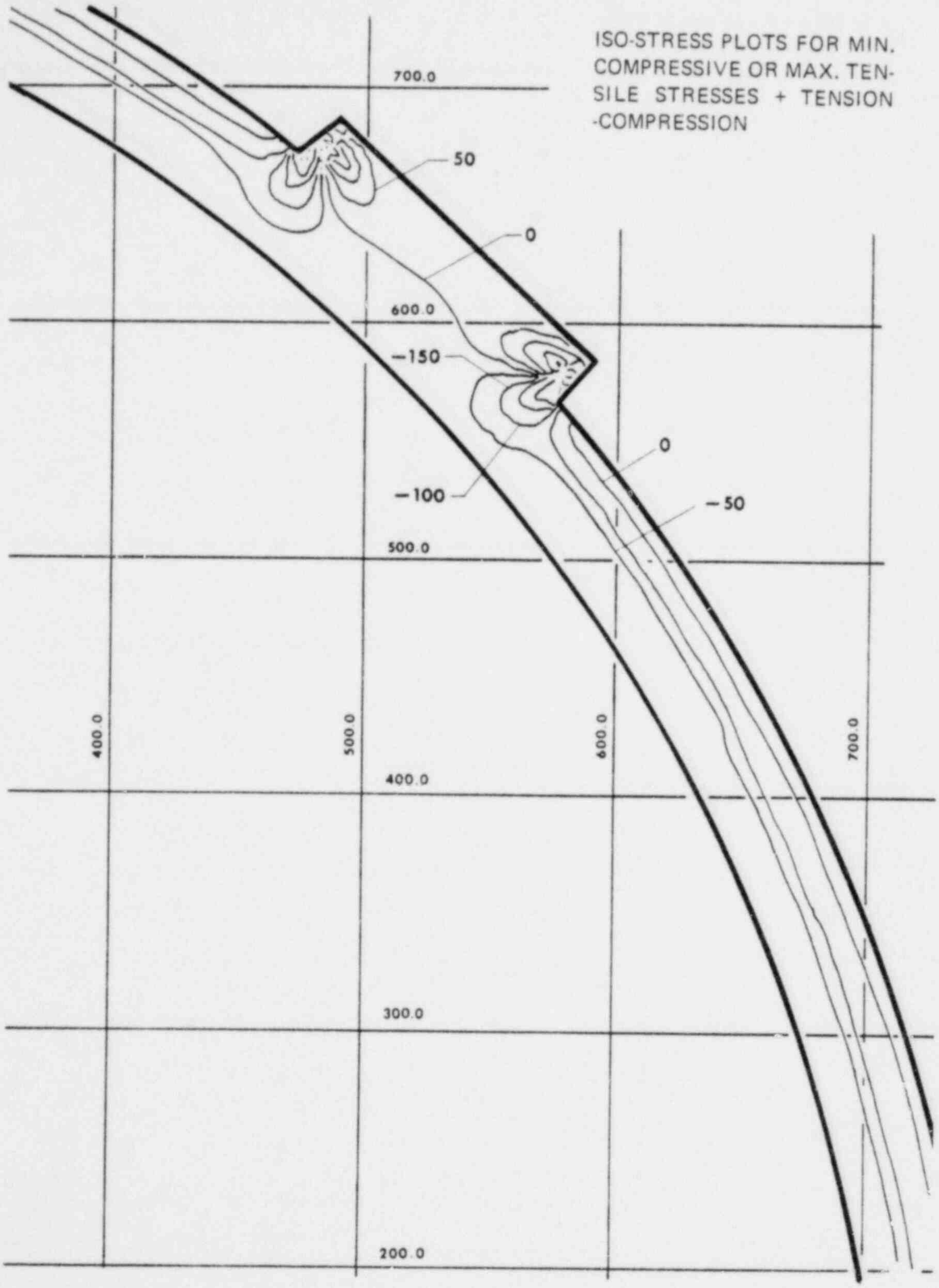

FIGURE 5J.7.5-2
MAXIMUM TENSION

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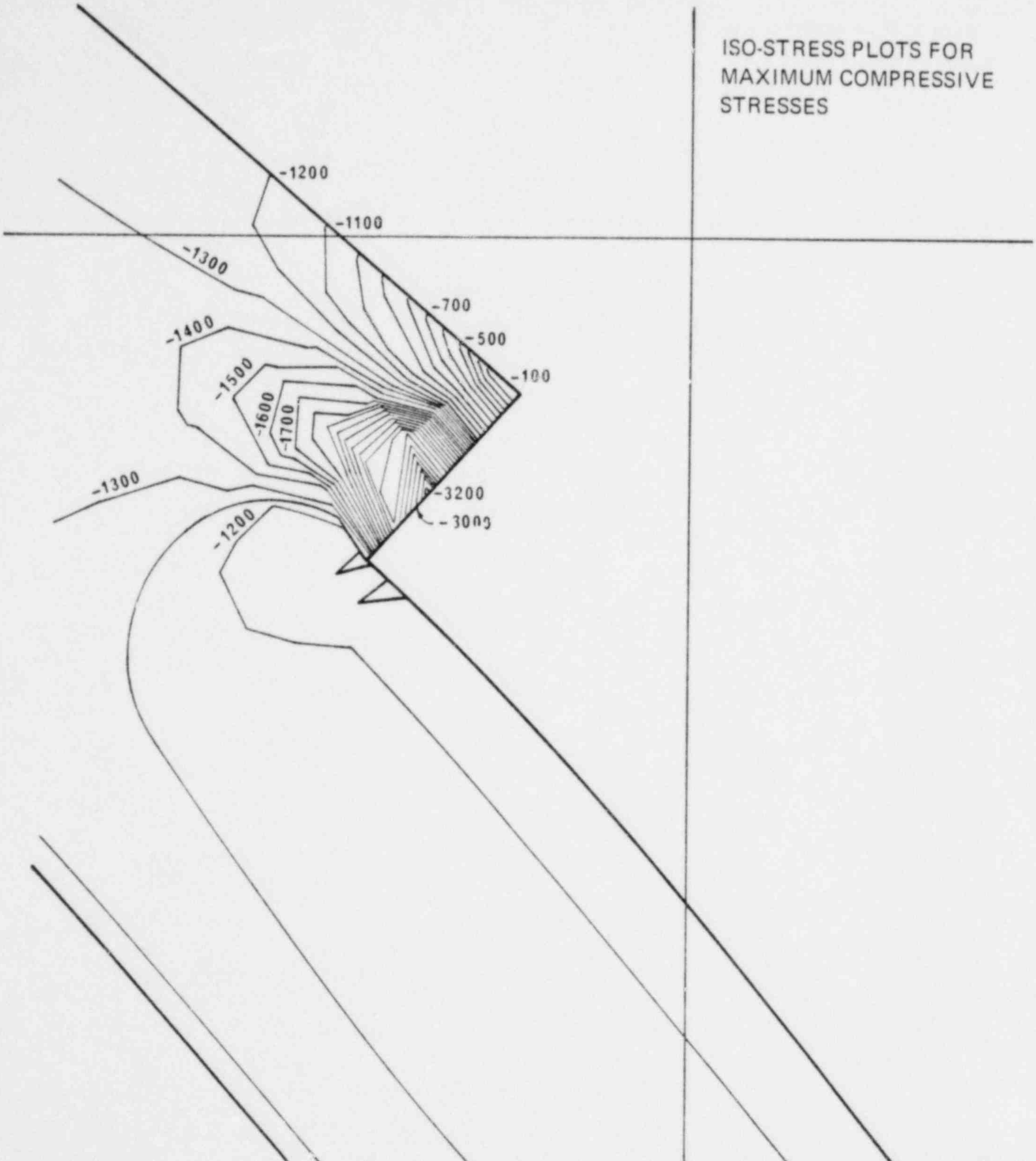

FIGURE 5J.7.5-3
MAXIMUM COMPRESSION

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Amendment 3

5J.7.6 (DRL 5.2.6)

It is not clear whether the computer program which is used for the design will take into account the cracking of concrete, and the resulting variation of E_c and μ_c. Should not the program also be able to handle loadings that are not axisymetric which act on structures that are not axisymetric?

ANSWER

The finite element computer program used does not take into account the cracking of concrete. Another form of the finite element program handles uncracked axisymmetric structure with nonaxisymmetric loading and is available at the University of California. It has been used in the approximate analysis for the equipment hatch area.

No three dimensional finite element analysis is applied as of now in the containment design. However, the combination of the several methods employed in analyzing the problem areas results in a safe design.

5J.7.7 (DRL 5.2.7)

If the effect of temperature rise in the liner will be represented by a uniform pressure increase, provide a justification for this approach.

ANSWER

We do not consider the temperature rise in the liner as a uniform pressure increase. Equal expansion forces in both the circumferential and vertical direction are used.

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5J.7.8
(DRL 5.2.8)

Indicate whether the following has been considered:

(a) Possible reversal of stresses, due to creep during cold shut down:
(b) Cracking of the cylindrical wall, which makes it more flexible than the uncracked mat;
(c) Capacity of the ground around to restrain deformations of the wall.

ANSWER

(a) The possible reversal of stresses due to creep during cold shut down is being considered.
(b) The finite element analysis does not take into account the decreased stiffness of the cracked concrete sections.
(c) The soil supporting or surrounding the structure is also analyzed by the finite element method, its restraining effects are therefore included.

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5J.7.9 (DRL 5.2.9) For the loadings of the base slab, describe the analytical procedures used to arrive at the forces, moments, and shears, considering loading that is not axisymmetric and deformations of the mat. State whether you considered transient thermal gradients.

ANSWER The structural requirements of the base slab are dictated by the loading conditions imparted by normal plant operation and by postulated accident conditions. These loadings are both of an axisymmetric and asymmetric nature, and it becomes a distinct analytical advantage to separate the two for structural design.

The axisymmetric loads are handled in the finite element computer analysis performed for all loading conditions on the entire containment shell. This program takes full recognition of the elastic properties of the slab as well as those of the surrounding soil. This permits an accurate estimate of moments, forces, and shears, including steady state and transient thermal effects.

The non-axisymmetric loads at the present time are out of the range of the existing finite element computer program used by the designers. As such, the effects of these loadings are handled separately. This portion of the analysis, however, is handled using the "beams on elastic foundations" approach, which considers the elastic properties of the soil beneath the mat as well as the accompanying soil deformations. In general, it has been found that effect of the interior structure does not invalidate the axisymmetric analysis of the mat near the junction of the cylinder and the base slab. Small localized areas where this may occur are reinforced to insure complete structural integrity under all loading conditions, both operational and factored. Complete deformational compliance is maintained throughout the entire analysis. Although in general the foundation is relatively insensitive to variations in soil properties, a number of soil conditions will be checked to insure that an overall conservative approach is being used.

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5J.7.10 (DRL 5.2.10) What were the elastic properties of the soil used for design of the mat?

ANSWER The following preliminary elastic soil properties are representative of those which will be used for design of the containment building foundation mat:

a. Soil Modulus of Elasticity E_s = 100,000 psi.

b. Poison's ratio μ = 0.30.

These values are based upon the soils and foundations investigation report included in Appendix 2F.

Variations in the above values will be investigated to insure the values used are reasonable.

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QUESTION 5J.7.11 (DRL 5.2.11)

Provide some clarification of the design procedures and stress limits by describing the extent to which liner participation is relied upon to provide resistance to lateral (earthquake) shear. If liner participation is not included, describe how the corresponding strains are transmitted to the liner and their effect on the liner. Consider possible cracking of concrete.

ANSWER

There are no design conditions in which the liner plate is relied upon to assist the concrete in maintaining integrity of the structure even though the liner will, at times, provide assistance in order to maintain deformation compatibility.

Loads are transmitted to the liner plate through the anchorage system and direct contact with the concrete and vice versa. Loads may be, at times, also transmitted by bond and or friction with the concrete. These loads cause, or are caused, by liner strain. The liner is designed to withstand the predicted strains.

Possible cracking of concrete has been considered and reinforcing steel will be provided to control the width and spacing of the cracks. In addition, the design is made such that total structural deformation remains small during the loading conditions and that any cracking will be orders of magnitude less than sustained in repeated attempts to fail Model 1, and even smaller than the concrete strains of overpressure tests of Model 2 (Both at General Atomic Ref. 1 and Ref. 2.)

As described, the structural integrity consequences of concrete cracking are limited by the bonded reinforcing and unbonded tendons provided in accordance with the design criteria (PSAR Section 5.1.4.). The effect, of concrete cracking, on the liner plate has also been considered. The anchor spacing and other design criteria are such that the liner will sustain orders of magnitude of strain, for example, less than did the liner of Model 1 at General Atomic (Ref. 1) without tensile failure.

References

1. "Prestressed Concrete Reactor Vessel, Model 1," GA7097, H.T.G.R. and Laboratory Staff.

2. "Prestressed Concrete Reactor Vessel, Model 2," GA7150, Advance H.T.G.R. Staff.

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QUESTION 5J.7.12 (DRL 5.2.12)	Explain whether one-third increase in allowable stresses will be used. This increase in allowable stresses is not considered in keeping with its usage in normal practice, particularly with respect to the D + L + S + T loading. Discuss this problem and provide a criterion that considers biaxial and triaxial loading effects. Justify the values of shear (as a measure of beam strength in diagonal tension) for a structure of this type. Discuss your design criteria in this area, keeping in mind possible biaxial tension stresses, and two-dimensional cracking.
ANSWER	Stress limitations, as mentioned under "Design Loads" and "Loads Necessary to Cause Structural Yielding" in the criteria discussed in PSAR under Section 5.1.4 (Reference 1), will be used in the design.

The allowable concrete stresses, of 0.30 f_c' for membrane compression and 0.60 f_c' for flexural compression combined with membrane compression as mentioned under design loads (Reference 1), have not been established on the basis of an arbitrary one third increase of the ACI Code allowable stresses. They were established on the basis of what seems reasonable for a structure of this type which is subjected to a detailed analysis that also includes thermal loads.

In this design, the prestress is primarily used to induce membrane compressive stresses and it was necessary to place a limit on predicted concrete compressive stress that related to creep effects as well as strength capability. The value of 0.30 f_c' is considered a conservative limit for concrete compressive stress and was imposed to ensure that the membrane creep losses will be generally small and linear. The limit of 0.60 f_c' applies to combinations of predicted membrane and flexural compression when the thermal loads are included and in locations where the resultant creep would not have a significant effect on prestressing loads and structural integrity. These limits are considered to be conservative in view of the fact that the values of 0.60 f_c' and 0.30 f_c' bracket the code allowable value of 0.45 f_c' for compression in concrete and that the 0.60 f_c' is a code allowable stress which allows a one third increase for wind, earthquake, etc.

The particular load combination D + L + S + T is not considered under design loads. (See Reference 1)

The predicted state of stress in the containment is essentially biaxial, since radial stress is in general, of small magnitude.

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A proven criterion for allowable concrete biaxial and triaxial predicted stresses, higher than allowed by present code, cannot be given. Continuing work has been done by others, for defining the failure surface for concrete that is subjected to biaxial and triaxial states of stress, so that increased allowable stresses could be evolved by the use of suitable safety factors. However, none of the experimenters has, among other things, sufficient proof available to allow use of their work to supercede existing stress allowables for analyses which assume ideally elastic material. One major problem is that, although experimentation has supported experience in giving indication of a failure strength of concrete greater than f'c for biaxial and triaxial load conditions, the appropriate values for Young's modulus and Poisson's ratio are not available. Furthermore, the predicted and significant stresses for this containment are so low that criterion for higher allowable stresses is unnecessary.

Vertical cracks have little or practically no effect on transfer of radial shear (i.e., shear parallel to a radial plane.) Radial cracks, (cracks parallel to radius lines) are important in determining radial shear resisting capacity of the concrete section. Dr. Mattock, a Professor at the University of Washington, carried out an extensive testing program on reinforced concrete beams subjected to shear loads, as well as axial tension or compression, in order to determine the ultimate shear resisting capacity of concrete as indicated by diagonal cracking as well as web cracking. From the results of Dr. Mattock's tests, Formula 26-12 of ACI Code 318-63 has been modified for the containment design. So as to ensure proper interpretations of the tests as applied to containment structures, Dr. Mattock has been consulted and his work will be used as guidance in the containment design. Criteria for radial shear design as described in Section 5.1.4.6 has been revised to incorporate the comments made by Dr. Mattock.

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5J.7.13 (DRL 5.2.13) Under incident conditions concrete may be cracked and the crack pattern may be two-dimensional. Explain how, under this condition, the radial, vertical, and tangential shears are transferred through the section.

ANSWER Please refer to the answer to the Question 5J.7.12 for transfer of radial shear through the crack. For resisting membrane shear, please refer to the criteria described under Section 5.1.4.6 of PSAR. The effect of membrane shear will be to contribute to the membrane principal tension and cracking will occur when this principal tension stress becomes equal to the tensile strength of the concrete. An equal amount of reinforcing will be provided in each direction of the containment wall and dome so that the resistance to tension across the crack intersecting the reinforcing mesh will be independent of the angle at which the crack intersects the mesh.

5J.7.14 (DRL 5.2.14) The reinforcing steel may be stressed to the yield point. This stress is larger than the guaranteed minimum yield point of the liner which is 30,000 psi. Does this mean that, under certain conditions, the liner may be stressed beyond the yield point in shear? Clarify this point.

ANSWER: Under the transient maximum credible accident load condition, the liner plate is predicted to be beyond yield in compression in both the longitudinal and hoop direction. The above condition will also create a maximum principal shear stress which is also above yield. The main contributor to the yield condition in the liner plate is the thermal component of the maximum credible accident load conditions and this is considered in the analytical approach described in the answer to Question 5J.7.4.

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5J.7.15 (DRL 5.2.15)

Because of cracking of concrete due to shrinkage, to testing, to thermal stresses and during an accident, the problem of adequate bar anchorage is of special concern. Provide information on how the reinforcing bars are anchored at certain critical points, such as: center of dome, at intermediate terminal points of radial bars in the dome, bars provided to take discontinuity stresses, some diagonal bars, bars connecting the buttresses to main shell, bars under prestressing anchors, etc?

ANSWER:

Anchorage at the top of the dome is not necessary, since a rectangular mesh system is applied there on a circle with a radius of about two-thirds of that of the containment structure. Outside of this circle, a radial hoop reinforcement is continued to the edge of the dome. Where the two systems meet, they overlap on an area of a ring having a width equal to the required lap splice length for the applied reinforcement. No radial bar is terminated at an intermediate point.

Wherever the structure has a re-entrant corner, the reinforcements close to the surface cross each other, and they are deeply embedded into the concrete with the required anchorage length. Such conditions exist at the top and bottom of the ring girder of the dome cylinder connection, at the buttresses, at the edges of the thickened portions around penetrations, at the base of the cylinder, and at some details in the base slab.

Tension bars, terminated at penetrations, have standard hooks with ties being placed parallel with the axis of the penetrations to serve as trim or face steel for the inside of hole. The main vertical reinforcement at the top and bottom of the cylinder extends beyond the point where it is no longer needed, by the required anchorage length. The surface temperature reinforcement laps with the heavier discontinuity reinforcement on a lap splice length necessary to develop strength of the temperature steel, that is, some part of the strength of the local reinforcement is smoothly continued into the whole structure.

Behind the prestressing anchorages such reinforcement is applied as recommended and tested by the supplier of the post-tensioning system; basically similar to column reinforcement. In addition to this local reinforcement, the main reinforcement is continuous through the areas surrounding the anchorages to assure the transfer of stresses from the region just behind the anchorages into the whole surrounding concrete volume.

5J.7.16 (DRL 5.2.16) With respect to seismic design of the containment, please describe:

(a) The general analytical model for the containment including mass determination and distribution, stiffness coefficients, modes of vibration, and analytical procedures for arriving at a loading distribution on the containment structure.

(b) The order of magnitude of lateral earth pressure under seismic loading and indicate how such loading will be factored into the containment design.

(c) The manner in which damping will be considered in the structural design. In this description, justify the damping values employed for the various components of the structure, considering possible cracking and different modes.

(d) The extent and manner in which the horizontal, vertical, and rocking motions will be considered in the design, and how the corresponding damping values will be included. Describe the motion of the structure with respect to ground using the above three components of motion.

ANSWER

(a) The general analytical model tentatively selected for the seismic design of the containment structure is given in Figure 5J.7.16-1. In general, consideration will be given to the three basic characteristics involved in the lateral seismic behavior of the system: (1) structural deformation, (2) secondary vibration due to rocking, (3) secondary vibration due to translation of structure relative to ground. These three physical components of motion are used as generalized coordinates. The structure is treated as a continuous system with distributed (and lumped) masses and stiffnesses. The method of generalized coordinates based on the Ritz Method will be used.

Each of the three modes of vibration of the three-coordinate system will include components of each type of motion. The resultant of each coordinate is taken as the square root of the sum of the squares of the component of the coordinate in each mode. The distribution of lateral seismic load is obtained from these resultant coordinate motions.

Section 5.1.5.6 describes the seismic analysis in further detail.

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(b) The lateral earth pressure under seismic loading will be calculated by considering pressure waves propagating in a continuous medium (the earth) and striking an obstacle (the structure). The resulting earth stress at the location of the containment will be the dynamic soil pressure and will be of the order of magnitude of 500 psf. This load will be applied to the portion of the containment structure embedded in the earth. The passive pressure due to translational secondary vibration (motion relative to ground) is negligible relative to dynamic pressure. The overall effect will be treated as a non-asymmetric distributed load on a cylindrical shell.

(c) Each of the physical coordinates described in part (a) has a damping ratio which is obtained by using lower-bound values determined by considering different sources of damping. The structural damping was estimated taking into account concrete stresses and assuming moderate cracking of the prestressed structure. The list of structural damping values listed in Section 5.0 of Appendix 5A other than for rocking are taken from "Design Criteria for Nuclear Reactors Subjected to Earthquake Motions" by N. M. Newmark (presented at the International Atomic Energy Agency Meeting in Tokyo, 1967.) Radiation damping for rocking and translational components was computed using response curves from "Forced Vibration of a Body or an Infinite Elastic Solid" by R. N. Arnold, et. al. J. A. M., ASME, Vol. 22, Mo. 3, 1955 (Figures 4c and 4d). Internal soil damping was estimated using available information presented in "Design Procedures for Dynamically Loaded Foundations" by Whitman and Richart, ASCE, Vol. 93, Mo. SM6, 1967. Plastic soil damping was estimated on the basis of computed soil pressure and was assumed to occur only during the maximum hypothetical earthquake condition. These references are listed in Appendix 5A.

The following values of component damping will be used in the seismic analysis of the containment structure and contribution of each will be pro-
4 portioned as described in part (d) to obtain the total damping value for the containment structure:

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	Design * Earthquake	Max. * Earthquake
Structural Deformation	2%	5%
Translational Motion	30%	30%
Rocking	5%	9%

*These damping values apply only to the containment structure. Damping values for other strucures and components are as stated in Appendix 5A. 4

(d) The vertical ground acceleration is one-half of the horizontal acceleration. The soil flexibility will be considered in obtaining the actual acceleration of the structure due to the vertical ground acceleration. The technique used for analysis is similar to that employed for horizontal and rocking motion. The soil properties are used to obtain equivalent springs as outlined in References 6 and 7 of Section 7.0 of Appendix 5A of the PSAR. 4

The horizontal components of displacement are rocking, secondary translation, and structural deformation, as described in part (a).

The system's damping for each mode would be sum of the contributions of each component in proportion to the ratio of that component's contribution to the mode deformation. There are five components of motion, three horizontal and two vertical, as described above.

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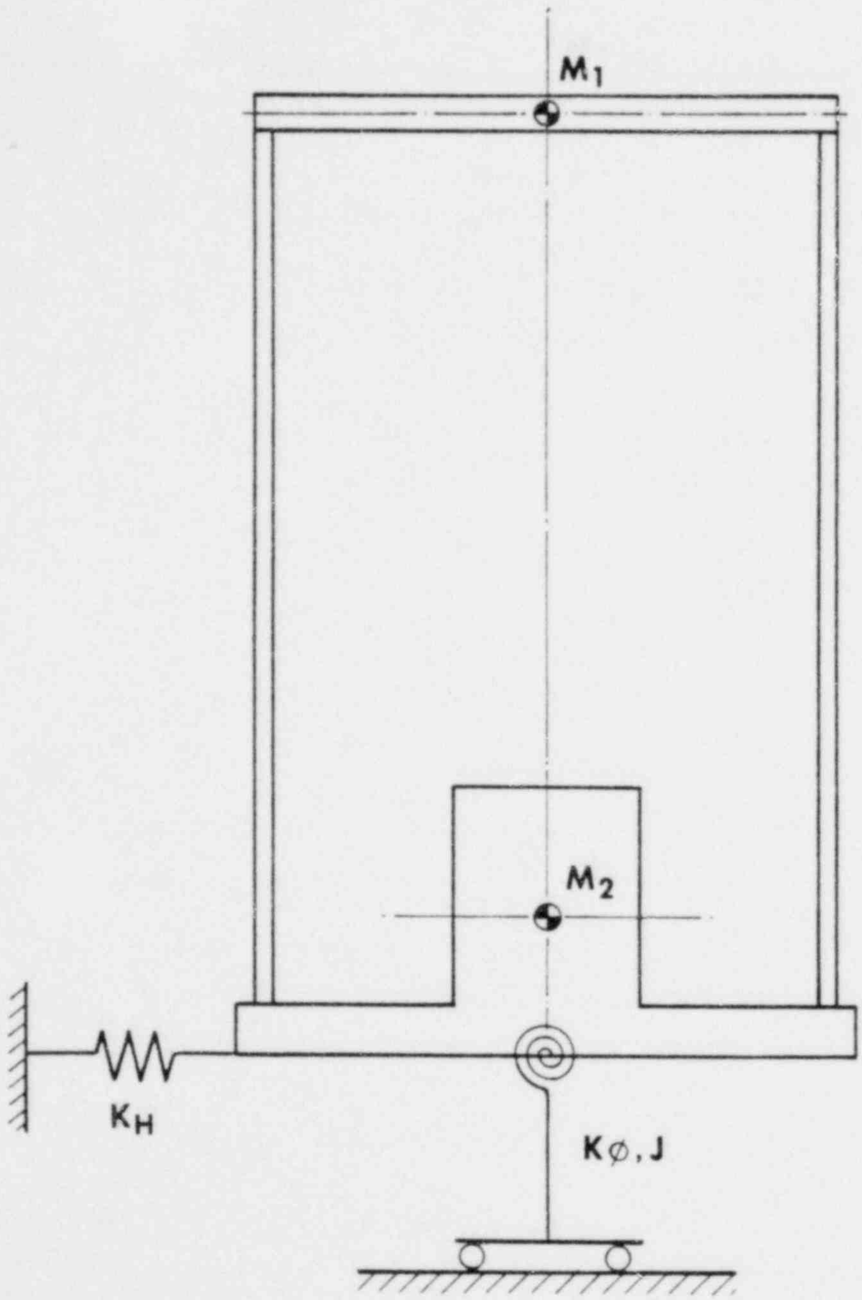

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FIGURE 5J.7.16-1
GENERAL ANALYTICAL MODEL
CONTAINMENT STRUCTURE

5J.7.17 (DRL 5.2.17) The design spectra shown in the PSAR, have been scaled from the El Centro spectrum. Indicate the degree to which this scaling was examined in connection with the Rancho Seco site.

ANSWER The design spectra, as recommended, were based on the information contained in the reports and publications:

a. Report on Seismic Hazards at the Clay Site P. Byerly, Aug. 1967*

b. Geophysical Report on Rancho Seco Power Plant Site, Boyle Brothers Drilling Co., July 1967*

c. Soil and Foundation Investigation Logs of Boring, Bechtel Corporation*

d. Nuclear Reactors and Earthquakes TID 7024.

The ground at the site has layers of well consolidated sandy-silty-gravelly materials. The upper layer, approximately 20 feet in thickness as described in the geophysical report, has seismic velocities in the range of 1200 to 2600 feet per second. The zone underlying this has seismic velocities in the range of 3000 to 4200 feet per second. This material is firm enough that there will be no adverse effect on the ground motion to preclude the use of the response spectra, as developed in TID 7024 and scaled up for the PSAR.

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* Reports included in the PSAR. Appendix #2.

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5J.7.18 (DRL 5.2.18) With respect to liner design, describe:

(a) Types and combinations of loading considered with regard to liner buckling, and the safety factors provided. Include the influence of large tangetal strains due to possible opening and closing of cracks in concrete;

(b) The geometrical pattern, type, and spacing of liner attachments; and the analysis procedures, boundary conditions, and results with respect to buckling under the loads cited above;

(c) Tolerance on liner plate thickness and liner yield strength variation of their bases;

(d) The possibility of both types of buckling; elastic and inelastic. In this study, discuss the influence of all pertinent parameters, such as:

Variation of plate thickness;
Variation of yield point of liner steel;
Influence of variation of Poisson's ratio;
Erection inaccuracies (local bulges, offsets at seams, wrong anchor location);
Prestressing;
Shrinkage of concrete;
Creep of concrete;
Variation of Young's modulus and Poisson's ratio for cracked and uncracked concrete, and as a function of stress level in concrete (elastic and plastic);
Ground water infiltration, and back pressure, earthquake, temperature loading, vacuum loading; and
Furnish sample calculations.

ANSWER: Please refer to question 5J.7.4.

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5J.7.19
(DRL 5.2.19)

Provide information on:

(a) The stress and strain limits used for the liner, the bases for these limits, and the extent to which these limits relate to liner leakage;
(b) The type, character, and magnitude of cyclic loads for which the containment liner will be designed, including a discussion of earthquake cycling;
(c) The analytical procedures and techniques to be used in liner anchorage design, including sample calculations; and
(d) The failure mode and failure propagation characteristics of anchorages. Discuss the extent to which these characteristics influence leak tightness integrity. What design provisions will be incorporated to prevent anchorage failures from jeopardizing leak-tight integrity?

ANSWER:

(a) Please refer to Section 5.1.4.9.

(b) Please refer to Section 5.1.4.9. Earthquake cycling is considered similar to the DBA cycling.

(c) Please refer to the answer of Question 5J.7.4.

(d) The answer to Question 5J.7.4 states that the individual anchors are designed to preclude failure of an individual anchor. The load-deformation tests referred to in answer to Question 5J.7.4 indicate that the alternate stitch fillet weld used to secure the anchor to the liner plate will fail in the weld and not jeopardize the liner plate leak tight integrity.

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5J.7.20 (DRL 5.2.20)	For the design of the anchors, elastic and inelastic buckling of the liner should be considered as well as the different modes of buckling of adjacent plates. Consider, for the design of the anchors, the possibility of unbalanced loads acting on several anchors. The study should prove that no chain reaction can occur and that the possibility of massive buckling of the liner, and mass failure of anchors is excluded.
ANSWER:	Please refer to question 5J.7.4.

5J.7.21 (DRL 5.2.21)	What plastic strains can the liner material accommodate without cracking?
ANSWER:	Due to the difficult task of evaluating ultimate (cracking) principal strains due to biaxial or triaxial stress conditions without an extensive test program, the allowable strain for the liner plate is conservatively assumed to be of the same magnitude as the allowable amplitude of strains due to 10 cycle loads specified in the ASME Boiler and Pressure Vessel Code, 1965 Section III, Article 4.

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5J.7.22 (DRL 5.2.22) Describe the design approach that will be used where loadings must be transferred through the liner such as at crane brackets or machinery equipment mounts; provide typical design details and computations.

ANSWER In designing for loads applied perpendicular to the plane of the liner plate, or loads transferred through the thickness of the liner plate, the following criteria will be used:

1. The liner plate will be thickened to reduce the predicted stress level in the plane of the liner plate. The thickened plate with the corresponding thicker weld will also reduce the probability of the occurrence of a leak at this location.

2. The criterion for brackets is as follows:

   Under the application of a real load applied perpendicular to the plane of the liner plate, no yielding is to occur in the perpendicular direction. By limiting the predicted strain to 90 percent of the minimum guaranteed yield value, the above criterion will be satisfied.

3. The allowable stress in the perpendicular direction can be calculated using the allowable strain in the perpendicular direction together with the predicted stresses in the plane of the liner plate.

4. In setting the above criterion the reduced strength and strain ability of the material perpendicular to the direction of rolling will also be considered, if the bracket does not penetrate the liner reinforcing plate.

5. The Quality Assurance Program will provide adequate inspection to assure the necessary plate characteristics.

Typical details have been shown in Figure 5.1-1 of the PSAR. Design calculations are considered proprietary and will not be furnished in this document.

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5J.7.23 (DRL 5.2.23) It is noted that the bottom liner is not accessible for inspection during the life of the plant. It is therefore very important to avoid any unnecessary stresses and strains in the bottom liner. The arrangement for load transfer through the liner under the bottom of the interior structure should provide for transfer of shears parallel to the liner. Indicate how the shears, especially those due to thermal expansion and earthquake, will be accommodated.

ANSWER The installation of leak chases at the bottom liner plate seams offers a leak isolation device for all bottom liner welds. Since the lateral and vertical loadings throughout the primary and secondary shield walls will vary, consideration will be given to shear transfer along a plane parallel to the liner in the following manner:

a. In areas of low-to-moderate shear the liner plate will be checked for local stress concentrations. Under these conditions the bearing capacity of the Cadweld splices on each side of the liner plate, as well as any other mechanical keys whose resistance may be utilized with full design assurance, will be used. It should be noted that current technical information incorporating recent shear research is being evaluated for possible application to this area, as well as throughout the containment structure. These papers (referenced below) are expected to be published shortly and pertinent features of which are expected to modify the shear provisions of the ACI code. The main contribution to the code values will originate from the "shear-friction" theory, which utilizes the fact that reinforcement perpendicular to a shear plane offers considerably more resistance to shear failure than is currently recognized.

b. In areas of large shears whose resistance cannot be developed throughout available internal anchorages, the walls will be keyed into a depressed leak-tight liner trough so that the shear strength can be transferred from the wall concrete to the mat concrete in bearing.

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c. References

1) Shear Transfer In Reinforced Concrete

by J. A. Hofbeck, I. A. Ibrahim and Alan H. Mattock - Structural Research Laboratory of the University of Washington, 1968 (Submitted to ACI for publication)

2) Design of Auxiliary Reinforcement in Precast Concrete Connections

by Robert F. Mast - Presented at the American Society of Civil Engineers Structural Engineering Conference, Miami Beach, Florida; Jan. 31, 1966. (Submitted to ASCE for publication)

5J.7.24 (DRL 5.2.24) Provide the latest liner arrangement to be used at the base-cylinder to liner juncture, the strain limits imposed at the juncture, and analysis of the capability of the chosen liner arrangement to absorb these strains under the design basis accident and earthquake conditions. Discuss the influence of local cracking on liner anchors.

ANSWER A detail of liner joint at the junction of the cylinder and the base has been added to Figure 5.1-1.

In the liner plate anchorage system, the forces in the liner plate are kept in equilibrium by the steel anchorage. The anchorage will also control the liner deformations. The strains at the junction should be about 0.0025 in/in. Refer to answer 5J.7.11 regarding the effects of cracking.

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5J.7.25 (DRL 5.2.25) Describe the analytical procedures for analysis of liner stresses around openings. Also, provide the method of liner design to accommodate these stresses and the related stress limits. Justify the proposed thickening of the liner at penetrations. Discuss the liner anchors at this location.

ANSWER: Using the theory of elasticity, the stress concentrations around openings in the liner plate will be calculated. The stress concentrations will then be reduced by the use of a reinforcing plate around the opening. In the case of a penetration with no appreciable external load, the anchor bolts will maintain strain compatibility between the liner plate and the concrete. Inward displacement of the liner plate at the penetration will also be controlled by the anchor bolts.

In the case of a pipe penetration in which large external loads are imposed upon the penetration, the stress levels from the external loads will be limited to the design stress intensity values, S_m' given in the ASME Boiler and Pressure Vessel Code, Section III, Article 4 the stress levels in the anchor bolts from external loads will be in accordance with the A.I.S.C. Code.

The combining of stresses from all effects will be done by the methods outlined in the ASME Boiler and Pressure Vessel Code, Section III, Article 4, Figure 414. The maximum stress intensity will be the value from Figure N-415 (A) of the previously referenced code. Shown in Figure 5J.7.25-1 is a typical penetration and the applied loads.

The stresses from the following effects will be calculated and the stress intensity will be kept below S_m: pipe loads, pressure loads, dead load, and earthquake.

The stresses from the remaining effects will be combined with the above calculated stresses and the stress intensity will be kept below S_a.

5J.7.26 (DRL 5.2.26) A general statement that all penetrations will be anchored into the concrete wall and that the anchorage will develop at least the plastic strength of the penetration sleeve would not be satisfactory if not followed by an explanation what plastic strength is meant. Provide this explanation in terms of the tension, bending, shear, and combined components.

ANSWER: The statement that "the anchorage will develop at least the plastic strength of the penetration sleeve" has not been stated in the PSAR.

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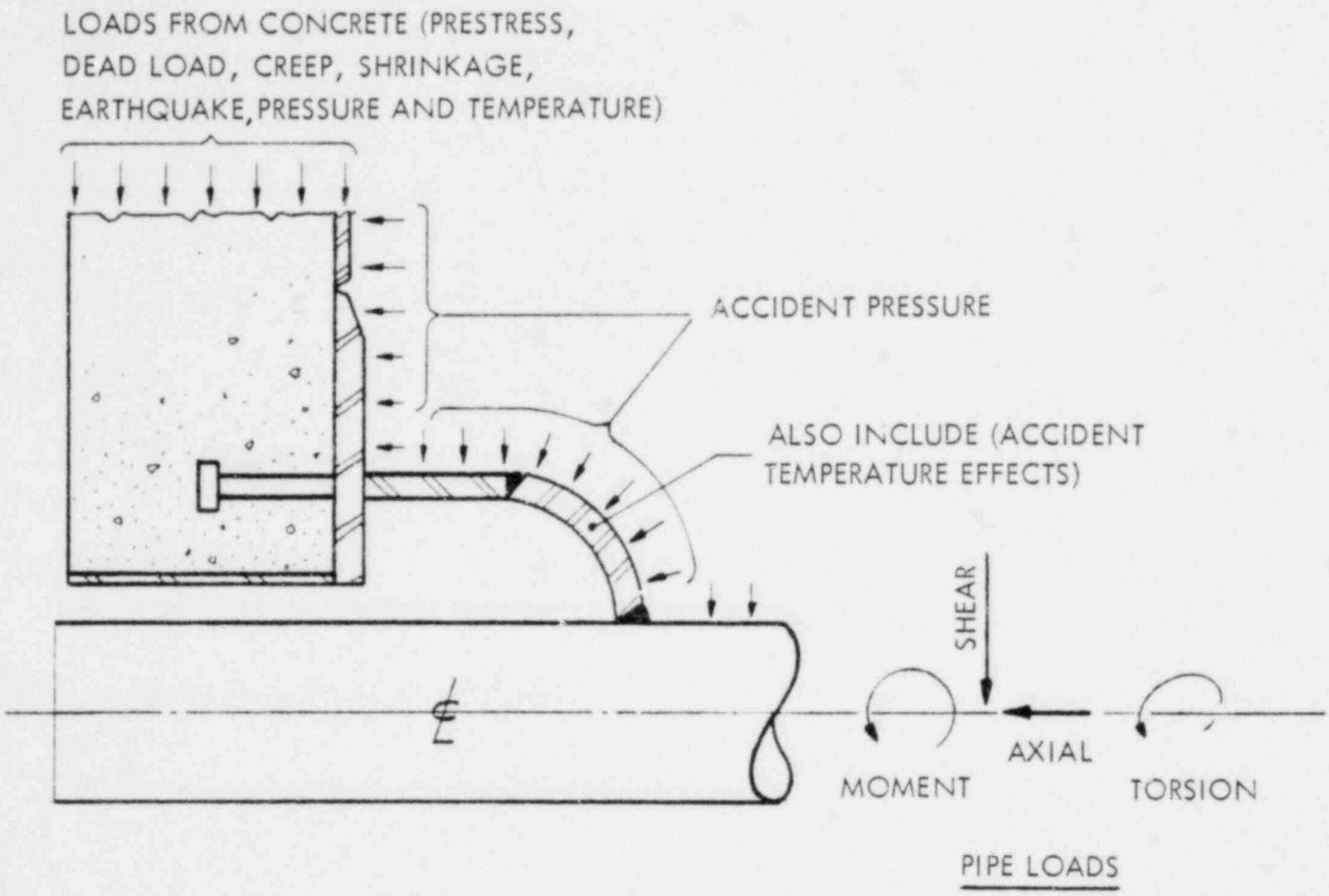

FIGURE 5J.7.25-1
PENETRATION LOADS

5J.7.27
(DRL 5.2.27)

With regard to penetration design, describe:

(a) The design criteria to be applied to ensure that piping loads under the postulated design basis accident which could result in pipe rupture or relative displacement of the internal systems relative to the containment, a subsequent pipe rupture due to torsional, axial, bending, or shear, will not cause a breach of the containment. Also, include the detailed design criteria with respect to pipe rupture between the penetration and containment isolation valves. These piping sections represent an extension of the containment boundary under a condition when isolation is required. What codes will be used? Provide typical designs to illustrate how the criteria are applied.

(b) The extent to which the penetrations and their surrounding liner regions will be subjected to vibratory loading from machinery attached to the piping systems. Indicate how these loads will be treated in design.

(c) The criteria for concrete thermal protection at penetrations; include the temperature rise permitted in the concrete under operating conditions and the (time dependent) effect that loss of thermal protection would have on the containment's structural and leak-tightness characteristics. What thermal gradients are used?

(d) The manner in which axial stresses, hoop stresses, shear stresses, bending stresses (in two directions) and shear stresses due to torsion are combined in the plastic domain, if the full plastic strength of a pipe with regard to torsion, bending and shear is to be used. What failure criterion is used? Indicate how the exterior loads are combined, including jet forces. Give factored loading combinations for all loads and all cases considered in the design. Explain how the Standard Code for Pressure Piping-Power Piping, B31.1.0-1967 will be used for all loading cases. Will factored load combinations be used with this code?

ANSWER

(a) The design criteria to ensure that piping loads under the postulated design basis accident will not result in a breach of containment are described in appendix 5A Section 3.1.3 and 3.2 and Sections 5.2.1.2 and 5.2.2 of the PSAR.

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Rupture between the penetration and the containment isolation valves will be prevented by designing this area as the strongest point in the system, either by pipe stops, increased pipe thickness, or other means.

Penetrations will conform to the applicable sections of ASA N6.2-1965 "Safety Standard for Design, Fabrication and Maintenance of Steel Containment Structures for Stationary Nuclear Power Reactors", as stated in Section 5.2.2.1. The basis for limiting strains in the penetration steel will be the ASME Boiler and Pressure Vessel Code for Nuclear Vessels, Section III, Article 4, 1965.

Typical illustrated designs are shown in Figure 5.1-2.

(b) Piping penetrations will not be subject to vibratory loading from operating machinery. The length and piping configuration will be such that it would be impossible for operating machinery to transmit vibratory loading to the penetration. In addition, design of the piping supports will take into account vibratory loading in order to eliminate them completely.

(c) The criteria for concrete thermal protection at the penetrations is described in Section 5.2.2.2 and an illustration of a typical hot penetration is shown in Figure 5.1-2.

(d) Combining Stresses in Plastic Domain. Concurrent design loads are checked in the containment penetration to assure that the construction will be carrying these loads elastically after shakedown in the plastic region.[1]

Failure Criterion. (a) Concurrent primary and secondary design stresses will not exceed 3 S_m for Class B vessels per the Nuclear Vessel Code, ASME Boiler and Pressure Vessel Code, Section III. (b) Concurrent primary and secondary stresses under the "no loss of function" criterion that includes stresses due to jet forces are limited to 1.2 times the code-based stress limits as previously stated elsewhere. (See p. 5J-7 Amendment 2)

Piping deformations at the penetrations will be limited so that the resulting liner plate strains do not exceed the criteria stated in PSAR Section 5.1.4.9. 4

Factored Loading Combinations and all Cases Considered in the Design. Primary loads include those due to internal pressure, weight of pipe insulation and contents, snow, earthquake and wind loads. Secondary loads include those due to thermal expansion or self-equilibrating or self-limiting loads or deflections.

Use of B31.1.0-1967. Piping analyzed for sustained and thermal expansion load follows the requirements of the Standard Code for Pressure Piping - Power Piping USAS B31.1.0-1967 in two separate analyses each using the specified modulus of elasticity, Poisson's ratio, and flexibility and stress intensification factors. Sustained loads of B31.1.0 are considered primary loads under ASME Section III, while the thermal expansion load gives rise to reactions considered secondary loads in consideration of the penetration nozzle to the containment under ASME Boiler and Pressure Vessel Code Section III. The piping analysis considers snow and earthquake or wind loads with those due to pressure and weight as a sustained load combination.

---

[1] $S = 1/2[\sigma_{ax} + \sigma_h + \sqrt{4 S_s^2 + (\sigma_{ax} - \sigma_h)^2} \leq 3 S_m$ using design loads, where

σ_{ax} = Axial stress + bending stress calculated using the resultant of two bending moments

σ_h = Hoop stress

S_s = Shear stress

for the sum of primary and secondary stress. Maximum principal primary stresses are examined first to assure average primary membrane stress does not exceed S_m or 1.5 S_m considering structural discontinuities.

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5J.7.28 (DRL 5.2.28) Provide criteria with regard to opening sizes that constitute large openings; hence, meriting special design consideration. List the number and indicate the size of the large openings for the containment.

ANSWER In general, special design consideration is given to all openings in the containment structure. Previous analysis of similar openings, however, indicates that the degree of attention required depends upon the penetration size. Small penetrations are those with a diameter smaller than 2 1/2 times the shell thickness; i.e., approximately 8 ft in diameter or less. In general, the existing concrete wall thickness has been found to be capable of taking the imposed stresses using bonded reinforcement, and the thickness is increased as required to permit space requirements for tendon deflection. The induced stresses due to normal thermal gradients and postulated rupture conditions distribute rapidly and are of a minor nature, compared to the numerous loading conditions for which the shell must be designed. Typical details associated with these openings are indicated in Figure 5.1-?.

The personnel lock and equipment hatch are classified as large openings and criteria and details are discussed in Sections 5.1.4.6, 5.1.4.9, 5.1.5.2, 5.1.5.3 and 5.1.5.5, and shown on Figure 5.1-3. It should be noted that the continuity of tendons is maintained in all openings.

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5J.7.29 (DRL 5.2.29)

Add the following information:

(a) For all penetrations, indicate the criteria for the bending of reinforcing bars which have to clear the openings. Maximum slope and minimum bending radius to avoid local crushing of concrete should be shown.

(b) For penetrations greater than about 9 inches and up to and including about 4 feet, explain how normal, shear, bending, and torsional stresses are covered by the prestressing and by the reinforcing bars.

(c) Justify the length required to anchor the bars in cracked concrete, and the use of ACI code 318 or any other code to determine anchorage requirements for concrete under biaxial tension, and cracked in two directions.

ANSWER

(a) Horizontal and vertical reinforcing steel will be spaced such that bending of these bars around penetrations will not be necessary. Any hooks on bars terminating at penetrations will be in accordance with ACI 318-63 Code, Section 801. Since hoop reinforcement will have a large radius it will also comply with the above code.

(b) The state of stress around penetrations having diameters from .75 to 4.0 feet will be calculated in accordance with the design criteria given in the PSAR;* stress concentrations will also be considered in the analysis. The prestressing only increases or decreases the magnitude of stress; prestressing is not relied upon to resist concrete stresses above the allowable values. Reinforcing will be provided when the concrete stress exceeds the acceptable values stated in the PSAR.*

Horizontal and vertical reinforcement will be provided to help resist membrane and flexural loads; this reinforcement will be located on both the inside and outside face of the concrete. Stirrups will be used if necessary to resist shear loads. The torsional effects on small holes are negligible.

(c) The required anchorage lengths specified in the ACI 318-63 Code are applicable in structures where two dimensional tension stresses exist such as flat slabs and chimneys.

* Section 5.2 is devoted to the design, construction and testing of penetrations.

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5J.7.30 (DRL 5.2.30) With respect to large opening design, describe:

(a) The primary, secondary, and thermal loads that will be considered in the design of the openings, and how they were established;

(b) The stress analysis procedures that will be used in design;

(c) The method that will be followed for the design; the working stress design method or the ultimate strength design method, or both; If the ultimate strength design method is used, the factored load combinations should be given together with corresponding capacity reduction factors;

(d) How the existence of biaxial tension in concrete (cracking) will be taken care of in the design; How the normal and shear stresses due to prestressing, to axial load, two-directional shear, and torsion, will be combined; Clarify these points and establish criteria for the design of the thickened part of the wall around the opening (ring girder). Reference to recent pressure tests of similar openings would not be conclusive, since the thermal and earthquake loads were not applied during tests, and since these tests have not established the safety factor provided in the structure (tests have not been continued till failure occurred).

(e) The method to check the design of the thickened stiff part of the shell, around large openings and its effect on the shell; Include prestressing, creep and shrinkage. The comparison with stresses in a circular flat plate would not be convincing, since it eliminates one of the most important effects, i.e. the effect of torsion. Present a method which checks torsional stresses.

(f) Additional information on reinforcing pattern, i.e., rebar size and spacing, and prestressing pattern that will be used around large openings;

(g) The safety factor provided in design at large openings; Sample computations should be provided, listing all the criteria and analyzing the effect of all pertinent factors such as prestressing, cracking, etc.

ANSWER (a) Primary Loads: -

These are the loads, resulting from:

(1) Dead load

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(2) Prestress load

(3) Accident pressure load

(4) Earthquake load

Secondary Loads: -

These are the secondary effects caused by above mentioned loads. These can be summarized as follows:

(5) Effect of deflection of tendons around opening

(6) Effect of thickening around opening

(7) Any secondary effect resulting from closeness of opening to the base of the cylinder.

Thermal Loads: -

These will be considered in two parts:

(8) Thermal loads under operating condition

(9) Thermal loads under accident condition

Loads described under primary loads are mainly membrane loads. In addition to membrane loads, accident pressure also produces punching shear around the edge of the opening. The values of these loads for design purposes will be the magnitude of these loads at the center of the opening. These are fairly simple to establish knowing the values of hoop and vertical prestressing, value of accident pressure and geometry and location of the opening.

Secondary loads summarized above will be predicted by the following methods:

(5) The membrane stress concentration factors and effect of the deflection of the tendons around the equipment hatch will be analyzed for a flat plate by the finite element method. The stresses predicted by conventional stress concentration factors will be used for comparison with these values. It will be demonstrated the deflection of the tendons does not significantly affect the stress concentrations. This is a plane stress analysis, however, it does not include the effect of the curvature of the shell. However, it gives an assurance of the correctness of the assumed stress pattern caused by the prestressing around the opening.

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(6) See answer to subquestion (b).

(7) These loads will be predicted from axisymmetric finite element computer analysis carried out for dead load, prestress load and internal pressure load.

Thermal loads will be established from the computed thermal gradient at the large opening.

(b) In addition to the steps outlined in Section 5-1.5.5 of PSAR, the following method will be used to account for the thickened part of the opening.

(1) With the help of Reference A, Section 5-1.5.5 of the PSAR, stress resultants around the large opening will be found for various loading cases. Comparison of the results found from this reference with the results of a flat plate of uniform thickness with a cylindrical curvature on stress concentrations around the opening.

(2) Corrections will be applied to the stress resultants calculated in Part 1 above to account for the thickening on the outside face around large opening. These effects will be considered using a separate axisymmetric finite element computer analysis for both a flat plate and a dome with anticipated thickening on the outside face. This finite element computer program will handle axisymmetric and non-axisymmetric loads. The computer result gives six components of stresses; three components for normal stress and three components for shear stress. This finite element computer program will also be used to predict the effect of concentration of hoop tendons (with respect to containment) at the top and bottom of opening.

Based on the past experience of the analysis and design of large openings and equipment hatch openings, the following results are of interest:

(1) The governing design condition for the sides of the opening at the outside edge of the opening is the accident condition.

(2) Under the condition mentioned in (1)

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2.1 Approximately 60 percent of the total bonded reinforcing steel needed at the

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edge of the opening at outside face, is a result of the thermal load.

2.2 Excluding thermal load, the remaining stress (equivalent to approximately 40 percent of the total load including thermal) at the edge of the outside face is the contribution of the following stress resultants:

2.2.1 Stresses resulting from membrane forces, including the effect of thickening, contribute approximately minus 35 percent (minus 14 percent of total).

2.2.2 Stresses resulting from the moments caused by thickening on the outside face contribute approximately 150 percent (60 percent of total).

2.2.3 Stresses resulting from membrane force and moments caused by the effect of cylindrical curvature contribute approximately minus 15 percent (minus 6 percent of total).

In order to minimize the effect of tensile stresses at the outside face and to distribute the concentration of radial forces exerted by hoop tendons in a more uniform manner, the inside row of vertical tendons were given a reverse curvature (they are deflected outward as they pass the opening) so as to reduce the inward acting radial forces (due to hoop tendons) at the top and bottom of the opening and to produce inward acting forces on the sides (no inward radial force acts on sides because of absence of hoop tendons) of the large opening.

(c) The working stress method (elastic analysis) will be applied to both the load combinations for design loads, as well as for yield loads, and the analytical procedures are described in the answer to subquestion (b). The only difference is the higher allowable stresses under yield conditions. The design assumption of straight line variation of stresses will be maintained under yield conditions.

The different factored load combinations have been given under 5.1.4.6. Various capacity reduction factors have also been specified under 5.1.4.7 and these will be used for the yield load combinations using the working stress design method.

(d) The biaxial cracking changes the stress distribution that is predicted when assuming uncracked concrete. When the cracking is at the outside face, thermal moments will be reduced, i.e. self relieving. For proper control of crack width and spacing, well distributed reinforcing steel will be provided in hoop and radial direction.

A typical element at the equipment hatch will have the following stress-components acting on it. (relative to the opening unless otherwise stated)

(1) Hoop stress

(2) Radial Stress

(3) Radial stress relative to containment

(4) Radial shear stress

(5) Hoop shear stress

(6) Inplane shear stress

The above mentioned stresses will be calculated for various loading conditions as mentioned in answering subquestion (a) and then they will be combined for various load combinations as described in Section 5.1.4 of the PSAR.

Large openings will be thickened, if necessary for the following reasons:

(1) To reduce the larger than acceptable predicted stresses around the opening.

(2) To accommodate tendon placement.

(3) To accommodate bonded steel reinforcing placement.

(4) To compensate for the reduction in the overall shell stiffness due to the opening.

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The method of analysis to account for the thickening around large openings is described in the answer to the subquestion (b). The other finite element program mentioned in the answer to the subquestion (b) is the latest one developed at the University of California - Berkeley which can analyze axisymmetric structures with non-axisymmetric loadings.

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We will be reviewing and maintaining constant touch with the developments of new techniques that can be useful for analysis and design of large openings.

(e) The method of analysis and design of thickened stiff part is explained in answer to the subquestion (b).

The effect of thickening is to concentrate more membrane loads in comparison with an unthickened opening. The effect of thickening on the shell is to create local disturbances in predicted stress pattern. The behavior of the shell in general is not affected by it.

Creep and shrinkage will not be considered in the analysis. However, compatibility of strain between general vessel shell and the area around the opening will be maintained by thickening the concrete around large opening.

Normal shear forces (relative to opening) will be modified to account for the effect of twisting moments as shown in reference (a) mentioned in Section 5.1.5.5 of the PSAR. These modified shear forces are called Kirschoff's shear forces. Horizontal wall ties will be provided to resist these shear forces.

(f) Figure 5.1-3 gives the information regarding preliminary rebar size and spacing and prestressing pattern that will be used around large openings. Please note that the drawing does not show the final design.

(g) It is very difficult to define exactly the safety factors for a large opening. This may be different for different loading combinations.

In a general sense, however, the large opening will be able to withstand the specified load combination with the specified stress limitations in the criteria given under Section 5.1.4.

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5J.7.31 (DRL 5.2.31) List the spectrum of external missiles that the containment will be designed to withstand and the procedures to be used in checking the containment design to withstand such missile hazards.

ANSWER

The design of the containment structure for missile protection is discussed in considerable detail in Sections 1.4.40, 5.1.4.10, 5.1.4.11 and Appendix 5D. The spectrum of potential missiles within the reactor building is listed in Table 5J.7.31-1. The spectrum of potential missiles generated from external sources includes those from two sources, those which could result from the failure of the turbine-generator rotating elements, and those originating from unusually high wind conditions. The former source is discussed in Section 5.1.4.11 and is summarized in that paragraph as non-existent. The latter has been checked in considerable detail on similar containments for wind velocities considerably higher than those at the Rancho Seco site. It has been found that the prestressed concrete containment structure has considerable reserve strength in its ability to withstand these potential external missiles. The specific list of these objects are analogous to those outlined in Section 5.1.4.10 and are found adjacent to the containment structure, specifically various types of nuts and bolts, sections of pipe up to 10 inches in diameter (larger sizes will not control due to the large impact area), and valve bonnets and stems.

The procedures selected for the design of the impacted structures or protective components will be those outlined in the references listed below. It should be demonstrated that these references contain many sub-references defining the current state-of-art in this area and that this additional bibliography will be used in the analysis.

<u>References</u>:

a. <u>U.S. Reactor Containment Technology</u> - Vol. I - Oak Ridge National Laboratory and Nuclear Information Center, ORNL-NSIC-5 (Chapter 6)

b. <u>Nuclear Reactors And Earthquakes</u> - United States Atomic Energy Commission - Division of Technical Information, TID-7024 (Chapter 7)

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TABLE 5J.7.31-1
POSSIBLE MISSILES WITHIN THE REACTOR BUILDING

A. REACTOR VESSEL AND CONTROL ROD DRIVE

Missile Class	Description	Weight (lbs)	Pressure (psi)	Stroke (in)	Fl (
I	1. Closure head nut	80			
	2. Closure stud w/nut	660			
	3. 1 in. valve bonnet stud	0.5			
	4. CR nozzle flange bolt & nut	2.0			
	5. 1/2 in. vent valve bonnet nut	0.9			
	6. CRD vent cap stud w/nut	2.0			
	7. CRD gear box mtg. stud w/nut	12.0			
II	1. Gasket leakoff conn.-1 in. valve stem & wheel	4	2,200	8	
	2. CRD 1/2 in. vent valve stem & wheel	1	2,200	3	
	3. CRD seal water 1/4 in. valve stem	0.35	2,200	3	
III	1. CR drive assembly	1500			
	2. CRD vent cap w/valve	55			
	3. CRD motor & clutch assembly	750			
	4. Gasket leakoff conn.-1 in. valve bonnet and assembly	30			

Note: All CRD missiles listed above are based on rack and pinion drive.

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id Vel. t/sec)	Impact Area in^2	Velocity (ft/sec)	Kinetic Energy ft-lbs
	38	97	11,680
	71	97	96,400
	0.6	73.5	42
	3.1	97	292
	.34	73.5	7.5
	.4	73.5	167
	.8	73.5	1,000
	.3	84	440
	.2	84	110
	.05	71	27
448	64.0	(Dependent on	
558	13.4	(distance traveled	
558	47.0	(before impact	
448	27.0	(	
		(	

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TABLE 5J.7.31-1 CONTINUED

Missile Class	Description
I	1. 1½ in. vent valve bonnet s 2. Feedwater inlet flange bol 3. 16 in. ID manway stud, tub 4. 5 in. inspection opening c 5. 1 in. valve bonnet stud
II	1. 1½ in. vent valve stem & v 2. Sample line 1 in. valve st 3. Sample line 1 in. EMO valv wheel
III	1. 16 in. ID manway cover, t 2. 16 in. ID manway cover, s 3. 5 in. ID inspection cover side 4. 5 in. ID inspection cover side 5. 1½ in. vent valve bonnet assembly 6. Sample line 1 in. valve b assembly 7. Sample line, 1 in. EMO bo assembly

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B. STEAM GENERATOR

	Weight (lbs)	Pressure (psi)	Stroke (in)	Fluid Vel. (ft/sec)	Impact Area in^2	Velocity (ft/sec)	Kinetic Energy ft-lbs
tud	2.0				.8	73.5	167
t	0.3				.6	67.5	21
e side	8.0				2.1	67.5	566
over stud	1.5				1.2	73.5	125
	0.5				.6	73.5	42
heel	5.0	925	4.5		.45	44.5	154
em & wheel	4.0	925	3.5		.3	35.8	80
e stem &	4.0	925	3.5		.3	35.8	80
be side	955			515	615	(Dependent on	
ell side	478			1065	615	(distance traveled	
tube	80			515	150	(before impact	
						(	
shell	40			1065	150	(	
						(	
nd	24			1065	38	(	
						(	
nnet and	30			1065	27	(	
						(	
net &	115			1065	27	(	

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TABLE 5J.7.31-1 CONTINUED

C. PRESSURIZER

Missile Class	Description	Weight (lbs)	Pressure (psi)	Stroke (in)	F
I	1. 4 in. valve bonnet stud	3.0			
	2. 5 in. valve bonnet stud	3.0			
	3. 16 in. manway cover stud	7.5			
	4. Heater bundle stud	25.0			
	5. 3/4 in. valve stem stud	0.8			
II	1. Spray line 4 in. EMO valve stem	9	2200	14	
	2. Sample line 3/4 in. valve stem	4	2200	6	
	3. Sample line 3/4 in. EMO valve stem	4	2200	6	
III	1. 16 in. ID manway cover	250			
	2. Heater bundle assembly	2500			
	3. Spray line 4 in. EMO valve bonnet and assembly	325			
	4. 2½ in. x 6 in. relief valve bonnet and assembly	175			
	5. Sample line 3/4 in. valve bonnet and assembly	20			
	6. Sample line 3/4 in. EMO valve bonnet and assembly	115			

D. INSTRUMENTS

Missile Class	Description	Weight (lbs)	Pressure (psi)	Stroke (in)	F
III	1. RTE	1.0			
	2. RTE & plug	2.0			

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luid Vel. (ft/sec)	Impact Area in^2	Velocity (ft/sec)	Kinetic Energy ft-lbs
	1.8	73.5	250
	2.4	73.5	250
	3.1	67.5	530
	7.0	73.5	2100
	.45	73.5	67
	1.0	135.0	2560
	.3	72.7	330
	.3	72.7	330
375	615	(Dependent on	
375	850	(distance traveled	
523	150	(before impact	
		(	
375	65	(	
		(	
375	21	(	
		(	
375	21	(	
		(	

448	.2	(Dependent on	
448	4.0	(distance traveled	
		(before impact	

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Missile Class	[illegible]
	Core Flooding Li
I	14 in. CV bonn
I	14 in. valve b
II	14 in. CV chec
II	14 in. PO valv
III	14 in. CV bonn
III	14 in. PO valv
	LP Injection Lin
I	12 in. CV bonn
II	12 in. CV chec
III	12 in. CV bonn
	RV Outlet Line t
I	10 in. valve b
I	Relief valve b
I	Relief valve s
II	10 in. EMO val
III	10 in. EMO val
	RV Inlet Line fr
I	4 in. CV bonne
II	4 in. CV check
III	4 in. CV bonne
	SG Outlet Line t
I	1 in. drain va
II	1 in. drain va
III	1 in. drain va

* Dependent on distance trav

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E. SYSTEM PIPING

escription	Weight (lbs)	Pressure (psi)	Stroke (in)	Fluid Vel. (ft/sec)	Impact Area in^2	Velocity (ft/sec)	H E f
ne							
et stud	2.0				1.7	73.5	
onnet stud	3.5				4.0	67.5	
k pivot stud	10.0	2185	20		1.75	249	
e stem	98.0	2185	34		5.0	143	[illegible]
et & assembly	525.0			448	125	*	*
e bonnet and assy.	1900.0			558	650	*	*
e							
et stud	2.0				1.7	73.5	
k pivot stud	10	2185	20		1.75	249	
et and assy.	450			558	95	*	*
o LP System							
onnet stud	2.5				1.7	73.5	
onnet stud	0.5				.3	73.5	
em assy.	40				12.5	35.3	
ve stem	50	2185	27		3.1	130	1
ve bonnet & assy.	1270			558	415	*	*
om H.P. System							
stud	1.0				.8	73.5	
pivot stud	3.0	2185	8		.8	158	
and assy.	30			558	19	*	*
Pump Inlet							
ve bonnet stud	0.8				.6	73.5	
ve stem assy.	4.0	2185	8		.3	84	
ve & bonnet assy.	30.0			448	27	*	*

led before impact.

TABLE 5J.7.31 CONTINUED

E. SYSTEM PIPING

Missile Class	Description	Weight (lbs)	Pressure (psi)	Stroke (in)	Flu (f
	Pressurizer to CA System Line				
I	3/4 in. valve bonnet stud	1.0			
II	3/4 in. valve stem	4	2185	6	
II	3/4 in. EMO valve stem	4	2185	6	
III	3/4 in. valve bonnet and assy.	20			
III	3/4 in. EMO valve bonnet and assy.	115			
	Primary Pump Seal Water Return to H.P. System Line				
I	3 in. EMO valve bonnet stud	1.0			
II	3 in. EMO valve stem	25.0	2185	14	
III	3 in. EMO valve bonnet and assy.	285.0			
	Letdown Cooler Inlet & Outlet Lines				
I	1½ in. EMO valve bonnet stud	2.0			
II	1½ in. EMO valve stem	1.0	2185	10	
III	1½ in. EMO valve bonnet and assy.	250.0			
	Primary Pump Seal Water Inlet and Outlet Lines				
I	3 in. inlet CV bonnet stud	1.0			
I	3 in. outlet valve bonnet stud	2.0			
II	3 in. CV check pivot stud	3.0	2185	8	
II	3 in. outlet valve stem	25.0	2185	14	
III	3 in. inlet CV bonnet and assy.	25.0			
III	3 in. outlet valve bonnet and assy.	65.0			
	Primary Pump Vent & Drain Lines				
I	1½ in. vent & drain valve bonnet stud	2.0			
II	1½ in. vent & drain valve stem	5.0	2185	10	
III	1½ in. vent & drain valve bonnet	55.0			

* Dependent on distance traveled before impact.

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id Vel. t/sec)	Impact Area in^2	Velocity (ft/sec)	Kinetic Energy ft/lbs
	.45	73.5	83
	.3	73	330
	.3	73	330
448	21	*	*
448	21	*	*
	1.0	73.5	83.5
	.3	125.7	6150
523	85	*	*
	.8	73.5	167
	1.0	153.2	1830
448	38	*	*
	.8	73.5	83.5
	1.0	73.5	167
	.8	158.4	1170
	2.4	125.7	6150
558	85	*	*
523	85	*	*
	.8	73.5	167
	1.0	153.2	1830
448	38	*	*

5J.7.32 (DRL 5.2.32) If insulation is required, present a detailed study of it. Design requirements and performance specifications should be included to provide confidence that the insulating qualities will be achieved under accident conditions. Hence, provide a description of:

(a) The specified and tolerable temperature rise in the liner and the design safety factor provided on insulating performance;

(b) Means provided for fastening the insulating material to the backing liner and for precluding steam channeling in back of the insulation (from the top or through joints) and state whether the insulating panels be removable;

(c) An analysis of the consequences of one or more insulation panels being displaced from the liner during, or as a consequence of, an accident situation;

(d) The consideration given to increased conductivity due to humidity and compression during accident pressure transients and precompression from structural and leakage testing;

(e) The consideration that will be given to the compatibility of the insulation and liner.

ANSWER There is no insulation required for the liner plate. Insulation material will be provided at penetrations maintained at high temperatures, to the extent that the structural integrity of the surrounding concrete dictates. A description of the penetration insulation material is given in the answer to question 5J.2.27.

0359 ~~00275~~

5J.7.33 (DRL 5.2.33) Provide a description of the procedures used for analyzing anchorage zones and provide typical results of such analyses. Include consideration of biaxial tension in concrete.

ANSWER Section 3 of Appendix 5G describes the general method of analyses for anchorage zones. This answer will address itself to the analysis of the anchorages at the buttresses since they have been determined to be the most critical. The local stress distribution in the immediate vicinity of the bearing plates has been derived for a similar containment structure by the following three analysis procedures:

(a) The Guyon equivalent prism method. This method is based both on experimental photo-elastic results as well as on equilibrium considerations of homogeneous and continuous media. It should be noted that the relative bearing plate dimensions are considered.

(b) In order to include biaxial stress effects, use has been made of the experimental test results presented by S. J. Taylor at the March 1967 London conference (Group H, Paper 49). This paper compares test results with most of the currently used approaches (such as Guyon equivalent prism method). He also investigates the effect of the rigid trumpet welded to the bearing plate.

(c) The finite element method assuming homogeneous and elastic material was used in a plane strain approach. Refer to answer 5J.7.5 for the output of this method.

The Guyon approach yields the following results for a loading ratio:

$$A'/A = 0.9$$

Maximum compressive stress under the bearing plate:

$$\sigma_{C_{BPL}} = -2400 \text{ psi}$$

Maximum tensile stress in spalling zones:

$$\sigma_{Spalling} = 2400 \text{ psi}$$

Maximum tensile stress in bursting zones:

$$\sigma_{C_{BPL}} = 0.04P = 95 \text{ psi}$$

0360 ~~00276~~

According to S. J. Taylor's experimental results the anchor plate will give rise to a similar stress distribution pattern as Guyon's method. The main difference lies in the fact that the central bursting zone has a tensile stress peak of twice Guyon's value.

$$\sigma_{\text{Max. Bursting}} = 190 \text{ psi}$$

As to the finite element approach, the symmetric buttress loading yields a tensile peak stress in the bursting zone very close to the S. J. Taylor's value.

$$\sigma_{\text{Max. Bursting}} = 220 \text{ psi}$$

Biaxial Tension in the Concrete

A state of biaxial tension in the concrete will appear on the outside face under the case D plus F plus 1.5P plus TW. The superposition of the corresponding state of stress with the local anchor stresses will reduce the load carrying capacity of the anchorage unit and cause a reduction in the maximum tensile strain at cracking.

On the other hand, the uniform compressive state of stress (vertical prestress) applied to the anchorage zone increases the load carrying capacity of the anchorage unit, the maximum tensile strain at cracking being increased.

The considered buttress anchor zones will be submitted as such to additional vertical stresses, leading to pseudo biaxial state, the second direction being radially through the thickness.

For the above mentioned case D plus F plus 1.5P plus TW, the averaged vertical (meridional) stress component will be:

$$f_a \cong 400 \text{ psi}$$

The compressive bearing plate stress at 10" depth below the bearing plate will be:

$$f_c \cong -1500 \text{ psi}$$

(Note. The steel trumpet carries 7.2 percent of the prestress force.)

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Thus, the two values introduced in the biaxial stress envelopes in S. J. Taylor's article.

$$f_c/f'_c = \frac{1500}{5000} = 0.3$$

$$f_a/f'_c = \frac{400}{5000} = 0.08$$

Show that failure occurs and that vertical reinforcing is indispensible. In fact the maximum allowable vertical averaged tensile stress according to Taylor's interaction curve is:

$$f_a/f'_c = -0.03 \quad (\text{e.g. } f_a = 150 \text{ psi})$$

This value could be used as a guide line to decide on the necessity of special anchorage reinforcing. For this reason, special anchorage zone reinforcing is used in addition to that required by the loading cases. Such special reinforcing is based on the following considerations in addition to the designer's judgement.

1. Full scale load tests of the anchorage on the same concrete mix used in the structure and review of prior uses of the anchorage.

2. The post-tensioning supplier's recommendations of anchorage reinforcing requirements.

3. Review of the final details of the combined reinforcing by the consultin, firm of T. Y. Lin, Kulka, Yang and Associate.

The results of the analysis, with finite values, will be
4 made available for review when the liner design is completed, which will be by February 1969.

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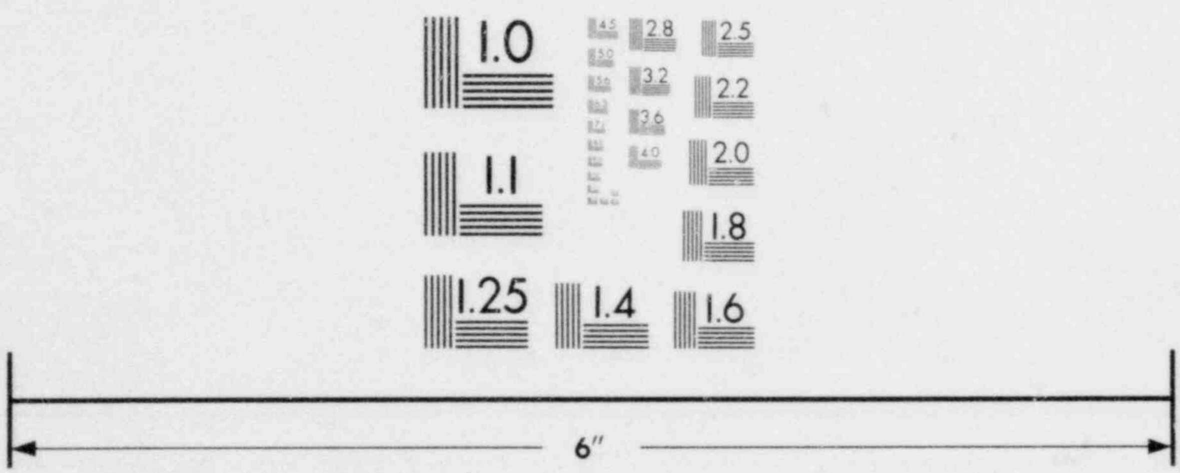
0362

IMAGE EVALUATION
TEST TARGET (MT-3)

6″

MICROCOPY RESOLUTION TEST CHART

IMAGE EVALUATION TEST TARGET (MT-3)



MICROCOPY RESOLUTION TEST CHART

QUESTION 5J.7.34 (DRL 5.2.34)

Provide typical details of anchorage zone reinforcing. Provide information that support its adequacy to resist the imposed anchorage loading (particularly under long-term loading). Justify bond values used for anchorages of reinforcing bars.

ANSWER

All suppliers of post-tensioning systems are required to demonstrate on a full-scale test, the adequacy of the anchorage reinforcement, hardware, and bearing plate using the same concrete materials that will be used in the structure.

If requested, typical details of reinforcing steel for the end anchor zone of the selected prestressing system will be provided after selection of the prestressing system.

At the buttresses and the ring girder, anchorage zone reinforcing is provided independently of the reinforcement provided for other loading conditions. Anchorage reinforcement will be provided in accordance with the post-tensioning suppliers recommendations and will be reviewed by Bechtel Corporation and the consulting firm of T. Y. Lin, Kulka, Yang & Associate. Both the supplier's recommendations and consultant's review are based on previous use of the system or similar systems over a period of approximately fifteen years.

5J.7.35 (DRL 5.2.35)

Indicate the criteria by which reinforcing steel will be provided in the containment shell for crack control, considering possible reversal of stresses during cold shut-down.

ANSWER

Please refer to Question 5J.7.8. (1)

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QUESTION 5J.8 (DRL 5.3) MATERIALS

5J.8.1 (DRL 5.3.1) Justify the type cement to be used, explain the basis for its selection, and describe the user verification testing to be performed.

ANSWER Cement for all structural concrete will be Type II as described in Section 5.1.3.1. This type will be specified because it is the most suitable type for large structures where heat generation during hydration must be minimized. User verification testing is described in Section 5.4.3.1.

5J.8.2 (DRL 5.3.2) Indicate the specifications to be used for the concrete aggregate and indicate the testing to be performed to assure the suitability of the selected aggregate. Indicate the specifications to be applied to the mixing water and the limits to be prescribed on agents which may attack prestressing tendons.

ANSWER Specifications for concrete aggregate are indicated in Section 5.1.3.1 and acceptability tests are listed in Section 5.4.3.1. Mixing water for structural concrete is described in Section 5.1.3.1. No further limits will be prescribed for corrosive agents due to the fact that prestressing tendons will not be in contact with the concrete. The aggregates in the Sacramento area are granitic and "nonreactive".

5J.8.3 (DRL 5.3.3) Describe the concrete mix procedures and indicate the scope and extent of testing of trial mixes. Indicate the type and extent of admixtures which may be used. Describe their purposes, their extent of compliance to ASTM specifications, and their testing. Describe the choice of slump values and list them.

ANSWER With the exception of the slump values the answer to the question is found in Paragraph 5.1.3.1.

At this time the concrete mixes have not been designed. Proper slump will be determined when the mixes are designed. Based on construction experience in the Sacramento area, there will be no problem in obtaining concrete of the high quality required for this project.

0002 ~~00280~~

5J.8.4 (DRL 5.3.4) Indicate, in detail, the extent to which splice stagger will be achieved.

ANSWER The criteria for splice stagger was presented in the answer to question 5J.1 Amendment No. 1.

The only anticipated exception will be for the nominal temperature reinforcing provided for crack control.

5J.8.5 (DRL 5.3.5) Indicate the extent to which splicing of reinforcing steel will be made by welding. State the location of those welds.

ANSWER See Question 5J.1

5J.8.6 (DRL 5.3.6) Add the description of the "splicing" of inclined bars, or horizontal stirrups provided to take the radial shears in the walls, with the vertical bars. If the "splicing" is done by lapping the diagonal bar with a vertical bar, or by bending the stirrup around a vertical bar, demonstrate that, despite biaxial tensile stresses in concrete and vertical and horizontal crack pattern, the load in the diagonal bars or stirrups can safely be transmitted to the vertical bars.

ANSWER: Other than at lapped diagonal splices, vertical cracks have little or practically no effect on transfer of radial shear. Horizontal cracks do have a significant effect on transfer of radial shear. Under these circumstances, the provisions of ACI Code 318-63 will be applicable to the problem of transfer of load from the diagonal bars or stirrups to the vertical bars. Special precaution will be taken at places of lapping diagonal shear bars with vertical bars, as explained in the article, "Design of Auxiliary Reinforcement In Pre-Cast Concrete Connections," by Robert F. Mast (Presented at the American Society of Civil Engineers Structural Engineering Conference, Miami Beach, Florida January 31, 1966). Also see answer to question 5J.7.12.

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5J.8.7 (DRL 5.3.7) Specify quality control for the strength welds of reinforcing bars to structural elements such as plates, rings, sleeves, and for occasional strength weld splicing of heavy reinforcing bars.

ANSWER Quality control for all field welds will follow procedures outlined in Appendix 5H. Procedures for placing of reinforcing bar using the Cadweld process are specified in Appendix 5C. Welding of reinforcing steel, if required, will be performed by qualified welders in accordance with AWS D12.1 "Recommended Practice for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction."

5J.8.8 (DRL 5.3.8) Provide the detailed material selections for containment penetrations, listing the corresponding ASTM specifications and indicating the NDTT considerations in their selection.

ANSWER The selection of materials for the containment penetrations is given in Section 5.1.3 including appropriate ASTM designations. The impact testing of penetrations will be accomplished in accordance with the requirements of Section III, Nuclear Vessels, of the ASME code specifically those of Paragraph N-1211. Section 5.1.4.9 of the PSAR gives other general information pertaining to this aspect of penetration material selection.

5J.8.9 (DRL 5.3.9) Provide a detailed description of the prestressing materials and hardware selected. Justify the prestressing system selection. This should include data with regard to ultimate tendon strength, elongation, anchorage strength, hardware dynamic performance, conduits, etc.

ANSWER A description of the prestressing materials, hardware, anchorage strength, dynamic capabilities, and sheathing is shown in Sections 5.1.3.3 and 5.4.3.4. The specific system has not been selected for this project. Detailed data are presently being requested from all potentially qualified suppliers.

~~00282~~

QUESTION 5J.9 (DRL 5.4) — CORROSION PROTECTION

5J.9.1 (DRL 5.4.1) — Describe the concrete cover provisions for reinforcing steel for the dome, slab, and cylinder. Include, for comparison, the minimum ACI 318-63 code requirements.

ANSWER — The concrete cover provisions for reinforcing steel and prestressing has been described in tabular form along with ACI minimum values in Table 5.1-1, submitted as a portion of Amendment 1.

5J.9.2 (DRL 5.4.2) — Discuss the extent to which cathodic protection has been considered and is being provided. State whether soil resistivity surveys have been conducted and, if so, provide the results.

ANSWER — To insure the safe and functional integrity of all structures, it will be necessary to evaluate the corrosive environment inherent in the site. In general, the electrochemical levels of all metals and the conductivity of the surrounding soils will be investigated to insure that corrosion will either be non-existent or that it will be controlled by a cathodic protection system specifically designed for the site and monitored by plant personnel.

Measurement of media resistivity will be accomplished by means of standard megger techniques. This will be followed by soil resistivity-probability plots so that definite soil classifications can be determined. Specific cathodic protection systems, if required, will be designed and located for maximum plant protection.

Additional corrosion protection for the exposed surface of the Reactor structure liner will be included in the form of surface cleaning, shop prime coat applications, and a final field finish coat on the exposed surface.

5J.9.3 (DRL 5.4.3) — Discuss the extent to which protective coatings will be applied to the liner.

ANSWER — Reference is made to Section 5.1.3.4 which describes the general approach to the liner plate protective coatings. The particular coatings for the liner plate are currently in a state of re-evaluation. Specifically under the sponsorship of the ASME and/or the ANS a committee is being formed through the Oak Ridge National Laboratory with the purpose of writing a protective coatings standard for reactor containment facilities as a USASI Standard.

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~~00283~~

The committee is comprised of USAEC licensing, USAEC contractor, coating manufacturer, reactor designer, and architect-engineering representatives. Bechtel Corporation has been invited to send representatives to the sessions commencing on May 9, 1968.

5J.9.4 (DRL 5.4.4) Discuss the corrosion protection of the prestressing system.

ANSWER The prestressing system will be protected against corrosion for the design life of the plant, using a petrolatum coating during transit and installation and a permanent sheathing filler material for the duration of the structural life of the containment. These protective greases are outlined in detail in Section 5.1.3.3.

5J.9.5 (DRL 5.4.5) Drainage provisions do not include a layer of porous concrete located at base. Also no provision has been made for a porous layer at the cylindrical wall of the containment. Justify the omission of drainage at such a critical location. Consider that, contrary to normal foundation work, the containment structure is continuously subjected to the effect of thermal gradients, which generate tensile stresses in the outside concrete layers and increase the danger of cracking.

ANSWER Special provisions for drainage in the form of a porous concrete layer have been evaluated for the Rancho Seco site. Due to the existing soil conditions, the low water table, and the characteristics of the containment wall provisions of this type are not visualized.

The containment structure wall prestressing is designed including the effects of both operating and accident thermal gradients. The base mat is not prestressed, however, it is extremely thick and heavily reinforced for all postulated accident conditions. Thermal gradients have been evaluated and give indication that thermal gradients both operational and postulated induce only local effects which would not affect the cracking tendency of the mat. Therefore, outside layers will not experience large tensile stresses under these conditions. The reinforcing that connects the base of the cylinder is designed to 0.5 Fy including operating thermal load initial prestress loads and dead loads. At this stress level cracking will be no significant than conventional building structures without thermal stresses.

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0006

QUESTION 5J.10 (DRL 5.5) CONSTRUCTION

GENERAL STATEMENT

The Rancho Seco plant will be constructed by contractors selected by public bidding. The specification on which the award of construction contracts are based will be specific as to the quality of work required.

The construction methods outlined previously and in the answers to the following questions demonstrate acceptable methods of construction, but are not intended to preclude other equally acceptable methods.

Experience gained on similar plants presently under construction will be used as a guide in determining what methods are acceptable.

The latitude available to the Contractor through normal procedures will be limited to construction methods which do not result in a change to the design criteria presented in the PSAR and the quality of work required both in the PSAR and the contract specifications.

Proposed changes will be reviewed in detail by both SMUD and the Engineer-Construction Manager. Changes which are approved will be fully documented. A complete set of records will be maintained at the plant site by SMUD for future reference. 4

If a construction method is proposed by the Contractor which would result in a change to the criteria and it is deemed that the change would be beneficial to the overall plant, SMUD will apply for an amendment.

5J.10.1 (DRL 5.5.1) Present a preliminary construction schedule.

ANSWER The preliminary construction schedule is shown in Figure 5J.10.1-1.

5J.10.2 (DRL 5.5.2) Indicate the codes of practice that will be followed in the containment construction.

ANSWER The construction of the containment will follow codes of practice listed in Section 5.4.2 and 5.4.3 as well as related practices described in Section 5.4.4.

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5J.10.3 (DRL 5.5.3) Indicate where and to what extent ACI 301 standard practice for construction will be exceeded, met, or not followed.

ANSWER All structural concrete work will meet or exceed ACI-301, "Specifications for Structural Concrete for Buildings".

Where this specification offers options as to the type of construction, the construction contract specifications will be explicit in what is required.

5J.10.4 (DRL 5.5.4) Indicate the specific extent to which ASME fabrication standards will be adhered to in liner manufacturing.

ANSWER The fabrication standards for liner manufacturing is outlined in considerable detail in Sections 5.1.3.4, 5.1.4.9, and 5.4.3.5. Also refer to the answer to Question 5J.10.22 for additional information.

5J.10.5 (DRL 5.5.5) The listing of codes should be supplemented with an additional list of codes covering items which are not covered in listed codes (Army Engineers, Bureau of Reclamation, AWS, etc.) but which may be used as basis for applicant's specifications to contractors. State the basis on which these supplementary, mandatory requirements for the contractors will be prepared.

ANSWER Selected codes and specifications which are used in specialized areas are listed in the paragraph describing the pertinent structure of interest. Attention is directed to specific design areas as described in the answers to Questions 5J.6.7, 5J.7.3, 5J.9.3, 5J.22, 5J.25 as well as Section 5 and Appendix 5A of the PSAR.

The contract specifications will provide the detailed requirements for the contractors. These specifications will cover all areas of work and will be the mandatory requirements.

5J.10.6 (DRL 5.5.6) ASME Standards define erection tolerances in a way that is not sufficient to ensure a satisfactory erection of the liner. For example, they do not cover local curvature deviations. Establish a comprehensive set of erection tolerance standards for the liner, specifying all inaccuracies likely to occur during erection.

ANSWER ASME Standards to define erection tolerances are not directly applicable since the liner plate is a leak-tight membrane not a pressure vessel and is not constructed under ASME jurisdiction. However, the ASME Code was used as a guideline in establishing dimensional tolerances which will be adhered to in the liner construction. These tolerances have been clarified in Amendment 1 to Section 5.4.3.5.

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0008

ITEMS*

	EXCAVATION
2.	BASE SLAB & TENDON ACCESS GALLERY
3.	CONTAINMENT WALLS & LINER
4.	SLAB LINER PLATE
5.	INTERIOR CONCRETE
6.	DOME SUPPORT STEEL & LINER PLATE
7.	ERECT POLAR CRANE
8.	INITIAL DOME CONCRETE
9.	DOME REBAR & TENDON TUBES
10.	MAIN DOME CONCRETE
11.	PLATFORMS, STAIRS & HANDRAILS
	ABOVE GRADE
	BELOW GRADE
12.	CLOSE CONSTRUCTION OPENING
	INSTALL & TENSION TENDONS
14.	CAP CONCRETE TENDON ANCHORAGE
15.	PIPING & VALVES – ALL SYSTEMS
16.	MISC. EQUIPMENT, INSTRUMENTS & CONTROLS
17.	ELECTRICAL INSTALLATION, CABLING & CONNECTIONS
18.	RECEIVE & SET HEAVY EQUIPMENT
19.	PAINTING & THERMAL INSULATION
20.	INSTALL REACTOR INTERVALS
21.	COLD HYDRO TESTS
22.	CONTAINMENT STRUCTURE STRENGTH AND LEAK RATE TEST
23.	HOT FUNCTIONAL TESTS
24.	CONTAINMENT LOAD FUEL

*FROM CPM SCHEDULE DATED 4/68

~~00287~~

0009

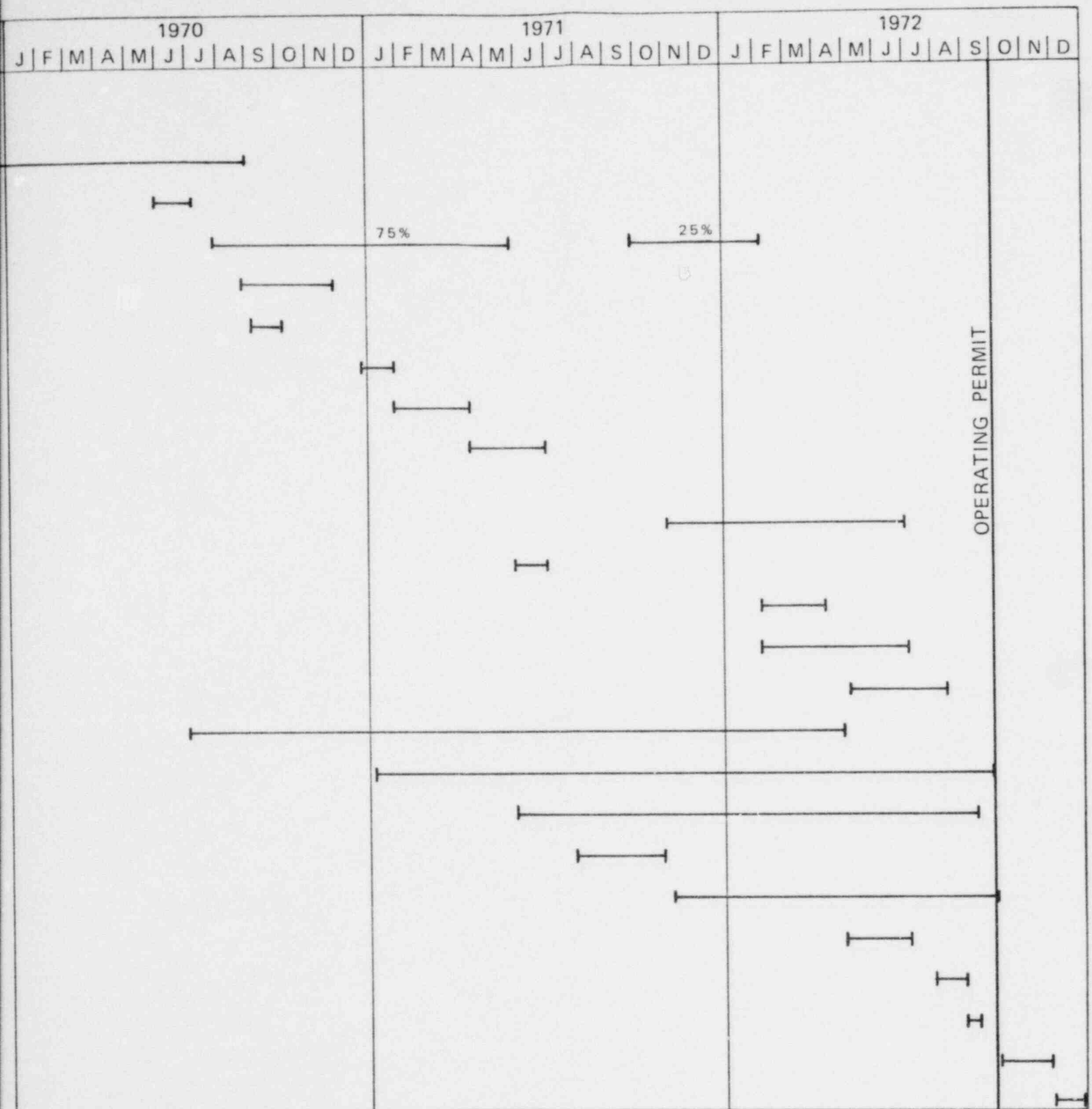

FIGURE 5J.10.1-1
CONSTRUCTION SCHEDULE
FOR REACTOR BUILDING

0010

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SMUD
SACRAMENTO MUNICIPAL UTILITY DISTRICT

5J.10.7 (DRL 5.5.7) Describe in more detail the general construction procedures and sequence that will be used in construction of the containment. Include excavation, ground water control, base slab construction, liner erection and testing, concrete construction in cylinder and dome regions, prestressing systems erection and prestressing sequence.

ANSWER The general construction procedures to be used in the erection of the containment structure are referenced as follows:

(1) Excavation and ground water control - Refer to the answer to Question 5J.6.6.

(2) Base slab construction - The base slab will be erected using placing techniques typical of large mat foundations. The access gallery will be placed first in six circumferential segments following the circumferential periphery in alternating segments. This is followed by the placing of concrete within a central "dollar" section. Following this, the mat is completed in single-lift alternating sections with vertical joints of expanded metal lathe. Bars are grouped where required to provide 1 ft 6 in. x 1 ft 6 in. openings for drop chutes extending from concrete conveyors. Carefully leveled tee-bars are set to provide screeds for the base pour and for accurate attachment of the base liner plate.

(3) Liner erection and testing and wall and dome construction - Refer to Section 5.4.4.3 as further clarified in Amendment 1 as well as the answer to Question 5J.10.12.

(4) Prestressing sequence - Refer to Section 5.4.4.2 of the PSAR as modified in Amendment 3 to reflect the preliminary prestress sequence planned for the Rancho Seco Project.

0011 ~~00289~~

5J.10.8 (DRL 5.5.8) Provide a detailed description of the erection of the bottom liner. Describe the provisions that will be made to ensure a good bearing of the liner on concrete below. State if grouting will be resorted to and how the liner plates will be fitted to the embedded anchors.

ANSWER Intimate bearing of the liner plate on the structural foundation slab is not considered to be a requirement for the following reasons:

1 It is insulated from any significant thermal gradients relative to the foundation slab.

4 2. The placement of the cover slab over the top of the liner plate will provide constraint of deformations perpendicular to the plate surface that might be caused by differential thermal gradients.

3. Whenever structural loads are transferred to the foundation slab specific structural inserts will be provided or the bearing and flatness of the plate will be checked to ensure that the load can be transferred by direct bearing.

Grouting will not be required. The erection of the floor liner plate will be as follows:

1. Structural steel shapes, probably wide flange beams, will be set at the liner plate floor level and the structural base slab concrete will be placed flush with the top of the embedded structural steel members.

2. The liner plate will be placed on top of the concrete, with the plate seams directly over the embedded steel members, and the seams will be welded.

3. The liner plate will be tested and the leak chase system installed and pressurized.

4. The 18" concrete cover will be placed.

5J.10.9 (DRL 5.5.9) Describe the procedures for concrete placing and curing.

ANSWER The procedures for concrete placement and curing are given in the answer to Question 5J.10.11 and will follow ACI recommended practices outlined in detail in Section 5.4.2.

~~00290~~

5J.10.10 DRL 5.5.10) — Describe the procedures for bonding between lifts.

ANSWER — Bonding between lifts is described in Section 5.4.4.1.

5J.10.11 (DRL 5.5.11) — Indicate the manner in which concrete lifts will be placed and staggered.

ANSWER — The placement of concrete lifts in the containment structure is described in conjunction with the erection of the liner plate in Section 5.4.4.3 as modified by Amendment 1.

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5J.10.12 (DRL 5.5.12) Give a detailed description of the placing of concrete in the dome, especially near the center portion of the dome.

ANSWER The following construction sequence is currently being used for the erection of the dome where the shell is lightly stiffened and truss-supported:

a. The trusses are erected on the adjustable support brackets after completion of the cylindrical wall construction.

b. The dome liner plate is erected in continuous operations using truncated conical plate segments working toward the high point of the dome.

c. Vertical supports are welded to the liner plate stiffeners for aligning and supporting the dome tendon sheathing.

d. A seven inch layer of concrete is placed over the entire dome surface to be used as additional support for the remaining dome concrete. Expanded metal is used as the form on the sloping portion of the roof.

e. The tendon sheathing and reinforcing steel is installed and positioned.

f. The placement of the remaining concrete commences with the construction of the ring girder in approximately three lifts following the pattern established in the walls.

g. The next step is the placement of approximately one-half of the remaining surface area of the dome in a complete ring with operation continuing toward the dome apex.

h. The center "dollar" section of the dome, in one lift, completes the structural concrete operation.

i. Concrete covers are placed over the protruding tendon anchorages upon completion of all tensioning operations. This operation is substantially later than step h.

Currently, studies are being carried out to provide a heavily stiffened liner plate which is self-supporting. The selection of this type of dome shell is a distinct possibility and may result in the modification of the above construction procedure and a more efficient construction schedule.

0014 ~~00292~~

5J.10.13 (DRL 5.5.13) Indicate how concrete will be placed in zones with congested reinforcing pattern.

ANSWER Placement of concrete will be in accordance with practices described in Chapter 6 of Building Code Requirements for Reinforced Concrete (ACI-318). In confined areas with congested reinforcing pattern and around embedded items special precautions and techniques will be employed.

Maximum aggregate size will be reduced where required and effective vibration will be assured by inspection. Concrete will not be dropped through dense reinforcing steel, but will be placed using spouts, elephant trunks, or other suitable means.

5J.10.14 (DRL 5.5.14) Describe the extent of concrete compression and slump testing to be used. Include the statistical basis for the proposed program and the standards for batch rejection and pour removal.

ANSWER The extent of concrete testing, including the basis of the program and standards for rejection, is described in Sections 5.1.3.1 and 5.4.3.2. ACI-214 provides a statistical basis for rejection of concrete. Requirements for "excellent" control will be the standard.

5J.10.15 (DRL 5.5.15) Indicate the planned program for user testing of reinforcing steel for strength and ductility. Include the statistical basis for the program and the basis for reinforcing steel shipment rejection.

ANSWER For user testing please refer to Section 5.4.3.3. The requirements set forth in this section equal or exceed those required by the ASTM Specification covering this material. Bend tests will not be required for No. 14S and No. 18S bars.

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5J.10.16 (DRL 5.5.16) Indicate the controls that will be provided to ensure that the proper specification reinforcing bars are received, at the site and, if different grades of steel are used, how errors will be avoided during construction.

ANSWER The only reinforcing steel used in the containment structure will be intermediate grade reinforcing steel under ASTM Specifications A-15 and A-408, and high strength reinforcing steel under ASTM Specification A-432 and A-431.

Each reinforcing steel bar will have The American Standard 'ar marks indicating new billet steel, the bar size, and whether it is high strength. High strength bars will have the minimum yield point indicated as a part of the standard marking. Bars without a yield point mark indicate intermediate grade A-15 or A-408. Hard grade is no longer rolled and structural grade is furnished only on special order under A-15 or A-408.

The ASTM Committee A-1, Subcommittee 5, on Reinforcing Steel has already approved a new specification for intermediate grade steel, ASTM A-615-68, dated February 14, 1968. Under this new specification, structural and hard grade steel under A-15 and A-408 are eliminated and the intermediate grade steel will be bar marked 40 to indicate its yield point. Under this specification, which will be published July 1, 1968, there will be no difficulty identifying different grade bars.

Inspection of the reinforcing steel will take place at delivery and at erection to assure that the correct specification and size bars are used in the proper locations.

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5J.10.17 (DRL 5.5.17) Describe the reinforcing bar welding procedures and associated quality control to be used in performing reinforcing bar strength welds. Include bar preparation, user verification testing for the reinforcing steel composition, maximum permissible alloy specifications, temperature control provisions, radiographic and strength testing requirements, and the basis for welded splice rejection and cut-out. Will any tack welding of reinforcing steel be permitted?

ANSWER In general welded splices will not be used. If welding is necessary, it will be done in accordance with AWS D12.1-61 "Recommended Practices for Welding Reinforcing Steel, Metal Inserts and Connections in Reinforced Concrete Construction", as stated in Section 5.1.3.2. This specification describes procedures for bar preparation, temperature control, and other associated criteria required for proper welding of reinforcing steel. Hi-strength reinforcing steel will be joined by lap splices or Cadweld, as outlined in Appendix 5C. Please refer to the answer to Question 5J.1.

5J.10.18 (DRL 5.5.18) Indicate the minimum percentage of reinforcing splices to be checked by welding inspector, using nondestructive inspection methods (X-raying, dye penetrant test, etc.).

ANSWER In general welded splices will not be used, as stated in Section 5.1.3.2 and the answer to question 5.J.1 of Amendment 1.

5J.10.19 (DRL 5.5.19) Describe the general sequence of liner erection and testing in relationship to the structural concrete construction.

ANSWER The liner plate is fabricated on horizontal welding jigs away from the wall area. Prior to fitup a prefabricated section is made long enough to reach from buttress to buttress. This section contains adequate stiffeners, which are spaced to prevent progressive liner buckling from compressive stresses originating from the design temperature and prestress. These stiffeners provide sufficient rigidity to permit handling and positioning. Upon completion of the welding, these segments are completely tested and inspected prior to the erection of tendon sheathing and reinforcement and the subsequent placement of concrete. In general, the fabrication of the liner plate leads the placement of concrete by two lifts (20 ft).

5J.10.20 (DRL 5.5.20) Indicate the controls to be employed in reference to liner plate out-of-roundness and local bulges.

ANSWER The specific dimensional variations permitted in the liner plate are demonstrated in Section 5.4.3.5, as amplified by Amendment 1.

5J.10.21 (DRL 5.5.21) Indicate the extent of user verification testing of certified liner NDTT properties, liner thickness, ductility, weldability, etc.

ANSWER User verification and testing of certified liner properties are discussed in Section 5.1.3.4 and 5.4.3.5. In general, certified copies of mill test reports describing the chemical, mechanical, and physical properties of liner plate steel will be submitted to the user for approval. Tests for qualifying welding procedures will be performed by the fabricator and monitored by the user. These tests will provide confirmation on weldability and weld ductility. The user will not duplicate tests performed by the steel supplier or the fabricator.

5J.10.22 (DRL 5.5.22) Indicate the applicable ASME or API code sections that will be adhered to in liner erection.

ANSWER All components of the liner which must resist the full design pressure, such as penetrations, shall be designed, constructed, inspected, and tested in conformance with the requirements of Subsection B of Section III, of the ASME Code.

The design, construction, inspection, and testing of the liner plate is not covered by any recognized code; however, the liner plate and structural shapes will be supplied to the requirements of ASTM A442 and ASTM A-36 respectively.

Design and construction of the liner plate will conform to the applicable portions of Part UW of Section VIII, of the ASME Code. Specifically, paragraphs UW-26 through UW-38 inclusive, will apply in their entirety. In addition, the qualification of all welding procedures and welders will be performed in accordance with Part A of Section IX, of the ASME Code.

5J.10.23 (DRL 5.5.23) Indicate the procedures and criteria for control of seam weld porosity.

ANSWER Seam weld porosity will be controlled by the adaption of Section IX of the ASME Code. By adopting this section, the liner will be welded by qualified welders, using strict pressure vessel construction criteria. The strict quality control program outlined in Appendix 5H will further remove any danger of seam weld porosity.

All liner plate seams will be 100 percent vacuum box soap bubble tested to check for any weld porosity. The seams will also be checked by 10 percent radiographic inspection. The criterion for radiographic acceptance of welds will be in accordance with Paragraph UW-51 Section VIII of the ASME Code, except that the maximum acceptable length of slag inclusion will not exceed 0.125 in., whereas the code allows .250 in.

5J.10.24 (DRL 5.5.24) Indicate the requirements that will be placed on seam and anchor welds to assure ductility.

ANSWER The adoption of Section IX of the ASME Code will necessitate the testing of welded transverse root and face bend samples in order to verify adequate weld metal and parent metal ductility. The ability of the test samples to withstand the cold 180°F test bends will be considered ample evidence of weld ductility. The testing of the anchor welds will also be made according to appropriate sections of the same code as evidence of complete weld ductility and material compatibility.

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5J.10.25 (DRL 5.5.25) Discuss the seam weld radiography program. Also, provide an evaluation of the liner radiography with respect to providing assurance that flaws which may develop into positive leakage paths under design basis accident conditions do not, in fact, exist.

ANSWER: Radiography of the liner seams shall be conducted in accordance with Paragraph UW-51 of Section VIII of the ASME Code. At least one spot radiograph shall be taken in the first 10 feet of welding completed in the flat, vertical, horizontal, and overhead positions by each welder. Thereafter, approximately 10 percent of the welding will be spot examined on a random basis in such a manner that an approximately equal number of spot radiographs will be taken from the work of each welder.

Radiography is not recognized as an effective method for examining welds to assure leak tightness. Therefore, the only benefit which can be expected from radiography in connection with obtaining leak-tight welds is an aid to quality control. Random radiography of each welder's work will verify whether welding is under control and is being done in accordance with previously established and qualified procedures. Additionally, employing random radiography to inspect each welder's work has been proved by past experience to have a positive psychological effect on improving overall welding workmanship.

5J.10.26 (DRL 5.5.26) Describe the quality control procedures for liner angle and stud welding.

ANSWER All welding shall be performed in strict accordance with approved welding procedure specifications. All welders shall be qualified by performing the tests required by Bechtel Welder Performance Specification WQ-F-1 (Conforms to ASME Section IX). No welder shall be permitted to perform production welding until he has passed the necessary tests and has the appropriate Bechtel Welder Performance Qualification Test Record, WR-1 (Conforms to ASME Section IX), on file at the jobsite. All liner angle and stud welding shall be visually inspected prior to, during, and after welding to insure that quality and general workmanship meets the requirements of the applicable welding procedure specification.

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5J.10.27 (DRL 5.5.27) Describe those quality control procedures and standards for field welding of the liner plate that differ from the general procedures and standards, include welder qualifications, welding procedures, post weld heat treatment, visual inspection, magnetic particle inspection, liquid penetrant inspection, and construction records.

ANSWER The quality control procedures and standards outlined in detail in Appendix 5H represent those in full compliance with acceptable codes, particularly the ASME Code. These procedures will permit full design requirements and job specifications to be satisfied and no additions are required or envisaged.

5J.10.28 (DRL 5.5.28) Indicate the factory quality control requirements that will be imposed on the prestressing system to ensure that production materials will meet design requirements and specifications.

ANSWER Specific details on factory quality control requirements on the prestressing system are detailed in Section 5.4.3.4.

5J.10.29 (DRL 5.5.29) Describe the corrosion protection that will be given to the prestressing wire or strand at the factory, through transportation, and in the structure prior to prestressing.

ANSWER The prestressing wires will be protected during shipment and installation by coating them with a thin film of petrolatum containing rust inhibitors, as outlined in Section 5.1.3.3.

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5J.10.30 (DRL 5.5.30) Describe the extent to which the tendon corrosion inhibiting wax or grease will be tested to ensure that no substances deleterious to the tendons are present.

ANSWER Manufacturer will submit tests on physical and chemical properties for every batch of factory production. Field Quality Control will consist of a representative sample taken at frequency of one sample per 5000 gallons and tested by an independent laboratory. The compound to be detected in the field quality control, the limits of test accuracy and methods of test are tabulated in Table 5J.10.30-1.

TABLE 5J.10.30-1

Compound	Allowable Maximum	Method of Test
1. Water Soluble Chlorides (Cl)	5.0 ppm	ASTM D-512-62T (Limit of Accuracy 0.5 ppm)
2. Water Soluble Nitrates (NO_3)	0.05 mg per liter	ASTM D-992-52 (Limit of Accuracy 0.01 mg per liter)
3. Water Soluble Sulfides (S)	5.0 ppm	ASTM D-12-55 (Limit of Accuracy 1.0 ppm)

5J.10.31 (DRL 5.5.31) Indicate the scope and extent of quality control testing of anchorage components and production anchorage assemblies.

ANSWER The scope and extent of quality control testing of anchorage components and assemblies will follow the procedures outlined in Section 5.1.3.3 and 5.4.3.4.

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QUESTION 5J.11 (DRL 5.6) CONSTRUCTION INSPECTION

5J.11.1 (DRL 5.6.1) Indicate the degree to which material preparation and construction activities will be subject to inspector surveillance.

ANSWER All material received at the jobsite and all construction activities will be subject to inspector surveillance.

The inspector surveillance of material preparation will depend upon the type of material. All large fabricated or specialized components will be shop-inspected.

Materials which are produced in large quantities for use throughout the construction industry will normally not be inspected during their manufacturing process. Certified test results will be required on these materials. Jobsite and laboratory testing will be used to verify that the material received conforms to the specifications.

Appendix 1B, "Quality Assurance Operations," outlines how the above will be accomplished.

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5J.11.2 (DRL 5.6.2) Discuss the manner in which records of quality control and inspection will be kept.

ANSWER Equipment, materials, manufacturing and construction processes to be subject to quality assurance operations are identified and requirements defined by the quality assurance engineer early in the detailed design phase of the project. Appropriate records are taken by the SMUD/Bechtel vendor equipment inspectors and field engineers, refer to Figure 1B-1. These records are taken in quadruplicate on pre-printed forms which require attention to relevant details.

The quality assurance engineer receives all four copies of the test/inspection report, reviews and confirms or rejects the inspector/field engineers conclusion and distributes copies as follows:

Original Inspector/Field Engineer	- 1
QAE Files at Jobsite	- 1
Project Engineering Group for Review	- 1
SMUD Files for Record	- 1

On completion of the project, the QAE files will be turned over to the station superintendant along with the station startup test records. Thus SMUD will retain two sets of records, one at the jobsite and one at the head office.

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5J.12 (DRL 5.7) TESTING AND IN-SERVICE SURVEILLANCE

5J.12.1 (DRL 5.7.1) Describe the sequence for structural testing.

ANSWER The PSAR Appendix 5I describes the containment instrumentation and provides general comments on the program for structural testing. It also describes the means for taking measurements in as much detail as reasonable until the design and construction planning is further advanced.

The general sequence is given in Reference 1. More specifically, the measurements will be made at several stages in the prestressing sequence and upon completion of post-tensioning. When the structural proof test is made, measurements will be made at five ascending and five descending pressure levels.

5J.12.2 (DRL 5.7.2) Describe the instrumentation program for structural testing, including:

(a) Identification of structural, and liner areas to be instrumented;
(b) Purpose, type, expected accuracy, and redundancy of instrumentation;
(c) The range of strains and deformations expected;
(d) The protective measures that will be taken to ensure instrument performance during structural testing, considering the interval between instrument installation and its use.

ANSWER: Appendix 5I, "Containment Structure Instrumentation," describes the planned program and expected results.

5J.12.3 (DRL 5.7.3) Evaluate the extent to which the test pressure will simulate design basis accident conditions by comparing the stresses under various test pressures with those in the structure under: (a) accident pressure plus temperature gradient, and (b) accident pressure plus temperature gradient, plus earthquake, (or other combinations, if governing), for the following structural elements: (a) circumferential reinforcing and prestressing; (b) axial (longitudinal) reinforcing and prestressing; (c) dome reinforcing and prestressing; (d) base slab reinforcing; and (e) large openings. Indicate the corresponding concrete stresses.

ANSWER: An answer to this question will be provided, if requested, as the design analysis are completed. However, please refer to the answer to question 5J.12.4 for an evaluation of the load conditions simulated by the proof test pressure.

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5J.12.4 (DRL 5.7.4) By comparing stresses and strains which are experienced by the structural elements under test loadings with those calculated to exist under design basis accident loading, provide a discussion in support of the selected test pressures. Include in this discussion the extent to which increased test pressure or design modifications might be considered in an effort to obtain closer test verification of structural integrity.

ANSWER The appropriateness of various test pressures was considered in the preparation of the PSAR Appendix 5B (Ref. 1) where a brief discussion is given. The intended test pressure is 1.15 times design pressure, a level which exceeds that deemed necessary for the purpose of insuring structural integrity. Assuming that the question requires elaboration on the Ref. 1 discussion, a more detailed account is given here.

The requirement that a pressure test be made is reasonably well indicated having its origins in ASME Boiler and Pressure Vessel Code, Section 3, Class B (Ref. 2). The inferred purpose for a pressure test is to demonstrate structural integrity independent from any analytical conclusion. Another pressure test requirement exists, for determining the leak rate from the containment, but is not specified by Ref. 2.

The ASME Code gives minimal guidance in any respect except the pressure levels for the tests. However, it does specify the required pressure for verifying the structural integrity of steel vessels and common practice requires leak rate tests at design pressure. Ref. 2 does not give guidance as to test phenomenon to be measured, nor for the interpretation standards that are useful for prestressed concrete containments. Substantial data, however, is available for measurements during the leak rate test.

Indirect verification is generally permitted for both structural integrity and leak rate tests. An indirect verification is considered to be one obtained by inferring that since integrity is shown for one or a combination of load conditions, it also exists for one or more load conditions which were not created by the test. Indirect verifications are evidently used to verify structural adequacy before any pressure tests are made since, because of the schedule consequences of lack of integrity, some evidences of integrity must be available to guide decisions to start the pressure test.

Large psychological influences on the selection of test pressures are considered in common practice, especially for satisfying opinions of the uninformed observer as to appropriateness. It is therefore advisable to follow common practice even though there is not a strong technical need. If these testing procedures are deviated from, there should be strong technical justification so as to compensate for the effort required to alter the opinion of the uninformed observers.

The following conclusions from the review of current testing practice are appropriate:

1. Pressure tests, at the pressure levels commonly used, are required unless there is a strong technical justification for an alternative.

2. It will be difficult to show a technical justification for departing from current procedures because the justification for their use is not well defined. Comparisons of technical needs for the tests between steel and prestressed concrete containment will thus be difficult.

3. Common testing practice should not be followed implicitly. Instead, the needs for prestressed concrete containment pressure tests should be determined independently. Comparisons should then be made with needs determined from common practice. Judgments can then be made as to whether or not justification exists for following normal procedures.

From a review of the needs for verification of structural integrity of prestressed concrete containments, it is apparent that the greater need exists prior to any pressure test because of the schedule delays which could occur.

Verification of structural integrity by analysis is a consideration but not deemed sufficient by itself since it assumes such things as the existence of the needed quality of materials and construction work, which are not obtainable by analysis.

Quality control and assurance for material and workmanship, such as used in common practice, are considered as one means of verification independent from analysis. They give only an indirect verification of structural integrity since no significant structural loads result from their use. However, for prestressed concrete containment, the stressing of each individual tendon provides direct verification of integrity at loads larger than predicted by analysis for all other design loading cases. This is due to the fact that the structure is loaded, at completion of posttensioning, with prestressing loads which are larger than subsequent prestressing loads. Hence, the structural integrity for prestressing loads and dead load are directly verified by test.

Indirect, or inferential, verification of structural integrity is also possible with observations during posttensioning since, for example, if deformations are similar to those predicted by analysis, inferences can be drawn that the analysis also provides reasonable predictions of structural integrity for other load conditions.

At completion of posttensioning, it is clear that more evidence is available concerning direct verification of structural integrity than for steel pressure vessels at the end of erection but prior to pressure test. The evidence available consists of material and workmanship quality control and assurance for both steel pressure vessels and prestressed concrete containment. For concrete containment, however, the additional verifications possible are individual tests of the tendons and the observation of structural response to the total prestressing loads as one load condition that is large compared to the structural strength. The latter verification is not possible for common steel pressure vessels until the pressure tests have been made.

At this stage in the evaluation, a tentative conclusion is possible that there might be no technical need for pressure testing the prestressed concrete containment structures to determine structural integrity, although the psychological impact of opinion inferred from common practice could overrule the technical considerations. However, such a technical conclusion would be supported primarily by indirect or indirect verification. This might or might not be suitable, depending on the number and degree of inferences required for extrapolating the structural response for other loads from the structural response at test loads.

The probable number and degree of inferences expected may be assessed by comparison of analytical results for the various load conditions. The comparisons of interest at this point are predicted strains, deformation and tendon loads for the prestressing and dead load conditions with similar data for the design basis accident load conditions.

The comparisons of other tests indicated that the predicted tendon loads do not change from the prestressed to design basis accident load conditions so as to exceed loads which had been previously imposed on the tendons during posttensioning. The comparisons indicated that, for design basis accident conditions, the inner 1/3 or so of the concrete thickness is generally in compression but at a lower level than for the prestressed load condition. Hence, the prestressing, load conditions appeared to provide a better test of concrete structural integrity for compression than would design basis accident conditions.

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The magnitude of predicted deformations for the comparison are similar in value but essentially opposite in sense. However, the design basis accident condition loads cause the deformed shape to approximate the shape prior to prestressing (i.e., that of a containment loaded only with dead load). The predicted deformations are not identically opposite in magnitude or sense, but, for the usual capabilities for measuring the phenomena, they could be so considered.

The major difference in predicted behavior between the prestressed and design basis accident load conditions is for the magnitude and direction of predicted strain at the concrete at the outer surface of the concrete, where tension is predicted for design basis accident conditions.

The interference of the structural integrity (that exists with the outer surface tension predicted for the design basis accident load condition and from observations of outer surface compressions at prestressed plus dead load conditions) may be considered the greatest degree of inference needed for this series of comparisons. The significance of this degree of inference is, therefore, evaluated by examining the significance of the predicted outer surface tension.

A significant point from the evaluation is that the structural integrity is not reduced by the tension predicted for the design basis accident. There is still average compression in the cylinder and dome should the depth of tension cracks at the outer surface increase, they will not propagate through the whole section. At the most then, the result of the increase in concrete cracking would be a local increase in the stress of the reinforcing steel in the cracked zone as the forces which limit the crack width and depth increase with the concrete cracking. The maximum change in the structural response between the two load conditions would be a slight "softening" of the structural stiffness for resisting flexure when the concrete is cracked by the tension.

The "softening" effect is controlled by the prestressing and reinforcing steel and for this design does not reduce force equilibrium but does change the resultant stress distribution.

The tentative conclusion, that structural integrity at design basis accident conditions could be inferred from observations made during prestressing, is valid. It is evident, however, that opinions, formed by common practice, might reject a technical argument based on logic without corroborating observed evidence.

This evidence is available from the reactions of prestressed concrete structures, such as bridge members to cracking loads where the cracking is caused by loads that do not decrease with the cracking of the structural members. Such instances

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are worse than the design basis accident conditions since the cause of cracking for the accident conditions is a self-relieving thermal load. Despite the difference, the only observable change for cracked bridge members is a "softening" of the structure as indicated by an increased deflection.

The nearest comparable systematic testing for this type of structure is for models of prestressed concrete reaction vessels (PCRV's). The Oldbury test series in England showed that a temperature gradient of about 30 F per inch showed no significant change in structural integrity (Ref. 3). Other tests such as described in Ref. 4 shows that temperature induced concrete cracking does not significantly affect structural integrity but does increase deformation. Tests at General Atomic (Ref. 5, 6) showed that the existence of gross cracking, due to pressure, with net tension across the concrete thickness, did not significantly alter structural response to subsequent retest at lower pressures, although it did leave visible evidence of the cracking. Further, the tests showed that for net tension conditions (similar to non-prestressed concrete) the structural integrity existed and was reasonably predictable. Although the PCRV models are thought of as thick walled structures, predictions using thin walled analytical formulae do not show gross differences from formulae which use the more complex thick wall formulaes, (Ref. 3) thus indicating that in reality the models tested were in the boundary region between thin and thick wall structures. Therefore, conclusions based on those model tests have strong bearing on this design where the significance of thermal load concrete cracking is concerned.

This part of the evaluation could not successfully challenge the tentative conclusion that inferential verification of structural integrity for design basis accident conditions could be inferred from observations during posttensioning. The effect of adding pressure to prestressing was considered to see if that addition changed the inferences needed to verify structural integrity at design basis accident conditions. For pressure test loads of 1.15 design pressure, the tendon loads are predicted to be only slightly greater than for prestressing alone, but less than those caused by the posttensioning process alone. The predicted prestress tendon load change from 0 psi to 1.15 design pressure did not exceed 6 percent of the tendon load before pressurization.

The predicted deformation was about the same magnitude and direction for test pressure as for design basis accident loads. The concrete is predicted to be more uniformly in compression than for design basis accident loads, but the maximum compression is lower. Therefcre, a greater degree of inference is required to relate predicted stresses at the inner face and a smaller degree of inference is needed to relate predicted stresses at the outer face of the concrete thickness. Overall, the test pressure load does not greatly decrease the amount of inference required as compared to the prestressing load observations.

The effect of increasing the test pressure was considered and the amount of inference needed increased with increases in test pressure since deformations became larger, tendon load changes became larger, the differences in average stresses became larger, and the differences in predicted stresses near the inner surface became larger. The only inference that became smaller was the relatively insignificant stresses at the layer of concrete at the outer surface. For increasingly higher test pressures, the concrete at the outer surface will crack more as a result of concrete shrinkage and temperature gradients normally present and dependent on the time of the year for the test. This is undesirable since the visible cracks could incorrectly undermine the confidence of observers who are not technically informed as to the significance. Such reduction of confidence could well result in the need to provide a cosmetic treatment to cover cracks which remained visible even when the test pressure was lowered and the concrete was again in compression. Also, cracking greater than normally found for concrete structures might require work to increase the corrosion protection for the reinforcing steel.

In essence, an unnecessarily high test pressure would prove essentially nothing technically for structural integrity verification that was not obtainable during posttensioning. Further, some technical disadvantages could result, depending on how high the test pressure was above design pressure. (An exception to this paragraph would be untested pressure closures for penetrations).

Lower test pressures were considered which in effect would be the equivalent to a design change of increasing both the amount of prestressing and concrete thickness for a given design pressure equal to the lower test pressure. It was obvious that this produced no significant change in a test pressure or posttensioning verification of structural integrity for design basis accident conditions. It did mean that a higher test to design pressure ratio would be required to cause the tension at the outer surface for the design basis accident conditions. Conclusions were thus unchanged as to the inferences needed for verification of structural integrity at design basis accident conditions from the inferences needed with the commonly accepted value of 1.15 design pressure.

Conclusions from the evaluation were:

1. At conclusion of posttensioning, it would be possible to have a better verification of structural integrity for design basis accident conditions than would be provided

by steel pressure vessel common practice after observations at the test pressure. The verification would be strongest if measurements were made of deformation of the containment during posttensioning since steel pressure vessel common practice has no counterpart to individual determination of tendon integrity nor to measurements of vessel deformation when loads are applied.

2. Pressure tests at any pressure level will not significantly increase the capability to verify the structural integrity for design basis accident conditions over that possible during posttensioning of the containment. With one minor exception, the amount and degree of inference from structural integrity at some test pressure level to the inferences needed from observations at higher pressure test increases with increasing pressure. An exception to this is the testing of any pressurized closure for the penetrations which has not been pressure tested in place by use of the techniques not requiring pressurization of the whole containment.

3. Common practice requires leak rate tests at design and other pressures. Some form of leak rate testing will always be necessary since the leak locations are not predictable as to location or frequency of occurrence. Common practice also requires verification of structural integrity by pressure test at pressures higher than the design pressure. Common practice, therefore, provides a strong psychological impetus for carrying out both leak rates and structural integrity tests with the whole containment pressurized. Until a leak rate test is developed which verifies leak tightness at welds and in the unwelded portions of steel plates, there is no means of demonstrating leak tightness of the containment except by pressurizing the whole containment.

   To satisfy common practice for both structural and leak tight integrity tests, and physically demonstrate both, it may be desirable to pressure test at a pressure high enough to be sure that design pressure was created when instrumental inaccuracies are considered. For structural integrity tests, 1.15 times design pressure is considered suitable by common practice and is not necessary for comparison with technical standards of overall structural integrity if measurements are made during posttensioning.

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References

1. PSAR - Rancho Seco Generating Station.

2. ASME Boiler and Pressure Vessel Code, Section 3.

3. "Testing a 1/8 Scale Cylindrical Vessel" by D. C. Price and M. S. Hinley Group G, Paper 43, 1967 (London Prestress Concrete Pressure Vessel Symposium)

4. "Comparison of Theoretical and Experimental Test Results for Ribbed Spherical Vessels" by M. L. A. Moncrief - Group G, Paper 92, Part 1, 1967 (London Conference on Prestressed Concrete Pressure Vessels).

5. "Prestressed Concrete Reactor Vessel Model 1", GA 7097, H.T.G.R. and Laboratory Staff.

6. "Prestressed Concrete Reactor Vessel Model 2", GA 7150, Advance H.T.G.R. Staff.

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5J.12.5 (DRL 5.7.5) Provide a table that compares the computed stresses for two different pressure test conditions with the computer stresses due to the incident alone, and to the earthquake plus incident. The information should be sufficient to evaluate the reliability of the stress computations. Explain in detail the methods used in the preparation of this table, the physical constants employed, etc. The following points should be carefully covered:

(a) Thermal stresses at large openings; evaluation of temperature gradients, stress computations for concrete and reinforcing steel, methods of combining stressed due to normal, tangential, bending, and torsional load, assumptions on cracking, stressed in stirrups, etc.;

(b) Prestressing;

(c) Influence of shrinkage;

(d) Creep;

(e) Influence of liner deformations (elastic and plastic);

(f) Stresses in the liner before cracking of concrete does occur; and

(g) Influence of transient thermal gradients.

ANSWER The data required to completely answer this question is not available at this time.

Section 5.1.5 describes the structural design analysis used for the containment, including large openings. The effects of item (a) through (g) have been discussed in previous submittals and in the answers to questions in this submittal.

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QUESTION 5J.13 (DRL 5.10)

From the preliminary plans presented in the PSAR it appears that there could be relative motions between the various structures of the Rancho Seco facility. What are the calculated magnitudes of the possible relative motions between buildings and what provisions are made in the design to accommodate these relative motions in both horizontal and vertical directions?

ANSWER

Relative motions between structures due to fault motions are not considered possible because of the low seismicity of the Rancho Seco site. This is based upon the specific recommendations of the Seismologist, P. Byerly, as outlined in Appendix 2D. Relative motion due to the dynamic and thermal response of structures will be considered.

The procedures to be used to obtain the relative motions between related structures are those described in PSAR Section 5.1.5.6 and summarized as follows:

1) A mathematical model is made incorporating the pertinent dynamic characteristics of the building.

2) The equations of motion are set up for each degree of freedom incorporating the stiffness characteristics of the model.

3) The characteristic or eigenvalue problem is solved for the natural frequencies and mode shapes. This part is normally solved by a computer analysis generally using the STRESS and SMIS programs. 4

4) The load distribution on the structure is obtained from the energy relationships using a modal participation factor.

5) The dynamic loads are obtained from the design earthquake spectrum using the appropriate damping coefficients listed in PSAR Appendix 5A.

6) The forces are obtained using the square root of the sum of the squares of each mode. A sufficient number of modes to accurately describe the motion are considered.

7) The relationships between acceleration, velocity, and displacement based upon the assumption of sinusoidal motion are used to obtain the corresponding maximum displacement at each concentrated mass.

8) The deflections due to vertical accelerations are computed in a similar manner. As in the above, the number of degrees of freedom are selected to accurately represent the anticipated response of the structure. In general, however, the number of degrees of freedom for vertical response is substantially less than for horizontal motion due to the inherent rigidity in the vertical direction.

9) The deflections obtained from these calculations are combined with thermal displacements to obtain the maximum deflection of that portion of the structure.

10) Consideration is given to the individual displacement of each structure to obtain the relative displacement of one structure with respect to another.

The preliminary calculated magnitudes of the maximum possible
4 horizontal relative deformations between the various structures
under maximum earthquake and temperature conditions are listed
below:

	Total Max. Relative Horizontal Deformation (in.)
Containment Structure Relative to Auxiliary Building	1-1/2
Containment Structure Relative to Fuel Storage Pool	1-1/2
Auxiliary Building Relative to Fuel Storage Pool	1
Auxiliary Building Relative to Turbine Building	1
Turbine Building Relative to Fuel Storage Pool	1-1/2

Relative displacements between structures in the vertical
direction are considered in two parts. First is the normal
anticipated settlement due to static foundation loads. The
foundation material is such that all measurable settlement
will occur during the construction phase when a load is
reapplied to the foundation material. (This is discussed in
detail in PSAR Appendix 2E Soil and Foundation Investigation
4 Report.) Therefore long-term settlement is not a governing
condition at this site. Second is the consideration of per-
manent settlement due to seismic loadings. Considering the
foundation material and the calculated pressures under foun-
dations during seismic loading, all seismic induced deforma-
tions are expected to be elastic and no permanent differential
settlement is anticipated during earthquakes.

The ultimate strength of the soil is approximately 220 kips per square foot and the maximum local soil pressure anticipated from any load combination, including seismic, is approximately 40 kips per square foot. The temporary displacement due to seismic loading will be less than one inch.

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The relative motions between structures listed above are minimum deformations for which structure separations must be designed. A factor of safety of at least two will be applied to the calculated displacements for design of individual structural separations. Separation will be accomplished using clear space, compressible material, and other suitable expansion joints. |4

QUESTION 5J.14 (DRL 5.11)

It is indicated on page 5.1-3 of the PSAR that the ratio of vertical to horizontal earthquake excitation will be one-half. Provide justification for the selection of this value for this particular site.

ANSWER

Studies indicate that the ratio of the vertical to the horizontal earthquake response spectra is less than one-half. Chapra, in "The Importance of Vertical Component of Earthquake Motions," Bulletin Seismological Society of America, Vol. 56, No. 5, October 1966, compared the ratio of the vertical to horizontal spectrum intensity of the El Centro 1940, Olympia, Washington 1949, and Taft 1952 earthquakes for the 20% critical damping and obtained ratio of 0.25, 0.20 and 0.31 respectfully. Housner in "Vibration of Structures Induced by Seismic Waves," Chapter 50, Shock and Vibration Handbook Edited by Harris and Crede, 1961 has stated that the Taft earthquake was produced by a predominantly vertical slipping on the fault rather ther the more usual horizontal slipping and the horizontal motion was not so intense at Taft as might have been expected for a shock of its size. Therefore, the comparison of the spectrum intensity for Taft is consistent with other earthquakes, although its vertical motion would be higher than other earthquakes, due to the nature of its dominant motion.

As such, the rati of one-half vertical to horizontal is considered a conservative value for the Rancho Seco site.

QUESTION 5J.15 (DRL 5.12)

No mention is found in the PSAR as to how the vertical and horizontal earthquake stresses will be combined with the dead load, live load, operating loads, and accident loads. It can be inferred from statements in several sections of the PSAR that the stresses from the vertical and horizontal earthquake excitation will be added linearly and directly to other applicable loadings, but confirmation of this fact is requested.

ANSWER

A discussion of interaction to be considered for stresses induced by vertical and horizontal acceleration is indicated in Section 5.1.2.4 and Sections 3.2 and 4.0 of Appendix 5A. To provide further clarification, the stresses from the vertical and horizontal earthquake excitation will be added linearly and directly to other applicable loadings.

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QUESTION 5J.16 (DRL 5.13)

It is noted that the containment structure will be embedded in the ground, al hough the depth of embedment is not precisely stated. .l the depth of embedment be such that it will be necess · to consider the interaction of ground and structure under sei .ic loadings? If so, what procedures for handling this interaction will be employed?

ANSWER

The cont ent structure foundation will be approximately 35 feet below the adjacent finished grade. Compacted backfill around the structure to this depth will create an effective embedment which will influence the interaction of ground and structure under seismic loading. Two effects due to this embedment will be considered in the analysis and design of the containment structure:

a. Pressure waves striking the structure will produce stresses in the shell. This effect will be considered as a local effect in the analysis of the structure. The effect of secondary translation relative to ground or lateral passive earth pressure has been found to be negligible.

b. The mass of the soil surrounding the structure influences the secondary vibrations due to soil flexibility. This effect will be considered by including in the mass of the structure a portion of the underlying soil considered to vibrate with the structure.

4 For additional information please refer to the answer to Question 5J.7.16 part (b).

~~06-16~~

0038

QUESTION 5J.17 (DRL 5.14)

It is indicated on page 5A-5 of the PSAR that all Class II structures, systems, and equipment will be designed in accordance with practices which will not be less restrictive than that required by standard applicable codes or by the requirements of the Uniform Building Code. Further amplification on the procedures to be employed for the design of Class II structures, systems, and equipment is required; and if a building code such as the Uniform Building Code is to be employed, the applicable seismic zone should be identified as well as other applicable factors relating to the design of these items.

ANSWER

All Class II structures, systems, and equipment will be designed in accordance with practices which are not less restrictive than those required by standard applicable codes as listed in the answer to question 5J.6.7. Seismic design criteria for Class II structures, systems, and equipment is described in detail in the answer to question 5J.6.5. Wind loading for Class II structures will be determined using the Uniform Building Code, using the specified resultant wind pressure zone of 20 pounds per square foot.

QUESTION 5J.18 (DRL 5.15)

With regard to the containment liner, it is noted that the maximum strain in the liner will be limited to one-half percent under the maximum or most severe loading conditions. It is also indicated that the buckling strength will be greater than the proportional limit. For purposes of clarification, provide the calculations leading to the buckling strength based on the proposed liner thickness and anchorage spacing to support this value. Describe the status with regard to buckling at strains as high as one-half percent. Provide the details of fastening the liner to the anchor angles as well as a description of the provisions that are taken to insure that, under loading conditions involving accident and seismic effects, rupture or tearing of the angle is not likely.

ANSWER:

Most of these questions are answered in 5J.7.4 where the consequences of having the buckling strength above or below yield have also been explained. Based on previously given information and the fact that the actual yield strength may be much higher than the specified minimum yield strength it has been decided that the buckling strength will not be above the actual yield strength.

0039

QUESTION 5J.19 (DRL 5.16)

The load combination equations to be employed in the design of the containment structure are listed on pages 5.1-14 of the PSAR. With regard to load combinations (b), (c), (d) and (e), provide information as to which of these expressions will be controlling for design of various components on the basis of the design made to date. In particular, under what conditions, or alternatively at what locations, will load condition (e) control the design?

ANSWER

The load combination equations to be employed in the design of the containment structure are listed on page 5.1-14 of the PSAR. Load combination equations (b), (c), (d) and (e), include earthquake as one of the loads.

In general, earthquake load will not have any controlling effect in the design of dome and ring girder. Either equation (b) or (d) will govern the design of cylinder for resisting shear from earthquake. The base slab and the base of the cylinder will be the two main components of the containment to be highly affected by earthquake. In general, equation (d) and (e) will control the design of base slab and base of the cylinder.

Equation (e) will control the design at the following locations:

Base slab:

1. Shear design at the edge.

2. Design of the reinforcing steel in the bottom portion at the edge.

3. Design of the reinforcing steel in the top portion at the section, located between center and edge of the base slab.

Base of the cylinder:

1. Design of the reinforcing steel at the outside face.

2. Radial shear design.

However, it should be noted that above mentioned factors are dependent on absolute and relative magnitudes of E and E', the type of soil supporting the base slab, and the relative stiffness of the base slab and the cylinder.

0040

QUESTION 5J.20 (DRL 5.17)

It is indicated in the first full paragraph on page 5.1-16 of the PSAR that the stresses from the maximum loading condition, considering the load factors presented, will not exceed yield strength. Further on it is noted that the strain in the liner will not exceed one-half percent. Since the wall of the structure and the liner must act together, clarification is required as to the conditions under which the strain in the liner could approach one-half percent and still maintain the remainder of the structure at less than yield.

ANSWER

As stated in the first paragraph, page 5.1-21 of the PSAR, the membrane strain of the liner plate subjected to design accident conditions will be approximately 0.0025 in/in. The limit of 0.005 in/in is based on what is acceptable for the liner, and includes an allowance for bending strain as well as membrane strain. Thus the membrane strain of the liner and the adjacent concrete surface could be nearly equal at approximately 0.0025 in/in, with the liner plate subjected to an additional bending strain of 0.0015 in/in due to relaxation curvature.

The statement in 5.1-16 refers to stresses for a maximum (i.e. factored) load condition and refers to the yield of the structure as a whole. The later statement refers to a portion of the structure only, i.e. the liner. It is true that the concrete of the structure and the liner must act essentially together but it is not true that there is no stress gradient across the wall thickness, hence the liner could be a yield while the structure as a whole is not at yield. An example of such a stress gradient can result from thermal loads where the liner heats quickly compared to the concrete. The liner thermal stress is caused by the fact that the concrete will not allow the full thermal expansion of the liner to take place since the average concrete temperature is lower than that of the liner and the concrete will not thermally expand as much as the liner. Should liner yield conditions be predicted for such a loading combination, the one-half percent yield strain criterion applies rather than an allowable stress limitation. This criterion could have been expressed as a fictitious stress allowable as has been done for example for the ASME Boiler and Pressure Vessel Code. For use in concrete containment, however, it is more convenient to use the limiting predicted liner strain.

0041 ~~00519~~

QUESTION 5J.21 (DRL 5.18)

It is noted in Section 5.1.4.6 that principal concrete tension resulting from combined membrane tension, membrane shear, and flexural tension due to bending moments where thermal gradients exist, will be limited to $6\sqrt{f'_c}$. Provide information which will illustrate the relationship of this criterion to the criterion that the yield strength will not be exceeded in the structure under the design load conditions.

ANSWER

The criterion in Section 5.1.4.6 states that "The load combinations, considering load factors given above, will be less than the yield strength of the structure." This statement implies that containment structure will have elastic response as a whole under different load combinations given under Section 5.1.4.6. This doesn't imply that modulus of rupture of concrete will not be exceeded. The value of $6\sqrt{f'_c}$ gives the principal tensile stress of concrete at which cracking initiates and reinforcing must be provided. The provision for reinforcing steel, in cases when the above mentioned tensile stress limit is predicted to be exceeded is given on pages 5.1-16 and 5.1-17 under (a) and (b) of the PSAR. Under the yield load conditions, the limitations on yielding are applicable to the reinforcing and prestressing steel for the structure in the context of the usual connotations of yield for steel in reinforced concrete, i.e., over a large region of tension and not at a point location. For design load conditions, concrete cracking at small regions and due to thermal loads is not predicted as affecting structural response to an extent more significant than for example, the first cracking load for a reinforced concrete structure.

QUESTION 5J.22 (DRL 5.19)

The reactor building crane must be designed to resist dislodgement during an earthquake and, moreover, designed in such a manner as to preclude damage to any critical items that would prevent safe plant shutdown. Provide information concerning the design criteria selected for these cranes.

ANSWER

The polar crane will be designed to meet the loads described by E.O.C.I. Specification No. 61 for electric overhead traveling cranes, except that seismic loading will be the seismic response of the containment structure at the crane supports level. In addition, the crane will be provided with mechanical guides on the rails to eliminate the possibility of derailment during the maximum earthquake condition. Furthermore, it will be detailed such that in no credible circumstance could the polar crane fall, even in the unlikely event of the failure of a rail. For additional discussion of crane support see answer to Question 5J.3 of Amendment 1.

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QUESTION 5J.23 (DRL 5.20)

The design of the piping, reactor internals, vessels, supports and other critical equipment for seismic loading receives little attention in the PSAR. On page 4.1-6 of the PSAR it is stated that the reactor coolant system components are designated as Class I equipment and are to be designed to maintain functional integrity during earthquake, and that the basic design guide will be AEC Publication TID-7024. Provide the loading and loading combinations applicable to the design of these elements as well as the allowable deformations for the various loading combinations. The presentation should be made in such a way that the margin of safety is clearly indicated for the various loading conditions and stresses or deformation criteria.

ANSWER

The loading combinations and allowable deformations applicable to the design of piping, reactor internals, vessels, supports and other critical equipment for seismic loading has been included in detail in Section 3.2 of Appendix 5A as further clarified in Amendment 2 particularly Appendix 5J Question 5J.4.

QUESTION 5J.24 (DRL 5.21)

There are many elements of the control room instrumentation, batteries, battery racks, etc., which are Class I items and which must survive seismic motions. Provide the design criteria for these items including an evaluation of the ability of the instruments to function under conditions of tilt as well as normal seismic loadings.

ANSWER

The elements of the control room instrumentation, batteries, battery racks, and miscellaneous auxiliary system components, which are Class I items, will be designed to survive the seismic accelerations which are specified in Appendix 5A, Section 4.0. The design criteria requires that these Class I items perform their specified functions under the "design earthquake" conditions and permit a safe shutdown of the plant under the "maximum hypothetical earthquake" conditions.

The design approach will be to consider the effective magnified acceleration at the location of the equipment, which is generated by the specified earthquake at the base of the auxiliary building. A dynamic analysis technique will be used in this design approach as in other critical structures. Since the auxiliary building is classified in its entirety as Class I, complete design consistency is maintained. Results of this analysis will be used to set specific criteria for the Class I equipment to assure that the equipment will function as specified.

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Based on manufacturers information, the accelerating force and
tilt for some of the Class I equipment is tabulated below.
4 The values given are the maximum that can be tolerated without
impairing the apparatus' capability to perform its principal
function, including its capability to perform satisfactorily
after the condition of shock has subsided.

Equipment	Accelerating Force g's	Tilt Degrees
Emergency Diesel Generator	4	10
4.16 kv-480 volt Transformers	2	15
4.16 kv Switchgear	2	5
480 volt Switchgear	2	5
480 volt Motor Control Centers	2	10
Batteries and Rack	5	20
Inverters and Battery Chargers	4	20
Motors	4	6
Instruments	*	*

The values given above are for equipment of standard commercial manufacture. If the above analysis shows that the accelerating force or the tilt for a particular component exceeds one of the respective values listed, that piece of equipment, its supports, or the system in which it functions will be modified as necessary to meet the specified design criteria.

4 *This information is not available but will be developed by mid 1971.

QUESTION 5J.25 (DRL 5.22)

The design of the on-site reservoir is described in Appendix 2G is noted to be a Class I item. Additional information is requested as to the manner in which the seismic analysis is to be made for the embankment.

4 ANSWER

Appendix 2G has been amended to provide the additional information requested.

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QUESTION 5J.26 (DRL 5.23)

From the list of Class I structures, systems, and equipment presented in Appendix 5A it is not clear whether the cooling towers or the water basins associated therewith are Class I structures and components. Provide a clarification of this point. Provide a description of the design criteria for those parts of the cooling system which are Class I components.

ANSWER

No structures or components of the circulating water cooling system will be Class I.

QUESTION 5J.27 (DRL 5.24)

In the sketches presented in the PSAR it appears that at least one of the personnel hatches protrudes significantly beyond the containment building shell. Describe the procedures which will be incorporated in the design to insure that this structural element can not be damaged during an earthquake or otherwise cause damage to the containment system.

ANSWER

The length of the personnel hatch is dictated by the mechanical requirements of an operational double lock. The personnel lock, as such, will require certain design precautions in addition to those used in the other containment openings. Specifically all penetrations will satisfy the requirements of Paragraph N-1211 of Section III, Nuclear Vessels, of the ASME Code. The selection of ASTM A-516 Grade 60 or 70 made to ASTM A-300 has been made to meet these requirements. In order to minimize transfer of forces and structural interaction through the personnel hatch, the portion of the lock resting upon the appropriate exterior supporting platform will be provided with a bearing support possessing adequate bearing characteristics plus a surface possessing a low coefficient of frictions or the ability to deform laterally. A material such as Dupont "Teflon" or equivalent is contemplated. In addition, all loads imported from earthquake or otherwise, although reduced using the above details, will be taken by the containment structure.

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QUESTION 5J.28 (DRL 5.25)

Describe, so far as possible at this time, the long-term surveillance program that is contemplated for this plant.

ANSWER

The containment structure will be given considerable attention during both the construction phase and on a long-term basis. The details of this program are of a preliminary nature and are discussed in appropriate sections of the PSAR, as outlined below. A summary of the more important aspects of long-term surveillance and the appropriate PSAR reference is as follows:

a. The containment system equipment will be tested and inspected as indicated in Section 5.5 and 6.2.4.

b. The leakage monitoring system will be in effect as described in Section 5.8.

c. Periodic testing of the liner plate and penetrations will be based upon requirements of the AEC and as indicated in Section 5.2.4.

d. Section 5.5 has been amended to provide an outline of the in-service tendon surveillance.

e. A series of reference markers will be established on the containment shell for accurate structural deformation from testing, as well as normal operating loads. Markers will be monitored using accurate surveying instruments.

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