3.8 DESIGN OF CATEGORY I STRUCTURES

3.8.1 CONCRETE CONTAINMENT

3.8.1.1 Description of the Containment

The primary containment is divided by a horizontal diaphragm slab into two major volumes: the drywell and the suppression chamber. The drywell encloses the reactor vessel, reactor recirculation system, and associated piping and valves. The suppression chamber stores a large volume of water.

The primary containment, shown on Figures 3.8-1 through 3.8-8, is in the form of a truncated cone over a cylindrical section, with the drywell being the upper conical section and the suppression chamber being the lower cylindrical section. These two sections comprise a structurally integrated, reinforced concrete pressure vessel, lined with welded steel plate and provided with a steel domed head for closure at the top of the drywell. Connection of the drywell head to the top of the drywell wall is shown on Figure 3.8-9. The diaphragm slab is a reinforced concrete slab structurally connected to the containment wall, as shown on Figure 3.8-10.

The primary containment is structurally separated from the surrounding reactor enclosure.

The concrete dimensions of the primary containment are as follows:

a. Inside diameter

   1. Suppression chamber - 88'-0"

   2. Base of drywell - 86'-4"

   3. Top of drywell - 36'-4½"

b. Height

   1. Suppression chamber - 52'-6"

   2. Drywell - 87'-9"

c. Thickness

   1. Base foundation slab - 8'-0"

   2. Containment wall - 6'-2"

3.8.1.1.1 Base Foundation Slab

The containment base foundation slab is a reinforced concrete mat, the top of which is lined with carbon steel plate.

3.8.1.1.1.1 Reinforcement

The base foundation slab is reinforced with #18, Grade 60 rebar at the top and bottom faces. The maximum rebar spacing is 18 inches. Shear reinforcement consists of #8 and #9 vertical and inclined ties. Cadweld splices are used for splicing all main reinforcing bars. Figure 3.8-11 shows the plan and section views of reinforcement.

3.8.1.1.1.2 Liner Plate and Anchorages

The steel liner plate is ¼ inch thick, and is anchored to the concrete slab by structural steel beams embedded in the concrete and welded to the plate. Figure 3.8-12 shows details of the liner plate and anchorages.

3.8.1.1.1.3 Reactor Pedestal and Suppression Chamber Column, Base Liner Anchorages

Figures 3.8-13 and 3.8-14 show the base foundation slab liner anchorages for the reactor pedestal and the suppression chamber columns, respectively. For the pedestal anchorage, Cadweld sleeves are welded to the top and bottom surfaces of the thickened base liner to permit anchoring of the pedestal vertical rebar into the base foundation slab. Metal studs are welded to the top and bottom surfaces of the thickened base liner in order to transfer radial and tangential shear forces from the pedestal to the base foundation slab. For the suppression chamber column anchorage, pipe caps are welded to the thickened base liner at the locations where the column anchor bolts penetrate the base liner, in order to ensure the leak-tight integrity of the base liner.

3.8.1.1.2 Containment Wall

The containment wall is constructed of reinforced concrete 6'-2" thick, and is lined with carbon steel plate on the inside surface.

3.8.1.1.2.1 Reinforcement

The containment wall is reinforced with #18, Grade 60 rebar at the inner and outer faces. The inner rebar curtain consists of two meridional layers and one hoop layer. The outer rebar curtain consists of one meridional layer, two hoop layers, and two helical layers. Radial shear reinforcement consists of #6 horizontal and inclined ties. Cadweld splices are used for splicing all main reinforcing bars. Figures 3.8-15 and 3.8-16 show sections and developed elevation views of the suppression chamber wall reinforcement and drywell wall reinforcement, respectively.

3.8.1.1.2.2 Liner Plate and Anchorages

The steel liner plate is ¼ inch thick, and is anchored to the concrete wall by vertical stiffeners, using structural tees spaced horizontally every 2 feet, or less. Horizontal plate stiffeners provide additional stiffening. Figures 3.8-17 and 3.8-18 show details of the liner plate and anchorages.

Loads from internal containment attachments, such as beam seats and pipe restraints, are transferred directly into the containment concrete wall. This is accomplished by thickening the liner plate, and attaching structural weldments that transfer any type of load to the concrete, without relying on the liner plate or its anchorages. Where internal containment attachment loads are large, the structural weldments penetrate the liner plate, rather than being welded to opposite sides of the liner plate. This eliminates the possibility of lamellar tearing. Section 3.8.1.1.2.5 contains a further description of internal containment attachments.

3.8.1.1.2.3 Penetrations

Services and communications between the inside and the outside of the containment are performed through penetrations. Basic penetration types include pipe penetrations, electrical penetrations, and access hatches (equipment hatches, personnel lock, suppression chamber access hatches, and CRD removal hatch). Each penetration consists of a pipe sleeve with an annular ring welded to it. The ring is embedded in the concrete wall, and provides an anchorage for the penetration to resist normal operating and accident loads. The pipe sleeve is also welded to the containment liner plate to provide a leak-tight penetration.

Meridional and hoop reinforcement is bent around typical penetrations. Additional local reinforcement in the hoop and diagonal directions is added at all large penetrations, as shown on Figures 3.8-19 and 3.8-20. Local thickening of the containment wall at penetrations is generally not required. Section 3.8.2.1 contains further discussion of penetrations.

a. Pipe Penetrations

Details of typical pipe penetrations are shown on Figure 3.8-21. There are two basic types of pipe penetrations. For piping systems containing high temperature fluids, a sleeved penetration is furnished, providing an air gap between the containment concrete wall and the hot pipe. This air gap is large enough to maintain the concrete temperature below 200°F in the penetration area. A flued head outside the containment connects the process pipe to the pipe sleeve. For piping systems containing low temperature fluid, a separate sleeve for the penetration is not furnished. For this type of penetration, the process pipe is welded directly to the two ends of the embedded pipe penetration.

b. Electrical Penetrations

Figure 3.8-22 shows a typical electrical penetration assembly used to extend electrical conductors through the containment. The penetrations are hermetically sealed, and provide for leak testing at design pressure.

c. Equipment Hatches and Personnel Lock

Two equipment hatches, with inside diameters of 12 feet, are furnished in the drywell wall. One of these equipment hatches includes a personnel lock. Figure 3.8-21 shows a detail of an equipment hatch. Figures 3.8-16 and 3.8-23 show details of reinforcement around the equipment hatches. Additional meridional, hoop, helical, and shear reinforcement is used to accommodate local stress concentrations at the opening. The containment wall is thickened at the equipment hatches to accommodate the additional rebars.

d. Suppression Chamber Access Hatches

Two access hatches, with internal diameters of 4'-4", are furnished in the suppression chamber wall, as shown on Figure 3.8-21. Figure 3.8-15 shows a detail of reinforcement around the suppression chamber access hatches. Additional local reinforcement in the meridional and diagonal directions is added as shown on this figure.

3.8.1.1.2.4 Drywell Head Assembly

The drywell head lower flange is anchored to the top of the drywell wall by rigid attachment to 108 meridional reinforcing bars in the inner curtain of the containment wall, as shown on Figure 3.8-9. The remainder of the drywell head assembly is discussed in Section 3.8.2.1.1.

3.8.1.1.2.5 Internal Containment Attachments

The principal items attached to the containment wall from the interior are the diaphragm slab, beam seats, pipe restraints, and the seismic truss.

a. Diaphragm Slab Embedments

   The diaphragm slab is attached to the containment wall by a structural weldment at the junction of the two components, as shown on Figure 3.8-10. Radial force and bending moment, carried by the diaphragm slab main reinforcement, are transferred to the containment wall by Cadwelding the diaphragm slab rebar to the top and bottom flanges of the structural weldment. The top and bottom flanges of the structural weldment are embedded in the containment concrete wall, and are anchored using structural steel anchors. Flexural shear in the diaphragm slab is transferred to the containment wall through the web of the structural weldment, which is welded to opposite sides of the thickened containment liner plate.

b. Beam Seat Embedments

   Beam seats are provided to support the drywell platforms. A typical beam seat embedment is shown on Figure 3.8-24.

c. Pipe Restraint Embedments

   Pipe restraints are provided to prevent pipe whip caused by rupture of high energy piping. Typical pipe restraint embedments are shown on Figure 3.8-25.

d. Seismic Truss Support Embedments

   The seismic truss provides lateral support for the reactor vessel and reactor shield. A typical seismic truss support embedment in the drywell wall is shown on Figure 3.8-26.

3.8.1.1.2.6 External Containment Attachments

There are no major external structural attachments to the primary containment wall, except brackets providing vertical support for some of the reactor enclosure floor beams. These floor beams support checkered plate blowout panels, and are small enough to not cause any vertical interaction between the containment structure and the reactor enclosure. In addition, the beam-to-bracket connections are sliding connections, preventing horizontal interaction between the containment structure and the reactor enclosure.

3.8.1.1.2.7 Steel Components Not Backed by Structural Concrete

Descriptions of steel portions of the primary containment that are not backed by concrete, such as the drywell head, equipment hatches, personnel lock, suppression chamber access hatches, CRD removal hatch, and piping and electrical penetrations, are given in Section 3.8.2.

3.8.1.2 Applicable Codes, Standards, and Specifications

The codes, standards, and specifications used in the design and construction of the primary containment are listed in Table 3.8-1.

Specifications were prepared to cover the areas related to the design and construction of the containment. These specifications were prepared by Bechtel specifically for this containment. These specifications emphasize important points of the industry standards for this containment, and reduce options such as would otherwise be permitted by the industry standards. Unless specifically noted otherwise, these specifications do not deviate from the applicable industry standards, and, as such, are not included in the UFSAR. These specifications cover the following areas:

a. Furnishing and delivering concrete

b. Forming, placing, finishing, and curing concrete

c. Furnishing, detailing, fabricating, delivering, and placing reinforcing steel

d. Splicing reinforcing bars

e. Furnishing, delivering, and erecting liner plate

Section 1.8 provides references to regulatory guides discussed in the UFSAR. Regulatory guides specific to this section are discussed in this section.

3.8.1.3 Loads and Loading Combinations

Table 3.8-2 lists the loading combinations used for the design and analysis of the containment. Loading combinations listed in ASME Section III, Division 2 were considered for the containment design. Table 3.8-21 identifies and explains differences between the loads listed in Table 3.8-2 and the ASME Code.

The containment is also analyzed and designed for hydrodynamic loads resulting from MSRV discharge and LOCA phenomena. For a definition of these loads and loading combinations, including hydrodynamic loads, refer to Reference 3.8-1 and Appendix 3A.

3.8.1.3.1 Dead Load

The dead load includes the weight of the structure and major equipment, plus any other permanent loads, such as soil or hydrostatic loads, or operating pressure.

3.8.1.3.2 Live Load

The live load includes those loads expected to be present when the plant is operating, such as movable equipment, piping, and cables.

3.8.1.3.3 Design Basis Accident Pressure Load

Transients resulting from the DBA LOCA are presented in Section 6.2.1, and serve as the basis for the containment internal design maximum pressure of 55 psig.

3.8.1.3.4 Thermal Loads

The thermal loads used in the design of the primary containment are shown in Table 3.8-3 for the operating and the postulated accident conditions.

Thermal effects anticipated at the time of the structural integrity test are not considered. The ambient air temperature during testing is close to that of construction; therefore, the effects of thermal gradient are negligible.

3.8.1.3.5 Wind and Tornado Loads

Wind and tornado loads are not considered because the containment is enclosed by the reactor enclosure.

3.8.1.3.6 Seismic Loads

Loads from the OBE result from a horizontal ground acceleration of 0.075 g and a vertical ground acceleration of 0.05 g, acting simultaneously.

Loads from the SSE result from a horizontal ground acceleration of 0.15 g and a vertical ground acceleration of 0.10 g, acting simultaneously.

3.8.1.3.7 External Pressure Load

The containment shell is designed to withstand an external pressure of 5 psi above the internal pressure.

3.8.1.3.8 Pipe Rupture Loads

The containment wall is designed to withstand the loads due to a postulated rupture of a 26 inch diameter main steam pipe, which produces the largest loads on the containment wall. These loads include the effects of jet impingement, pipe whip, and pipe reaction. An equivalent static load of 1000 kips is considered. This load includes an appropriate dynamic load factor to account for the dynamic nature of the load. Section 3.6 contains a detailed discussion of postulated pipe ruptures and their effects. For pipe whip and impact on restraints, the steel stress may exceed yield stress. For jet impingement, these stresses should not exceed yield stress. See Section 3.8.1.4 for further explanation.

3.8.1.3.9 Prestress Loads

The primary containment uses reinforced concrete; therefore, prestress loads have not been considered.

3.8.1.4 Design and Analysis Procedures

This section describes the procedures used for the design and analysis of the containment. For a description of the design and analysis procedures that consider the effects of hydrodynamic loads resulting from MSRV discharge and LOCA phenomena, refer to Reference 3.8-1 and Appendix 3A.

The analysis procedure consists of two parts. First, the uncracked forces, moments, and shears for both axisymmetric and nonaxisymmetric loads are determined. Axisymmetric loads are dead load, live load, design accident pressure load, vertical seismic load, and operating and design accident thermal loads. Nonaxisymmetric loads are horizontal seismic load and localized pipe rupture load. The second part of the procedure consists of taking into account the expected cracking of the concrete, and determining the concrete and reinforcing steel stresses and strains. The liner plate is not assumed to resist any load.

The 3D/SAP computer program, described in Section 3.8.7.1, is used to determine the uncracked forces, moments, and shears due to axisymmetric loads. The operating and design accident temperature gradients are computed using the ME 620 computer program, which is described in Section 3.8.7.9. For transient loads such as design accident pressure and thermal loads, the most critical combination of these loads is considered.

The seismic loads on the structure are determined by seismic analysis using the methods described in Section 3.7. The 3D/SAP program is used to analyze the containment for nonaxisymmetric loads due to missile and postulated pipe rupture, and for local construction loads.

The Concrete Element Cracking Analysis Program (CECAP), described in Section 3.8.7.3, is used to determine the extent of concrete cracking, and the concrete and rebar stresses and strains. The input data for the CECAP program consist of the uncracked forces, moments, and shears calculated by the 3D/SAP and seismic analysis programs. The CECAP program models a single element of unit height, unit width, and depth equal to the thickness of the wall or slab. The program assumes isotropic, linear-elastic material properties, and uses an iterative technique to obtain stresses, considering their redistribution due to cracking. The program determines the redistribution of thermal-stresses due to the relieving effect of concrete cracking.

3.8.1.4.1 Containment Wall

Figure 3.8-27 shows the 3D/SAP finite-element model used to analyze the containment wall for axisymmetric loads. A 10° wedge of the containment is modeled using solid finite-elements having isotropic, linear-elastic material properties. The model includes the containment wall, base foundation slab, diaphragm slab, reactor pedestal, and the foundation material. Boundary conditions are imposed on the analytical model by specifying nodal point forces or displacements. With reference to Figure 3.8-27, the nodal points lying along boundary A are allowed to move within the X-Z plane, and those along boundary B within the X-Y plane. Points along boundary C are prevented from moving in the radial direction, and points along boundary D are prevented from moving in the hoop direction. Nodal forces, moments, and shears are applied to boundaries E and F to account for reaction loads from the drywell head, and from the reactor vessel and reactor shield wall, respectively.

Figure 3.8-28 shows the 3D/SAP finite-element model used to analyze the drywell wall for nonaxisymmetric pipe rupture loads. A 180° half-model of the drywell wall, consisting of isotropic, linear-elastic, solid finite-elements, is used. With reference to Figure 3.8-28, the nodal points lying along boundary A are allowed to move within the X-Z plane. Points along boundary B are prevented from moving in the vertical and radial directions. Nodal forces, moments, and shears are applied to boundary C to account for reaction loads from the drywell head.

Tangential shears caused by seismic loads are totally resisted by helical reinforcing bars and concrete in compression. In calculating the reinforcing steel requirement, the helical reinforcement is designed to resist stresses due to design accident pressure and thermal loads, as well as tangential shears caused by seismic loads.

3.8.1.4.2 Base Foundation Slab

Figure 3.8-29 shows the 3D/SAP finite-element model used to analyze the base foundation slab. A 180° half-model of the base foundation slab, consisting of isotropic, linear-elastic solid finite-elements is used. The model includes the base foundation slab, a portion of the containment wall, and the foundation material. By virtue of continuous modeling of base slab and wall elements as finite elements in the 3D/SAP analysis, strain compatibility and stress equilibrium at the wall and slab boundary are satisfied. With reference to Figure 3.8-29, the nodal points lying along boundary A are allowed to move within the X-Z plane, and those along boundary B within the X-Y plane. Points along boundary C are prevented from moving in the radial direction. Axisymmetric forces, moments, and shears calculated using the 3D/SAP containment model, and seismically induced, tangential shears are applied to boundary D. The height of the model is chosen so that the overturning moment caused by the tangential shear is the same as the overturning moment determined by the seismic analysis. In order to be able to consider uplifting of the base foundation slab from its foundation, a thin layer of foundation material immediately beneath the foundation slab is considered separately from the remainder of the foundation material. If the computer output indicates tension in any of the elements in this thin layer of foundation material, the elements' modulus of elasticity is reduced to almost zero. Then a second computer run is made, and any additional uplift is identified. Further iterations and modifications of foundation material properties are made until the complete extent of uplift is determined.

3.8.1.4.3 Analysis of Areas Around Equipment Hatches

Figure 3.8-30 shows the 3D/SAP finite-element model used to analyze the areas of the containment wall around the equipment hatches. A 60° wedge of the containment wall is modeled using solid finite-elements having isotropic, linear-elastic material properties. To reduce the size of the analytical model, boundary A follows the equipment hatch's vertical plane of symmetry. The points delineating the outermost boundaries of the model are located at a sufficient distance from the opening so that the behavior of the model along the boundaries is compatible with that of the undisturbed shell. With reference to Figure 3.8-30, the nodal points lying along boundary A are allowed to move within the X-Z plane, and those lying along boundary B within the X-Y plane. Points along boundary C are prevented from moving in the hoop direction. Axisymmetric forces, moments, and shears calculated using the 3D/SAP containment model are applied to boundary D. Seismic loads calculated by the seismic analysis are applied locally to the elements. Seismically induced tangential shears around the equipment hatches are resisted by helical reinforcing bars and concrete in compression.

3.8.1.4.4 Liner Plate and Anchorages

The design and analysis of the liner plate and anchorages are performed in accordance with Reference 3.8-3.

3.8.1.5 Structural Acceptance Criteria

3.8.1.5.1 Reinforced Concrete

The containment wall, the diaphragm slab, and the reactor pedestal are designed for the factored load combinations listed in Table 3.8-2, in accordance with the strength method of ACI 318. The following allowable stresses are used:

a. Concrete

1. Compression - 0.85 $f_c^{'}$

2. Tension - not permitted

3. Radial shear - as specified in Chapter 11 of ACI 318 (1971)

4. Tangential shear - not permitted

b. Reinforcing Steel

1. Tension - 0.90 F

2. Compression - 0.90 F

The notations are defined as:

$f_c^{'}$ = specified compressive strength of concrete

F = specified yield strength of reinforcing steel

The calculated stresses are within the allowable limits.

3.8.1.5.2 Liner Plate and Anchorages

The structural acceptance criteria for the liner plate and anchorages are in accordance with Reference 3.8-3.

3.8.1.6 Materials, Quality Control, and Special Construction Techniques

3.8.1.6.1 Concrete Containment

The concrete and reinforcing steel materials for the containment are discussed in Section 3.8.6.

3.8.1.6.2 Liner Plate, Anchorages, and Attachments

a. Materials

Liner plate materials conform to the requirements of the standard specifications listed below:

Item	Specification
Liner plate (¼ inch thick)	ASTM A285, Grade A, Firebox Quality
Liner plate (>¼ inch thick)	ASME SA516, Grade 60 conforming to the requirements of ASME Section III, Subsection B, Article 12
Structural steel shapes, plates, and bars used for anchorages and attachments to the liner plate	ASTM A36

Pipe restraint attachments	ASTM A36 or ASTM A441

b. Welding

Liner plate and structural steel welding conforms to the applicable portions of Part UW, "Requirements for Unfired Pressure Vessels Fabricated by Welding" of ASME Section VIII. Welders and weld procedures are qualified in accordance with ASME Section IX. For seam welds of liner plate and attachments that penetrate the liner plate (penetrations), Paragraphs UW-26 through UW-38, except UW-35, apply in their entirety. In lieu of Paragraph UW-35, undercuts that do not exceed 1/32 inch and do not encroach on the required section thickness are in accordance with ASME Section III, Subsection NB(4424). The welding of liner plate butt welds and penetration attachment welds are performed by either the shielded metal arc, or the automatic, submerged arc process. The minimum number of individual weld layers for welds that must maintain leak-tightness is two. In addition, liner plate seam and penetration attachment welds are inspected in accordance with paragraph (d) below. Welding of liner anchorages and internal containment attachments to the liner plates are also performed in accordance with ASME Section VIII Paragraphs UW-26 through UW-38, and the visual inspection/acceptance of the nonpressure-retaining attachment weldments are in accordance with NCIG-01 (Reference 3.8-22).

c. Materials Testing

Liner plate material greater than ¼ inch thick, used with Cadweld connectors and other anchorages, is vacuum- degassed and ultrasonically tested, in accordance with the ASME Section III, Section A-N321-1, and conforms to the requirements of Article 12, Materials, of ASME Section III, Subsection B.

d. Nondestructive Examination of Liner Plate Seam Welds

Nondestructive examination of welds complies with Regulatory Guide 1.19, with the following alternate approaches:

Spot radiographic examination is performed for all radiographable liner plate seam welds. Radiography is performed in accordance with ASME Section V, Article 2. Personnel performing radiographic examinations are qualified in accordance with the Society for Nondestructive Testing's Recommended Practice No. SNT-TC-1A, Supplement A, Radiographic Testing Method, plus any additional requirements of the ASME Section V. Acceptance standards are in accordance with Paragraph UW-51 of ASME Section VIII, Division 1. Twelve inches of the first 10 feet of weld for each welder, and welding position are radiographed. Thereafter, one 12 inch long radiograph is taken for each welder and weld position, for each additional 50 foot increment of weld. A minimum of 2% of all liner seam welds are examined by radiography.

Where nonradiographable weld joints are used, the entire length of weld is magnetic particle examined. All magnetic particle examinations conform to the ASME Section V. Personnel performing magnetic particle examinations are qualified in accordance with SNT-TC-1A, plus any additional requirements of the ASME Section V. Acceptance standards are in accordance with the ASME Section VIII, Division 1, Appendix VI.

The vacuum box soap bubble test is performed on all accessible liner plate seam welds, except welds connecting thickened reinforcing liner plate to penetration nozzles with nominal diameter of 6 inches or less. A 5 psi minimum pressure differential is maintained for a minimum of 20 seconds. The leak detecting solution is continuously observed for bubbles that indicate leaks. If a leak is detected, the defective weld is repaired, and reinspected by vacuum box testing.

Welds that are inaccessible for vacuum box testing are 100% liquid penetrant tested. Liquid penetrant examinations conform to the ASME Section V. Personnel performing liquid penetrant examinations are qualified in accordance with SNT-TC-1A, plus any additional requirements of the ASME Section V. Acceptance standards conform to the ASME Section VIII, Division 1, Appendix VIII.

A leak chase channel system is provided only on liner plate seam welds which are inaccessible after the completion of construction, and those under water in the suppression chamber. Following installation, the leak chase system is pressurized with air to 72 psig. The pressure is monitored by valving off the air supply, and measuring any pressure decay with a pressure gauge. Any pressure decay in excess of the pressure gauge rated accuracy, observed within 15 minutes, is cause to reject that portion of the liner plate seam welds and the leak chase system. All leaks are repaired, and following repair, the affected portion of the leak chase system is retested.

e. Quality Control

Quality control requirements during construction are discussed in the document "Limerick Generating Station Units 1 and 2; Summary Description of the Quality Assurance Program for Design and Construction," referenced in FSAR Section 17.1.

f. Erection Tolerances

The erection tolerances for the liner plate are summarized as follows:

1. The slope of any 10 foot section of cylindrical liner plate, referred to as true vertical, does not exceed 1:180. The deviation from the theoretical slope of any 10 foot section of conical liner plate, measured within a vertical plane, does not exceed 1:120.

2. The cylindrical shell is plumb within 1/400 of the height, plus a 2 inch allowance for local "out-of- roundness." The vertical axis of the conical shell, as established at the top and bottom of the conical section, is plumb within 1/200 of the height.

3. The radial dimension to any point on the liner plate does not vary from the design radius by more than ±1 inch in the suppression chamber, or ±1½ inches in the drywell, except that the radial tolerance is ±2 inches for local "out-of-roundness." Local "out-of-roundness" tolerance is used where the maximum diameter minus the minimum diameter at the elevation does not exceed 4 inches. Radial measurements are taken at 24 locations, spaced equally around the containment at any elevation.

4. Plates joined by butt-welding are matched accurately, and retained in position during the welding operation. Misalignment in completed joints does not exceed the limitations of Paragraph UW-33 of ASME Section VIII, Division 1.

3.8.1.7 Testing and Inservice Surveillance Requirements

3.8.1.7.1 Preoperational Testing

3.8.1.7.1.1 Structural Acceptance Test

Structural and Pressure Integrity Tests were performed according to Article CC-6000 of the ASME Section III, Division 2, as applicable, after complete installation of all penetrations in the drywell and suppression chamber, and prior to initial fuel loading. The Structural and Pressure Integrity Tests complies with Regulatory Guide 1.18, with the following alternate approaches:

Paragraph C.1

A continuous increase in containment pressure, rather than incremental pressure increases, was used. This was considered justifiable because data observations at each pressure level were made rapidly by the computerized data acquisition system. "Rapidly" is defined as requiring a time interval for the data point sample sufficiently short so that the change in pressure during the observation would cause a change in structural response of less than 5% of the total anticipated change. Also, the rate of pressurization was limited to 3 psig/hr average to ensure that the structure responds to the pressure loading without any time lag. However, retesting for Unit 2 is performed at a rate of pressurization limited to 8 psig/hr average with adequate hold time at peak pressure to assure there is no time lag for the structural response. For a description of the structural integrity test for each unit, see the alternate approach to Paragraph C.12 in this section (3.8.1.7.1.1).

Measurements were recorded at atmospheric pressure, and at 5 psi increments of the pressurization and depressurization cycles. The data acquisition system allowed the measurements at all locations to be made and recorded simultaneously.

The pressure was held constant for at least 1 hour at the maximum test pressure, or for such time as was necessary for recording crack patterns.

Paragraph C.2

The number and distribution of measuring points for monitoring radial deflections were selected so that the as-built condition could be considered in the assessment of general shell response. In general, the locations of the measuring points for radial deflections were in agreement with figure B of the Regulatory Guide, with the exception of Point 1. Point 1 was provided at a distance of two times the wall thickness (12 feet) from the basemat. This variation was made to properly predict the containment behavior near the basemat to wall connection. If Point 1 was located at a height of three times the wall thickness (18 feet), it would be very close to Point 2 (suppression chamber wall midheight is 26 feet) and would not yield any additional behavior pattern of the containment.

Paragraph C.3

Radial deflections, but not tangential deflections, were recorded. The magnitude of the expected local tangential deformation under the test pressure conditions is negligibly small. The diametrical displacement of the hatch parallel to the tangential direction was measured and compared to the predicted displacement. This was in the location of largest predicted displacements for the hatch discontinuity and provided the most comprehensive check on structural integrity. The measurement of absolute tangential deflections at locations away from the immediate edge of the discontinuity was difficult because fixed reference points are difficult to define, making it impractical to attempt measurement of these small local deflections.

Paragraph C.9

The LGS containment is located inside and isolated from the reactor enclosure. The environmental conditions under which the test was conducted were controlled by the reactor enclosure internal environment. Therefore, limitations on testing during periods of extreme weather conditions, such as snow, heavy rain, and strong wind, had negligible effect on structural response and were not applicable.

Paragraph C.10

Rather than as suggested in the guide, if the test pressure dropped to, or below, the next lower pressure level due to an unexpected condition, the test would have continued without a restart at atmospheric pressure, unless the structural response deviated significantly from that expected. The LGS containment pressurization was not conducted using pressure increments. Therefore, the provisions of ASME Section III, Division 2, subsection CC-6252(b) were met, using a 3 psi pressure drop as a criterion for investigating deviation in structural response.

Paragraph C.12

LGS has provided a detailed description of the structural integrity test in the report prepared after the test.

The Structural and Pressure Integrity Test was conducted at 115% of the following design conditions:

a. A design pressure condition of 55 psig in both the drywell and suppression chamber

b. A design pressure condition of 55 psig in the drywell, and 25 psig in the suppression chamber

Additional pressure integrity tests may be made subsequently, during periods of plant shutdown.

The differential pressure test of the diaphragm slab described in item (b) above was accomplished by capping the downcomers above the diaphragm slab's upper surface.

The LGS Unit 1 test was performed in one test sequence which demonstrated the structural adequacy of the containment structure for both of the design conditions described above. However, for LGS Unit 2, the structural integrity test was conducted in two stages because the maximum differential pressure on the diaphragm slab could not be maintained during the initial pressurization of the containment structure. During the initial test, the structural adequacy of the containment structure for the design condition of 55 psig in both the drywell and the suppression chamber was satisfactorily demonstrated. Subsequent to the re-isolation of the drywell and the suppression chamber, the containment structure was retested at a rate of pressurization limited to 8 psig/hr average to 115% of the design condition "b" described above (drywell pressure equal to 63.25 psig and suppression chamber pressure equal to 28.75 psig).

Provisions are made to test the integrity of the primary containment system during the life of the plant.

The seals on the personnel airlock doors, hatches, and drywell head are capable of being tested for leakage at design pressure without pressurizing the drywell. In addition, provisions are made so that the space between the airlock doors can be pressurized to full drywell design pressure.

Electrical penetrations are testable at 115% of design pressure. Locations of test taps and seals allow electrical penetrations to be tested without entering or pressurizing the drywell or suppression chamber.

A more detailed description of the structural integrity test was provided in the report prepared after the initial test.

3.8.1.7.1.2 Leak Rate Testing

Preoperational leak rate testing is discussed in Section 6.2.6.

3.8.1.7.2 Inservice Leak Rate Testing

Inservice leak rate testing is discussed in Section 6.2.6.

3.8.2 ASME CLASS MC STEEL COMPONENTS OF THE CONTAINMENT

This section pertains to the ASME Class MC steel components of the concrete containment that form a portion of the containment pressure boundary, and are not backed by structural concrete. These components include the drywell head assembly, the equipment hatches, the personnel lock, the suppression chamber access hatches, the CRD removal hatch, and piping and electrical penetrations.

3.8.2.1 Description of the ASME Class MC Components

3.8.2.1.1 Drywell Head Assembly

The drywell head provides a removable closure at the top of the containment for reactor access during refueling operations. The drywell head assembly consists of a 2:1 hemi-ellipsoidal head and a cylindrical lower flange. The lower flange is supported on the top of the drywell wall as shown on Figure 3.8-9. The head is made of 1½ inch thick plate and is secured with eighty 2¾ inch diameter bolts at the 4 inch thick mating flange. The head-to-lower flange connection is made leak-tight by two replaceable gaskets. The space between the gaskets is provided with test connections to allow pneumatic testing from a remote location, outside the primary containment. The inside diameter of the drywell head at the mating flange is 37 feet 7½ inches. A double-gasketed manhole is provided in the drywell head. Figure 3.8-31 shows details of the drywell head assembly.

3.8.2.1.2 Equipment Hatches and Personnel Lock

Two 12 foot diameter equipment hatches are furnished in the drywell wall to permit the transfer of equipment and components into and out of the drywell. One hatch consists of a double-gasketed flange and a bolted dished door. The other hatch is furnished with a personnel lock welded to the removable door. The personnel lock is an 8'-7" diameter cylindrical pressure vessel, with inner and outer flat bulkheads. Interlocked doors, 2'-6" wide by 6' high, with double tongue-and-groove single element compression seals, are furnished in each bulkhead. A quick-acting, equalizing valve vents the personnel lock to the drywell to equalize the pressure in the two systems when the doors are opened and then closed. The two doors in the personnel lock are mechanically interlocked to prevent them from being opened simultaneously, and to ensure that one door is closed before the opposite door can be opened. The personnel lock has an ASME Code N-stamp. Figures 3.8-32 and 3.8-33 show details of the equipment hatch, and the equipment hatch with personnel lock, respectively.

3.8.2.1.3 Suppression Chamber Access Hatches

Two 4'-4" diameter access hatches are furnished in the suppression chamber wall to permit personnel access, and the transfer of equipment and components into and out of the suppression chamber. Each hatch consists of a double-gasketed flange and a bolted flat cover. Details of a suppression chamber access hatch are shown in Figure 3.8-21.

3.8.2.1.4 Control Rod Drive Removal Hatch

One 3 foot diameter CRD removal hatch is furnished in the drywell wall to permit transfer of the CRD assemblies into and out of the drywell. The hatch is furnished with a double-gasketed flange and a bolted flat cover. Figure 3.8-34 shows details of the CRD removal hatch.

3.8.2.1.5 Piping and Electrical Penetrations

A portion of each of the penetration sleeves extends beyond the containment wall, and is not backed by concrete. The entire length of any penetration sleeve, therefore, is considered an MC component, and, as such, is designed in accordance with ASME Section III, subsection B. Figures 3.8-21 and 3.8-22 show details of typical pipe and electrical penetrations, respectively.

3.8.2.2 Applicable Codes, Standards, and Specifications

The codes, standards, and specifications used in the design and construction of the containment are listed in Table 3.8-1.

Specifications are prepared to cover the areas related to the design and construction of the containment. These specifications are prepared by Bechtel specifically for this containment. These specifications emphasize important points of the industry standards for this containment, and reduce options that would otherwise be permitted by the industry standards. Unless specifically noted otherwise, these specifications do not deviate from the applicable industry standards, and as such are not included in the UFSAR. These specifications cover the following areas:

a. Furnishing and delivering concrete

b. Forming, placing, finishing, and curing concrete

c. Furnishing, detailing, fabricating, delivering, and placing reinforcing steel

d. Splicing reinforcing bars

e. Furnishing, delivering, and erecting liner plate

Section 1.8 provides references to regulatory guides discussed in the UFSAR. Regulatory guides specific to this section are discussed in this section.

3.8.2.3 Loads and Loading Combinations

Table 3.8-4 lists the loading combinations used for the design and analysis of the ASME Class MC components.

The ASME Class MC components are also assessed for Mark II hydrodynamic loads resulting from MSRV discharge and LOCA phenomena. For a definition of loads and loading combinations including hydrodynamic loads, refer to Reference 3.8-1 and Appendix 3A.

3.8.2.3.1 Dead and Live Load

For descriptions of dead and live load, see Sections 3.8.1.3.1 and 3.8.1.3.2, respectively.

3.8.2.3.2 Design Basis Accident Pressure Load

The drywell head is designed for a DBA internal pressure of 56 psi. The other MC components are designed for a DBA internal pressure of 62 psi.

3.8.2.3.3 External Pressure Load

The MC components are designed to withstand an external pressure of 5 psi above the containment internal pressure.

3.8.2.3.4 Thermal Loads

The operating and postulated design accident temperatures for the MC components are as follows:

	Temperature (°F)	
Condition	Drywell	Suppression Chamber
Operating	135 ①	95 (maximum)
Design Accident	340	220

Thermal cycles used in design are as follows:

a. Startup and shutdown - 500 cycles, 105°F range

b. Design Basis Accident - 1 cycle, 220°F range

3.8.2.3.5 Seismic Loads

The MC components are designed for acceleration values which were calculated using the methods described in Section 3.7.

The following seismic acceleration values are used for the design of the drywell head assembly:

a. OBE - 0.60 g horizontal, ±0.30 g vertical

b. SSE - 0.80 g horizontal, ±0.40 g vertical

The following seismic acceleration values are used for the design of the equipment hatch, personnel lock, and CRD hatch:

a. OBE - 0.40 g horizontal, ±0.20 g vertical

b. SSE - 0.45 g horizontal, ±0.25 g vertical

The following seismic acceleration values are used in the design of the suppression chamber hatches:

a. OBE - 0.20 g horizontal, ±0.15 g vertical

b. SSE - 0.30 g horizontal, ±0.20 g vertical

① This information is based on original design basis conditions. Further evaluation has validated the MC components for a drywell operating temperature of 150°F. The Stresses indicated in UFSAR Figures 3A-362 to 3A-380, Design Assessment Report for Containment Structures reflect current plant conditions.

3.8.2.3.6 Pipe Rupture Loads

The drywell head assembly is designed for a local pipe rupture load of 48,000 pounds, uniformly distributed over a circular area of 0.56 square foot, at any location on the drywell head. This load is due to the postulated rupture of the 6 inch diameter reactor head spray pipe, which produces the largest load on the drywell head. The head spray line does not exist for Unit 2 and has been removed from Unit 1. However, the analysis is still applicable because it envelopes loads from rupture of any other pipe in this region.

The equipment hatches are designed for a postulated pipe rupture load of 130,000 pounds, uniformly distributed over a circular area 12 feet in diameter.

The CRD removal hatch is designed for a postulated pipe rupture load of 160,000 pounds, uniformly distributed over a circular area 3 feet in diameter.

The loads on the equipment hatches and the CRD removal hatch are based on the rupture of a 28 inch diameter recirculation loop outlet pipe, which produces the largest load on the components.

The above values of static load include an appropriate dynamic load factor to account for the dynamic nature of the load. Section 3.6 contains a detailed discussion of postulated pipe ruptures and their effects, including pipe whip, jet impingement, environmental effects, flooding, and water spray.

3.8.2.3.7 Missile Impact Loads

As discussed in Section 3.5.1 and summarized in Table 3.5-7, missile impact is not considered credible for safety-related components inside the containment or the reactor enclosure.

3.8.2.4 Design and Analysis Procedures

This section describes the procedures used for the design and analysis of ASME Class MC steel components.

ASME Class MC components are also assessed for Mark II hydrodynamic loads resulting from MSRV discharge and LOCA phenomena. Assessment procedures that consider the effect of hydrodynamic loads are described in Appendix 3A.

3.8.2.4.1 Drywell Head Assembly

The analysis of the drywell head assembly uses the thin-shell computer program E0781, described in Section 3.8.7.7. This program calculates the stresses and displacements in thin-walled, elastic shells of revolution, when subjected to static edge, surface, and/or temperature loads with an arbitrary distribution over the surface of the shell.

The drywell head assembly is divided into two analytical models. Figure 3.8-35 shows the drywell head model and the lower flange model. Displacement compatibility of the two models at the mating flange surface is maintained in the analysis. Boundary conditions are imposed on the analytical models by specifying boundary forces or displacements. With reference to Figure 3.8-35, the translation and rotation of the top of the drywell wall are imposed as boundary conditions to boundary A. Boundary forces applied to boundary B are calculated in accordance with thin-shell theory.

See Section 3A.7.2.1.9.1 for discussion of analysis to reduce the drywell head bolt preload.

3.8.2.4.2 Access Hatches

Access hatches, including the equipment hatches, personnel lock, suppression chamber access hatches, and the CRD removal hatch, are designed as pressure-retaining components. The portions of the sleeves not backed by concrete are designed and analyzed according to the provisions of ASME Section III, Subsection B.

At the junction of the hatch cover to the flange on the sleeve, where local bending and secondary stresses occur, the computer program E0119, described in Section 3.8.7.7, is used for analysis. This program is also used for the analysis of the flat head covers.

3.8.2.4.3 Piping and Electrical Penetrations

For nuclear Class I flued head penetrations, the procedures used in design and analysis comply with ASME Section III, Subsection B.

For Class 1E electrical cable penetrations, the procedures used in design and analysis comply with ASME Section III, Subsection B.

3.8.2.5 Structural Acceptance Criteria

The structural acceptance criteria comply with Regulatory Guide 1.57, with the clarification that the Code Addenda in effect at the time of design rather than that cited in the guide were used. Table 3.8-4 lists the allowable stress criteria for the design and analysis of the ASME Class MC components.

3.8.2.6 Materials, Quality Control, and Special Construction Techniques

3.8.2.6.1 Materials

All carbon steel materials and stainless steel materials for the CRD hydraulic line penetrations conform to the requirements of ASME Section III, Subsection B. The drywell head assembly and other steel components are painted to protect against corrosion.

3.8.2.6.1.1 Drywell Head Assembly

Materials used in construction of the drywell head assembly conform to the following ASME specifications:

Item	Specification
Drywell head and lower flange	SA516, Grade 70, normalized
Bolts	SA320, Grade L43
Nuts	SA194, Grade 7

3.8.2.6.1.2 Access Hatches

Materials used in construction of the access hatches conform to the following ASME specifications:

Item	Specification
Sleeve and cover	SA516, Grade 60 or 70, normalized
Bolts	SA193, Grade B7
Nuts	SA194, Grade 7

3.8.2.6.1.3 Penetrations

Materials used in construction of piping and electrical penetrations conform to the following ASME specifications:

Item	Specification
Carbon steel sleeves	SA333, Grade 1 or SA516, Grade 60 or 70, normalized
Carbon steel caps for spare penetrations	SA234, Grade WPB
Stainless steel sleeves for CRD hydraulic line penetrations	SA312, Type 304L

Stainless steel fittings for CRD hydraulic line penetrations	SA182, Type 304

3.8.2.6.2 Welding

Welding conforms to the requirements of ASME Section III, Subsection NE-4000 except that all welding of the CRD hydraulic line penetrations conforms to the requirements of ASME Section III, Subsection NC. All pressure boundary welds are full penetration welds of double-welded, bevel-type. Welders and weld procedures are qualified in accordance with ASME Section IX.

Penetrations, access hatches, and the drywell head flange are postweld heat treated in accordance with ASME Section III, Article NE-4000. Penetrations are preassembled into the liner plate sections, and postweld heat treated as complete subassemblies.

3.8.2.6.3 Nondestructive Examination of Welds

All welds between penetrations and liner plate, access hatches and liner plate, and pressure-retaining welds not backed by concrete are examined in accordance with ASME Section III, Article NE-5000. Nondestructive examination of welds complies with Regulatory Guide 1.19, with the following alternate approaches:

Paragraph C.1.a

LGS uses an alternate approach with respect to radiographic examination of the first 10 feet of weld. For each welder and welding position, only 12 inches of the first 10 feet of weld is examined radiographically, instead of the entire 10 foot length. LGS conforms with all other guidelines for nondestructive examination of liner seam welds.

Paragraph C.4

LGS conforms with the intent, subject to the following interpretation: Nondestructive examinations are performed by employees of the liner plate Subcontractor. Bechtel, as licensee's agent, verifies that all personnel performing nondestructive testing are qualified in accordance with ASME Section V.

Paragraph C.7.a

LGS radiography acceptance standards are in accordance with UW-51 of the ASME Section VIII, Division I.

Paragraph C.7.b

Magnetic particle acceptance standards are in accordance with the ASME Section VIII, Division I, Appendix VI.

3.8.2.6.4 Quality Control

Quality control requirements during construction are discussed in the document "Limerick Generating Station Units 1 and 2; Summary Description of the Quality Assurance Program for Design and Construction," referenced in FSAR Section 17.1.

3.8.2.6.5 Erection Tolerances

The specified erection tolerances for ASME Class MC steel components of the containment are summarized as follows:

a. Suppression chamber penetrations are within 1 inch of their design elevations and circumferential locations.

b. Drywell penetrations are within 1 inch of their design circumferential locations. Critical penetrations, such as main steam, feedwater, core spray, etc, are within 1 inch of their design elevations. All other drywell penetrations vary from within 1 inch of design elevations for penetrations near the base of the drywell wall, to within 2 inches of design elevations for penetrations near the top of the drywell wall.

c. Alignments of penetrations are within 1° of the design alignments.

d. The average elevation of the drywell head lower flange is within 3 inches of the design elevation. The lower flange is within ½ inch of level.

3.8.2.7 Testing and Inservice Inspection Requirements

3.8.2.7.1 Preoperational Testing

3.8.2.7.1.1 Structural Acceptance Test

The drywell head assembly, equipment hatches, suppression chamber access hatches, CRD removal hatch, and piping and electrical penetrations are pneumatically tested to 1.15 times the design accident pressure during the containment structural acceptance test.

The personnel lock is pneumatically tested to 1.25 times the design pressure of 62 psig, following shop fabrication, or following field erection if final assembly of lock is performed in the field, to verify its structural integrity.

The CRD hydraulic line penetrations are hydrotested to 1.5 times the design pressure of 1250 psig following shop fabrication, in accordance with the ASME Section III, Subsection NC.

3.8.2.7.1.2 Leak Rate Testing

Leak-tightness of the containment pressure-retaining Class MC components is verified during the integrated leak rate test. Section 6.2.6 contains a description of the containment integrated leak rate test.

The personnel air lock is leak rate tested to 100% of the design accident pressure following shop fabrication, and following field erection. The maximum allowable leak rate for the personnel air lock shall be 5% of the allowable leak rate for the containment, L_a. L_a is 0.5% by weight of the air in the containment per 24 hours at test pressure, P_a (44.0 psig).

3.8.2.7.2 Inservice Leak Rate Testing

Inservice leak rate testing is discussed in Section 6.2.6.

3.8.3 CONTAINMENT INTERNAL STRUCTURES

3.8.3.1 Description of the Internal Structures

The functions of the containment internal structures include: support and shielding of the reactor vessel, support of piping and equipment, and formation of the pressure-suppression boundary. The containment internal structures are constructed of reinforced concrete and structural steel. The containment internal structures include the following:

a. Diaphragm slab

b. Reactor pedestal

c. Reactor shield wall

d. Suppression chamber columns

e. Drywell platforms

f. Seismic truss

Figures 3.8-1 through 3.8-8 show an overview of the containment including the internal structures.

3.8.3.1.1 Diaphragm Slab

The diaphragm slab serves as a barrier between the drywell and the suppression chamber. It is a reinforced concrete circular slab, with an outside diameter of 88 feet, and a thickness of 3'-6". Figure 3.8-36 shows details of the diaphragm slab reinforcement.

The diaphragm slab is supported by the reactor pedestal, the containment wall, and 12 steel columns. The connection of the diaphragm slab to the containment wall is shown on Figure 3.8-10. The diaphragm slab is penetrated by 87, 24 inch diameter downcomers. Additional reinforcement is furnished at downcomer penetrations. Section 6.2.1 contains a description of the downcomers.

A ¼ inch thick, carbon steel liner plate is provided on top of the diaphragm slab, and is anchored to it. The liner plate prevents bypass flow around the downcomers during a LOCA. Refer to Section 6.2.1, for a description of the bypass leakage requirements. Figure 3.8-37 shows the diaphragm slab liner plate and anchorage system.

3.8.3.1.2 Reactor Pedestal

The reactor pedestal is an 82 foot high, upright cylindrical reinforced concrete shell that rests on the containment base foundation slab, and supports the diaphragm slab, reactor vessel, and reactor shield wall, as well as drywell platforms, pipe restraints, and recirculation pumps. The connection of the reactor pedestal to the base foundation slab is shown on Figure 3.8-13. The reactor pedestal below the diaphragm slab has an inside diameter of 20'-1", and a wall thickness of 4'-9". The reactor pedestal above the diaphragm slab has an inside diameter of 20'-3", and a wall thickness of 4'-5". The thickness at the top of the pedestal increases to 5'-4" where it supports the reactor vessel and the reactor shield wall. Attachment of the reactor shield wall to the pedestal is described in Section 3.8.3.1.3. Attachment of the reactor vessel to the pedestal is accomplished through a ring girder, as described in Section 5.3.3.1.4.1. The ring girder is attached to the pedestal by 120, 3¼ inch diameter high strength anchor bolts, as shown on Figure 3.8-38.

Figures 3.8-39 and 3.8-40 show reinforcement details. Openings are provided in the reactor pedestal, to permit the flow of air and suppression pool water into and out of the pedestal cavity. Additional reinforcement is furnished at the openings. A ¼ inch thick carbon steel form plate is provided on the inside and outside surfaces of the reactor pedestal below the diaphragm slab. This plate acts as a concrete form during construction, and preserves the water quality of the suppression pool by preventing the leaching of chemicals from the reactor pedestal concrete into the suppression pool.

3.8.3.1.3 Reactor Shield Wall

The reactor shield wall is a 49 foot high upright cylindrical shell which rests on the top of the reactor pedestal, and provides primary radiation shielding, as well as support for pipe restraints and drywell platforms. The reactor shield wall is constructed of inner and outer carbon steel plates, with unreinforced concrete between the two plates. Figure 3.8-41 shows details of the reactor shield wall. The reactor shield wall has an inside diameter of 25'-7", and a wall thickness of 1'-9". The outer steel plate is 1½ inches thick, and is designed to withstand local loads transferred through pipe restraints and drywell platform attachments. The inner steel plate is ½ inch thick, and is designed to act with the outer plate to withstand local and nonlocal loads. The inner and outer plates are connected with shear ties spaced on approximately 5° centers in the hoop direction. The annular space between the inner and outer plates is filled with unreinforced high density concrete for radiation shielding. The reactor shield wall is connected to the top of the reactor pedestal by 48, 2 inch diameter high strength anchor bolts, as shown on Figure 3.8-38.

The seismic truss and reactor vessel stabilizer, which provide lateral support to the reactor vessel and reactor shield, are attached to the top of the reactor shield wall. Penetrations with hinged doors or removable plugs are provided in the reactor shield wall to accommodate piping connections to the reactor vessel, and to provide access for inservice inspection. The wall thicknesses of penetration sleeves are large enough to prevent local stress concentrations in the inner and outer plates.

3.8.3.1.4 Suppression Chamber Columns

Twelve hollow steel pipe columns are furnished to support the diaphragm slab. Each column is 52'-3" long, with an outside diameter of 42 inches, and a wall thickness of 1¼ inches. The columns are connected to the base foundation slab at the bottom, and to the diaphragm slab at the top with embedded anchor bolts. Figure 3.8-14 shows the connection to the base foundation slab; and Figure 3.8-42 shows the connection to the diaphragm slab.

3.8.3.1.5 Drywell Platforms

Platforms are furnished at six elevations in the drywell to provide access and support to electrical and mechanical components. The platforms consist of structural steel framing, with steel grating. Built-up box shapes are used for beams that must resist biaxial bending. Beams that span between the pedestal or reactor shield wall, and the containment wall are provided with sliding connections at one end. Figures 3.8-3 through 3.8-8 show details of the drywell platforms.

3.8.3.1.6 Seismic Truss and Reactor Vessel Stabilizer

The seismic truss and the reactor vessel stabilizer provide lateral support for the reactor vessel during earthquake and pipe rupture loading. The seismic truss horizontally spans the gap between the containment wall and the reactor shield wall; the reactor vessel stabilizer spans the gap between the reactor shield wall and the reactor vessel. The seismic truss is shaped like an eight-pointed star, and is fabricated of steel plates. Figure 3.8-43 shows details of the seismic truss. Figure 3.8-26 shows the connection of the seismic truss to the containment wall. This connection is designed to allow vertical and radial movement of the seismic truss relative to the containment wall, but to prevent tangential movement.

3.8.3.2 Applicable Codes, Standards, and Specifications

The codes, standards, and specifications used in the design and construction of the containment internal structures are listed in Table 3.8-1.

Specifications were prepared to cover the areas related to design and construction of the containment. These specifications were prepared by Bechtel specifically for this containment. These specifications emphasize important points of the industry standards for this containment, and reduce options that would otherwise be permitted by the industry standards. Unless specifically noted otherwise, these specifications do not deviate from the applicable industry standards, and as such are not included in the UFSAR. These specifications cover the following areas:

a. Furnishing and delivering concrete

b. Forming, placing, finishing, and curing concrete

c. Furnishing, detailing, fabricating, delivering, and placing reinforcing steel

d. Splicing reinforcing bars

e. Furnishing, detailing, fabrication, and delivering the suppression chamber columns, pedestal liner, structural steel for the diaphragm slab, and miscellaneous structural steel

f. Furnishing, delivering, and erecting liner plate

Section 1.8 provides references to regulatory guides discussed in the UFSAR. Regulatory guides specific to this section are discussed in this section.

3.8.3.3 Loads and Loading Combinations

Tables 3.8-2 and 3.8-5 through 3.8-8 lists the loading combinations used for the design and analysis of the containment internal structures. Tables 3.8-5 through 3.8-8 list only the most severe factored loading combinations used for miscellaneous internal steel components of the containment. These structures have been designed according to the working stress methods, except for the pipe restraints supported on the drywell platforms. Design of the pipe restraints and their box beam supports allows inelastic deformations due to postulated pipe rupture loads. However, there is no loss of restraint function due to pipe break load.

Apparent differences between the loads listed in Tables 3.8-5 through 3.8-8 and SRP 3.8.3, for the governing loading conditions, are identified and explained in Table 3.8-20.

The internal structures are also analyzed for hydrodynamic loads resulting from MSRV discharge and LOCA phenomena. For a definition of loads and loading combinations (including hydrodynamic loads), see Reference 3.8-1 and Appendix 3A.

3.8.3.3.1 Diaphragm Slab and Reactor Pedestal

Table 3.8-2 lists the loading combinations used for the design of the diaphragm slab and reactor pedestal. Descriptions of the loads are as follows:

a. Dead Load, Live Load, and Seismic Loads

   For a description of dead load, live load, and seismic loads, see Section 3.8.1.3.

b. DBA Pressure Load

   The diaphragm slab and the reactor pedestal are designed for the following pressures:

   1. Maximum pressure: 55 psig in the drywell and the suppression chamber

   2. Maximum differential pressure: 30 psig downward (55 psig in the drywell and 25 psig in the suppression chamber); 20 psig upward (55 psig in the suppression chamber and 35 psig in the drywell).

c. Thermal Loads

   The temperatures above and below the diaphragm slab for the operating and the postulated design accident conditions are shown in Table 3.8-3. The portions of the reactor pedestal above and below the diaphragm slab are designed for the drywell and suppression chamber maximum temperatures listed in Table 3.8-3.

   Thermal effects anticipated at the time of the structural acceptance test are insignificant, since the difference in temperatures inside and outside the containment during the test is small.

d. Pipe Rupture Loads

   The diaphragm slab and the reactor pedestal are designed to withstand the pipe rupture loads due to a postulated rupture of a 28 inch diameter recirculation loop pipe, which produces the largest loads on the structures. These loads include the effects of jet impingement, pipe whip, and pipe reaction. An equivalent static load of 1000 kips is considered. This load includes an appropriate dynamic load factor to account for the dynamic nature of the load. Section 3.6 contains a detailed discussion of postulated pipe ruptures and their effects.

3.8.3.3.2 Reactor Shield Wall

The reactor shield wall was designed as a steel member using AISC working stress methods. This wall is filled with a high density grout that has a main function of shielding. No composite action is assumed between the steel and the concrete. Table 3.8-5 lists the load combination used for the design of the reactor shield wall. The governing loading combination is the abnormal/extreme loading condition that combines the design basis accident loads with the maximum seismic loads. Descriptions of the loads are as follows:

a. Dead Load, Live Load, and Seismic Loads

For a description of dead load, live load, and seismic loads, see Section 3.8.1.3.

b. DBA Pressure Load

The reactor shield wall is designed for internal pressurization due to a postulated pipe rupture at the connection of the pipe to the reactor vessel nozzle safe end. The two following pressure conditions are considered:

1. Maximum unbalanced pressure: a pressure condition shortly after pipe break, which produces the largest lateral load on the reactor shield wall, resulting in a jet force of 1330 kips.

2. Maximum uniform pressure: a 94 psig internal pressure

c. Thermal Loads

The reactor shield wall is designed for the maximum drywell temperature listed in Table 3.8-3.

d. Pipe Rupture Loads

The reactor shield wall is designed to withstand the pipe rupture loads due to a postulated rupture of any high energy pipe penetrating the reactor shield wall, and connecting to the reactor vessel, such as the recirculation and feedwater pipes. These loads include the effects of jet impingement, pipe whip, and pipe reaction. Equivalent static loads are considered, which include an appropriate dynamic load factor to account for the dynamic nature of the load. Section 3.6 contains a detailed discussion of postulated pipe ruptures and their effects.

3.8.3.3.3 Suppression Chamber Columns

Table 3.8-6 lists the load combinations used for the design of the suppression chamber columns. The columns are designed to resist the reaction loads from the diaphragm slab, for the LOCA conditions. Section 3.8.3.3.1 includes a description of the diaphragm slab loads. Normal loading conditions are not listed because the abnormal loading conditions govern the design. Allowable stresses for the suppression chamber columns are in accordance with AISC (1970), 7th edition, for steel.

3.8.3.3.4 Pipe Whip Restraints/Drywell Platforms

The drywell platforms are designed using the AISC working stress design methods, except for the pipe whip restraints supported on the platforms. Design of the pipe whip restraints and their support structure allows inelastic deformations due to postulated pipe rupture loads. When considering elastic behavior, the design of pipe whip restraints is based on AISC working stress method with allowables up to 0.9 times the yield strength. For elasto-plastic behavior, the design is in accordance with the procedure outlined in Reference 3.8-6. However, there is no loss of restraint function due to pipe break loads. All safety-related items which the inelastic deformations may affect are evaluated to verify that no required safety function would be compromised.

Table 3.8-7 lists the load combinations used to design the pipe whip restraints and the drywell platforms. Design accident pressure loads are small in comparison with pipe restraint and jet force loads and therefore do not affect the designs of the drywell platforms. The thermal load has been included in the loading combination as given in Table 3.8-7. Seismic loads due to the dead weight of the platform members are insignificant when compared with pipe rupture loads and are ignored. The uniform design live load for the grating and framing beams is 200 lb/ft^2. The live load for the framing beams also includes the gravity load, thermal reaction load, and seismic SSE reaction load of all piping and equipment supported on the beams. This design procedure is also applicable to pipe whip restraints and their support structures outside of the containment structure.

3.8.3.3.5 Seismic Truss

The seismic truss is designed using the AISC working stress design methods. It is designed primarily for lateral seismic loads. However, it is also designed for jet impingement loads due to the postulated rupture of a 26 inch diameter main steam pipe. Design accident pressure and design live loads are negligible in comparison with the main steam pipe jet forces and therefore do not affect the design of the seismic truss. The expansion of structural members under thermal loadings is permitted. Therefore, thermal loads are negligible. Table 3.8-8 lists the governing load combination used to design the seismic truss.

3.8.3.4 Design and Analysis Procedures

This section describes the procedures used for design and analysis of the containment internal structures. All computer programs referenced are described in Section 3.8.7.

For a description of the design and analysis procedures that consider the effects of hydrodynamic loads resulting from MSRV discharge and LOCA phenomena, refer to Reference 3.8-1 and Appendix 3A.

3.8.3.4.1 Diaphragm Slab

The design and analysis procedures used for the diaphragm slab are similar to those used for the containment wall. Computer programs used in the analysis include 3D/SAP, CECAP, ME620, and seismic programs. See Section 3.8.1.4 for a detailed description of the analysis procedures.

Figure 3.8-44 shows the 3D/SAP finite-element model used to analyze the diaphragm slab for all loads, other than the seismic loads. A 15° wedge of the diaphragm slab is modeled using solid finite-elements having isotropic, linear-elastic material properties. The model includes the diaphragm slab, suppression chamber wall, reactor pedestal below the diaphragm slab, and a suppression chamber column. One vertical boundary plane goes through a suppression chamber column, and the other is halfway between two columns. Boundary conditions are imposed on the analytical model by specifying nodal point forces, or displacements. With reference to Figure 3.8-44, the nodal points lying along boundary A are allowed to move within the X-Z plane, and those along boundary B within the X-Y plane. Points along boundary C are prevented from moving in the hoop direction. Points along boundary D are prevented from moving in the radial direction, to account for the restraining effect of the inner portion of the diaphragm slab. Nodal forces, moments, and shears are applied to boundaries E and F, to account for reaction loads from the drywell wall and the reactor pedestal above the diaphragm slab, respectively.

Analytical techniques as described in Section 3.7 are used to analyze the diaphragm slab for seismic loads.

3.8.3.4.2 Diaphragm Slab Liner Plate and Anchorages

The design and analysis of the diaphragm slab liner plate and anchorages is in accordance with Reference 3.8-3.

3.8.3.4.3 Reactor Pedestal

The reactor pedestal is designed for axisymmetric loads using the FINEL computer program. Both concrete and reinforcing steel materials are included in the model. The operating and design accident temperature gradients are computed using the ME620 computer program. For transient loads such as design accident pressure and thermal loads, the most critical combination of these loads is considered. Figure 3.8-45 shows a vertical section through the FINEL model of the containment, used to analyze the reactor pedestal below the diaphragm slab. Points along boundary A are prevented from moving in the vertical direction, and points along boundary B are prevented from moving in the radial direction.

Figure 3.8-46 shows the FINEL model used to analyze the reactor pedestal above the diaphragm slab. The model includes the reactor pedestal above the diaphragm slab, and portions of the reactor vessel and the reactor shield wall. Local thermal effects at the top of the reactor pedestal due to heat input from the reactor vessel are determined by using the ME620 computer program. With reference to Figure 3.8-46, nodal points along boundary A are prevented from moving vertically and radially. Nodal forces, moments, and shears are applied to boundaries B and C to account for reaction loads from the reactor vessel and the reactor shield wall, respectively.

Nonaxisymmetric loads on the reactor pedestal include seismic loads, and reactor vessel and reactor shield reaction loads. Seismic forces, moments, and shears are calculated by the methods described in Section 3.7. Vertical forces, horizontal shears, and overturning moments at the base of the reactor shield wall are determined as described in Section 3.8.3.4.4. These loads are applied to the top of the reactor pedestal. Concrete and reinforcing steel stresses in the reactor pedestal due to the above loads are calculated using the design methods of ACI 307. ACI 307 includes equations for determining the neutral axis of reinforced concrete cylindrical shells subjected to axial force and overturning moment. The position of the neutral axis satisfies the equilibrium of internal stresses and external forces and moments.

Concrete and reinforcing steel stresses due to axisymmetric and nonaxisymmetric loads are combined to determine the total stress. Additional meridional, hoop, and shear reinforcement is provided at the top of the pedestal, as shown in Figure 3.8-38, to resist local loads on the pedestal from the reactor vessel and the reactor shield wall. Helical reinforcement is not needed in the reactor pedestal to resist seismically induced tangential shears. Meridional and hoop reinforcement is designed to carry the entire tangential shear by shear friction, using the design methods of ACI 318.

3.8.3.4.4 Reactor Shield Wall

The reactor shield wall is analyzed as an axisymmetric structure. The FINEL computer program is used for the axisymmetric loads, which include dead load and design accident thermal load. The temperature gradient across the thickness of the wall is determinedby using the ME620 computer program. For nonaxisymmetic loads, which include design accident pressure load, seismic load, and pipe rupture load, the ASHSD computer program is used. Figure 3.8-47 shows a vertical section through the model used for FINEL and ASHSD programs. Points along boundary A are prevented from moving vertically and radially. For nonaxisymmetric loads, boundary B, at the connection of the seismic truss to the containment wall, is prevented from moving radially. Total stresses in the reactor shield wall are determined by summing the axisymmetric and nonaxisymmetric stresses.

A second set of analyses of the reactor shield wall, as shown in Figure 3.8-48, was done by modeling a full 360° section of the wall, and by using the EASE finite-element computer program. This analysis ensures the integrity of the shield for loads equivalent to the DBA. The model includes one 64 inch diameter recirculation outlet penetration, and two adjacent 48 inch diameter recirculation inlet penetrations. Finer mesh sizes are used in the areas of the openings to obtain a good representation of the stress gradient. With reference to Figure 3.8-48, points along boundary A are prevented from moving vertically and radially. Boundary B is a free edge. No significant local stress concentrations are noted around the openings. The stiffening of the shell is provided by the thick walled penetration sleeves.

3.8.3.4.5 Suppression Chamber Columns

Axial force, shear, and moment in the columns due to axisymmetric loads, such as dead load, and design accident pressure and thermal loads, were determined using the FINEL computer program. Figure 3.8-45 shows the FINEL model of the containment used to analyze the suppression chamber columns. Boundary conditions are described in Section 3.8.3.4.3. Since the FINEL program can consider only axisymmetric structures, the 12 columns are modeled as an equivalent cylinder having the same axial stiffness as the total of the individual column stiffnesses. The axial force in the columns is calculated from the axial stress determined by the FINEL program. Shear and moment in the columns are calculated from relative displacements of the diaphragm slab and the base foundation slab determined by the FINEL program.

Axial force, shear, and moment in the columns due to seismic loads are determined using several methods. Axial force in the columns due to horizontal seismic load is determined using the ASHSD program. Figure 3.8-49 shows the model. Axisymmetric shell and solid finite-elements having isotropic, linear-elastic material properties are used. Nodal points lying along boundary A are prevented from moving vertically, and points along boundary B are prevented from moving radially. The load applied to the ASHSD model is the seismic horizontal shear and overturning moment for the containment calculated by the methods described in Section 3.7.

Shear and moment in the columns due to horizontal seismic load are determined using the analytical procedures described in Section 3.7. The lumped-mass model of the containment, including columns and downcomers, is shown in Figure 3.8-50.

Axial force in the columns due to vertical seismic load is determined by applying the vertical forces calculated from the containment seismic analysis, to the diaphragm slab at its connections to the containment wall and the reactor pedestal. The vertical force transmitted to the columns through the diaphragm slab is calculated considering the relative vertical stiffnesses of the containment wall, reactor pedestal, and columns.

The postulated rupture of a 28 inch diameter recirculation loop pipe produces a vertical jet impingement load on the top of the diaphragm slab, and, therefore, produces loads in the columns. Axial force, shear, and moment in the columns due to jet force are calculated by the CE 668 computer program. Figure 3.8-51 shows the 180° model of the diaphragm slab. A vertical jet force is applied along the axis of symmetry, and the reaction is calculated in the column adjacent to the applied load. Edges of the diaphragm slab along boundaries A and B are considered to have fixed supports.

The total axial force, shear, and moment in the columns for all load combinations are determined by summing the results of the separate analyses. Stability of the columns for the most critical load combinations is checked using the plastic design methods of Part 2 of the AISC "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings."

3.8.3.4.6 Drywell Platforms

The drywell platforms are designed using conventional elastic design methods, in accordance with the AISC Specification, Part 1.

3.8.3.4.7 Seismic Truss

Seismic forces in the seismic truss are calculated using the methods described in Section 3.7. Axial force, shear force, and moment in the seismic truss due to postulated pipe rupture loads are calculated using moment distribution. Figure 3.8-52 shows the rigid frame model including boundary conditions.

3.8.3.5 Structural Acceptance Criteria

3.8.3.5.1 Reinforced Concrete

The allowable stresses for the reinforced concrete portions of the containment internal structures are the same as the allowable stresses for the reinforced concrete portions of the containment, as discussed in Section 3.8.1.5.1. The calculated stresses are within the allowable limits.

3.8.3.5.2 Diaphragm Slab Liner Plate and Anchorages

The structural acceptance criteria for the diaphragm slab liner plate and anchorages are in accordance with Reference 3.8-3.

3.8.3.5.3 Structural Steel

Structural steel portions of the containment internal structures include the reactor shield wall, suppression chamber columns, drywell platforms, and seismic truss. For normal loading conditions, the allowable stresses are in accordance with the AISC Specification. For extreme environmental and abnormal loading conditions, the allowable stresses are as follows:

a. Bending - 0.90 F_y

b. Axial tension or compression - 0.85 F_y, except that where allowable stress is governed by requirements of stability (local or lateral buckling), allowable stress does not exceed 1.5 F_s

c. Shear - 0.50 F_y

where:

F_s = allowable stress according to the AISC Specification, Part 1

F_y = specified yield strength of structural steel

The calculated stresses in all the structural steel elements are within the above allowable limits.

3.8.3.6 Materials, Quality Control, and Special Construction Techniques

The criteria of ACI 349, "Proposed ACI Standard: Code Requirements for Nuclear Safety-Related Concrete Structures," applicable to this section of the UFSAR are not used by LGS. This section discusses the alternate criteria used.

3.8.3.6.1 Concrete Containment Internal Structures

The concrete and reinforcing steel materials for the containment internal structures are discussed in Section 3.8.6.

3.8.3.6.2 Diaphragm Slab Liner Plate, Anchorages, and Attachments

a. Materials

Liner plate materials conform to the requirements of the following standard specifications:

Item	Specification
Liner plate (¼ inch thick)	ASTM A285, Grade A, Firebox Quality
Liner plate (>¼ inch thick)	ASME SA516, Grade 60 conforming to the requirements of the ASME Section III, Article NE-2000
Anchorages and attachments	ASTM A36 or ASTM A441

b. Welding

Welding requirements for the diaphragm slab liner plate and anchorages are the same as the welding requirements for the containment liner plate and anchorages. See Section 3.8.1.6.2 for a description of the welding requirements.

c. Nondestructive Examination of Liner Plate Seam Welds

Spot radiographic examination is performed for all radiographable liner plate seam welds. All nonradiographable liner plate seam welds are 100% magnetic particle examined, and 100% vacuum box soap bubble tested. Welds that are inaccessible for vacuum box testing are 100% liquid penetrant tested. Examination procedures, personnel qualification, and acceptance standards are as described in Section 3.8.1.6.2.

3.8.3.6.3 Reactor Shield Wall and Seismic Truss

a. Materials

Materials used in construction of the reactor shield wall and the seismic truss conform to the following standard specifications:

Item	Specification
Inner reactor shield plate, seismic truss, and pipe restraint	ASTM A36 or ASTM A516, Grade 70
Outer reactor shield plate	ASTM A516, Grade 70

b. Welding and Nondestructive Examination of Welds

All welding and welder qualification procedures are in accordance with the requirements of ASME Section IX.

c. Materials Testing

The 1½ inch thick outer plate of the reactor shield wall is vacuum-degassed in accordance with supplementary requirements S-1 of ASTM A20, and is ultrasonically tested in accordance with ASME Section III, Subsection NB-2532-1.

d. Erection Tolerances

1. Each of the two concentric cylinders of the reactor shield wall is plumb within 1:500 of the height, plus a 1 inch allowance for local "out-of-roundness."

2. The radial dimension to any point on the reactor shield plates does not vary by more than ±1 inch from the center line as established by the design.

3. The clear distance between the two steel plates does not vary more than ±½ inch from the theoretical distance at any point.

4. The penetration sleeve center line is within ±1 inch of the projected center line of the theoretical location of RPV nozzles.

5. The elevation of the top of the shield is within ±¼ inch of that shown on the design drawings.

6. Seismic truss members do not deviate from axial straightness by more than 1/1000 of axial length.

3.8.3.6.4 Suppression Chamber Columns

a. Materials

The column shafts, base plates, and top plates are fabricated of ASME SA516, Grade 70 material.

b. Welding

Weld procedures and qualifications conform to the provisions of ASME Section III, Subsection NE, Class MC. All welders are qualified in accordance with ASME Section IX.

c. Nondestructive Examination of Welds

Complete magnetic particle examinations are performed on the welds of the suppression chamber columns, and on the top plate welds (anchor bolts to top plate and column to top plate) in accordance with the requirements of the ASME Section III, Subsection NE-5000.

d. Fabrication Tolerances

The specified fabrication tolerances for the suppression chamber columns are as follows:

1. The outside diameter, based on circumferential measurements, does not deviate from the theoretical outside diameter by more than 0.5%.

2. Out-of-roundness, defined by the difference between the maximum and minimum diameters related to the theoretical diameter, is in accordance with the ASME Section VIII, Division 1, Paragraph UG-80.

3. The finished length does not differ from the theoretical length by more than ¼ inch.

4. The finished column shaft does not deviate from straightness by more than 1/8 inch in 1 foot, with a maximum for the full length of 1/1000 of the total length.

3.8.3.6.5 Drywell Platforms

a. Materials

Materials used in construction of the drywell platforms conform to the following standard specifications.

Item	Specification
Box beams and built-up wide flange beams	ASTM A441 and ASTM A588, Grade A
Structural shapes, plate,	ASTM A36

and bar

Connection bolts ASTM A325

b. Welding

Welding is performed in accordance with the Structural Welding Code, AWS D1.1. Visual inspection and acceptance of structural welds is performed in accordance with NCIG-01 (Reference 3.8-22).

c. Nondestructive Examination of Welds

Welds in box beams and fabricated wide flange beams are subjected to magnetic particle examination in accordance with ASTM E109 (Reapproved 1971); the standards of acceptance are in accordance with paragraph 8.15 of AWS D1.1.

d. Erection Tolerances

Erection tolerances for the drywell platforms are in accordance with the AISC Specification.

3.8.3.6.6 Quality Control

Quality control requirements during construction are discussed in the document "Limerick Generating Station Units 1 and 2; Summary Description of the Quality Assurance Program for Design and Construction," referenced in FSAR Section 17.1.

3.8.3.7 Testing and Inservice Inspection Requirements

3.8.3.7.1 Preoperational Testing

3.8.3.7.1.1 Structural Acceptance Test

The diaphragm slab is tested to 1.15 times the design downward differential pressure. Section 3.8.1.7 contains a description of the structural acceptance test. Structural acceptance test results are available after testing is complete.

3.8.3.7.1.2 Leak Rate Testing

Preoperational leak rate testing is discussed in Section 6.2.6.

3.8.3.7.2 Inservice Leak Rate Testing

Inservice leak rate testing is discussed in Section 6.2.6.

3.8.4 OTHER SEISMIC CATEGORY I STRUCTURES

This section gives information on all seismic Category I structures, other than the primary containment and its internal structures. It also describes the turbine enclosure, which is a nonseismic Category I structure. The following structures are included in this section:

Seismic Category I, Safety-Related Structures

Secondary containment (Reactor enclosures and refueling area)
Control structure
Diesel generator enclosure
Spray pond pump structure
Spray pond
Miscellaneous structures

Seismic Category I, Nonsafety-Related Structure

Radwaste enclosure (including offgas portion)

Nonseismic Category I, Nonsafety-Related Structure

Turbine enclosure

The general arrangement of these structures is shown on drawing C-2.

3.8.4.1 Description of Structures

3.8.4.1.1 Secondary Containment

The reactor enclosures enclose the primary containments and, with the refueling area, provide secondary containment (Figures 1.2-2 through 1.2-16). The secondary containment houses the auxiliary systems of the NSSS, the spent fuel pool, the refueling facility, and equipment essential to the safe shutdown of the reactor. The secondary containment is structurally integral with the control structure described in Section 3.8.4.1.2.

The secondary containment, up to and including the roof slab, is of reinforced concrete construction. Exterior bearing walls are reinforced concrete, and are additionally designed as shear walls to resist lateral loads. The floors and roof are constructed of reinforced concrete, supported by steel beam and column framing systems. The concrete slabs are designed as diaphragms to transmit lateral loads to the shear walls. The structural steel beams and girders are supported by either structural steel columns, or reinforced concrete bearing walls. The steel columns are supported by base plates attached to the foundation. The reinforced concrete walls and floors meet structural, as well as radiation shielding, requirements. At certain locations, concrete block masonry walls are used to provide better access for erecting and installing equipment. The block walls also meet the structural and the radiation shielding requirements.

The refueling facility is located above the reactor enclosures. It consists of the spent fuel pool, the steam dryer and separator storage pool, the reactor well, the cask loading pit, the skimmer surge tank vaults, a 48 foot long refueling platform crane, and a 129 foot long reactor enclosure crane. The facility is supported by end bearing walls, and by two post-tensioned concrete girders with grouted tendons. The girders run east-west, and span over the primary containments without intermediate supports. Each girder spans approximately 162 feet, and is 6 feet wide. The depth is 46 feet at the supports, and is reduced to 26 feet at midspan, where the girders straddle the containments. The ends of the girders are supported by concrete pilasters. A gap between the bottom of the girders and the top of the containments ensures that loads from the refueling facility are not transferred to the containment. The details of the post-tensioned girders, including the tendon layout, are shown in Figure 3.8-53. The walls and slabs of the spent fuel pool, the cask loading pit, the reactor cavity, and the steam dryer and separator storage pool are lined on the inside with a stainless steel liner plate. The refueling facility meets the radiation shielding requirements.

The reactor enclosure crane consists of a main and an auxiliary hoist, with capacities of 125 tons and 15 tons, respectively. The crane is used during maintenance and refueling operations. It spans approximately 129 feet, and is 28 feet above the refueling floor. The crane is mounted on two 175 pound rails, supported by a pair of runway girders. The runway girders are supported by a series of built-up columns spaced at 27 foot centers, which in turn are supported by bearing walls. Figure 3.8-54 shows the details of the runway girders and the supporting columns. The reactor enclosure crane is discussed in Section 9.1.5.

The reactor enclosure is separated from the primary containment by a gap filled with compressible material. A gap is also provided at the interface of the secondary containment with the diesel generator, radwaste, and turbine enclosures.

3.8.4.1.2 Control Structure

The control structure, shown in drawings M-110, M-111, M-112, M-113, M-114, M-115, M-124, M-125, M-126, M-127, M-128, M-129, and M-130, is a reinforced concrete enclosure, structurally integrated with the secondary containment. The bearing walls are of reinforced concrete, and are additionally designed as shear walls to resist lateral loads. The floors and roof are constructed of reinforced concrete supported by steel beams. They are designed as diaphragms to transmit lateral loads to the shear walls. The beams span in the north-south direction and are supported at the ends by the bearing walls. The reinforced concrete walls and floors meet structural, as well as radiation shielding requirements. At certain locations, concrete block masonry walls are used to provide better access for erection and installation of equipment. The block walls also meet the structural and radiation shielding requirements.

The control structure is separated from the turbine enclosure by a seismic gap.

3.8.4.1.3 Diesel Generator Enclosure

The diesel generator enclosures, shown in drawings M-145 and M-146, house the standby diesel generators, which are essential for safe shutdown of the plant.

Concrete walls separate each diesel generator enclosure into four cells, one for each of the four diesel generators provided per unit. Each diesel generator unit is enclosed in its own concrete missile-protected cell. The walls between each generator unit are 2 feet thick and are 3 hour fire rated. All penetrations are small (2-3 inches), cast in concrete conduits for electrical cables. A concrete overhang on the south side of the enclosure serves as an air intake plenum. A concrete exhaust plenum is located on the north side of the enclosure roof.

The diesel generator enclosure is a reinforced concrete structure on wall foundations. The bearing walls are of reinforced concrete, and are additionally designed as shear walls to resist lateral loads. The floors and roof are constructed of reinforced concrete supported by steel beams. They are designed as diaphragms to transmit lateral loads to the shear walls. The north side of the enclosure bears on the pipe tunnel beneath. At certain locations, concrete block masonry walls are used to provide better access for erection and installation of equipment. The diesel generators are supported by the floors.

The diesel generator structures provide protection from missiles generated by internal rotating or pressurized mechanisms to prevent any credible missiles from damaging more than one engine. Any postulated missiles from a crankcase explosion are expected to be of low energy and incapable of penetrating the concrete barrier into any other cell. Failure of one or all of the starting air receivers by explosion is not expected to produce any credible missiles capable of penetrating the 2 foot thick cell walls. The diesel generator piping is designed to withstand pipe breaks and cracks.

3.8.4.1.4 Spray Pond Pump Structure

The spray pond pump structure, shown in drawings M-388, M-389 and M-390, contains the ESW and RHRSW pumps, auxiliary equipment, and related piping and valves.

The spray pond pump structure is a two-story reinforced concrete structure. The bearing walls are of reinforced concrete, and are additionally designed as shear walls to resist lateral loads. The operating floor and roof are constructed of reinforced concrete supported by steel beams. They are designed as diaphragms to transmit lateral loads to the shear walls. A mezzanine floor composed of grating over steel beams is provided to support the heating and ventilating equipment. An intermediate floor in the wing areas is provided to support valves and piping.

3.8.4.1.5 Spray Pond

The spray pond serves as the UHS for the plant. It is shown in Figures 3.8-55 through 3.8-57. The operation of the spray pond system is discussed in Section 9.2.6.

Spray pond dimensions are given in Table 9.2-18. The spray pond is designed so that normal operating water is retained in excavation only, i.e., not by constructed embankments.

An emergency spillway is provided at the north side of the pond. The only anticipated use of this spillway is either during a malfunction of the blowdown line, or during certain postulated conditions of heavy rainfall. The emergency spillway is designed to ensure that the maximum water level does not adversely affect the spray pond system, and to direct run-off water away from safety-related facilities in a controlled manner. The roadway surrounding the remainder of the spray pond provides a minimum freeboard of 4 feet.

The bottom and soil cut slopes of the spray pond are lined with a 12 inch thick layer of soil-bentonite lining. The soil-bentonite lining is covered with a 12 inch thick layer of soil. The rock-cut slopes are lined with shotcrete.

The stability of the subsurface materials, slopes, and lining at the spray pond site are discussed in Sections 2.5.4 and 2.5.5. Protection of slopes against waves is discussed in Section 2.5.5.

The spray network piping, which is located above the water, is supported by reinforced concrete columns. The columns are founded on bedrock or on concrete fill on top of bedrock.

3.8.4.1.6 Miscellaneous Structures

Subgrade pits, manholes, and tunnels which contain safety-related components are constructed of reinforced concrete. The locations of these miscellaneous structures are shown on Figure 3.8-58.

Safety-related piping, tanks, and electrical ducts, which are not located inside structures, are buried underground with adequate cover for missile protection. Additionally, soil erosion due to failure of nonseismic piping has also been considered. The integrity of safety-related seismic Category I buried pipe will not be impaired through soil erosion by a failure of one buried nonseismic Category I pipe. This conclusion was based on the following conditions.

a. All but approximately 170 feet of common trench has been constructed in rock, where erosion of the supporting medium would be insignificant.

b. The 170 feet of trench not constructed in rock is in Type 1 fill.

   The nonpressure pipes (gravity lines) in this section, such as the blowdown line and waste and storm lines, do not pose a significant erosion problem. The nonseismic Category I pressure lines in this section consist of a 36 inch Schuylkill River makeup water line and a 12 inch fire line (Figure 3.8-58).

   Failure of the nonseismic Category I pressure pipe may create progressive erosion in the Type 1 fill. It is anticipated that water under pressure would penetrate to the surface, creating a progressively enlarging crater. However, because the water will flow in the direction of least resistance, once the water penetrates to the surface, the crater will be enlarged at a relatively slow pace. The span capacity needed to support the weight of the safety-related pipes in the trench is conservatively estimated to be in excess of 30 feet, based on the maximum allowable spans given in the ASME code. A considerably long time would be required to erode a crater large enough to exceed this span capacity.

c. Instrumentation would give indication in the control room if a break occurred in the nonseismic Category I pressure pipe. Loss of flow from the makeup water line to the cooling tower would result in an alarm in the control room when low level is reached in the cooling tower basin. It is conservatively estimated that low level would be reached within 30 minutes.

   Low pressure in the 12 inch fire line, following a break, starts a fire pump that gives an alarm in the control room without a fire signal.

   Following an SSE, if either alarm described above is activated, personnel will investigate for evidence of a faulty condition in the pipelines described in (b) above and will initiate any necessary corrective action.

d. The procedures for operator response to a seismic event will include the requirement that, within two hours after an SSE, personnel will investigate for evidence of a faulty condition in the pipelines described in (b) above and will initiate any necessary corrective action.

3.8.4.1.7 Radwaste Enclosure

The radwaste enclosure is designed in accordance with seismic Category I criteria, even though: its integrity is not required to protect the RCBP, or to ensure the capability to safely shut down the reactor; and its failure would not result in potential offsite exposures comparable to the guideline exposures of 10CFR50.67. The radwaste enclosure, shown in drawings M-140 through M-144, houses systems for receiving, processing, and temporarily storing the radioactive waste products generated during the operation of the plant.

The radwaste enclosure, which includes the offgas enclosure, is a reinforced concrete structure. The bearing walls are of reinforced concrete, and are additionally designed as shear walls to resist lateral loads. The exterior walls are waterproofed, and are designed for hydrostatic effects as necessary. The floors and roof in the main portion of the radwaste enclosure are constructed of reinforced concrete supported by beam and column framing systems. They are designed as diaphragms to resist lateral loads. The columns are supported by base plates on the foundation. The floors and roof of the offgas portion of the radwaste enclosure are of reinforced concrete supported by steel beams and bearing walls. The reinforced concrete walls and floors meet structural, as well as radiation shielding requirements. At certain locations, concrete block masonry walls are used to provide better access for erection and installation of equipment. The block walls also meet the structural and radiation shielding requirements.

The radwaste enclosure is separated from the turbine enclosure and reactor enclosure by seismic gaps.

3.8.4.1.8 Turbine Enclosure

The turbine enclosure, which is shown in drawings M-110, M-111, M-112, M-113, M-114, M-115, M-124, M-125, M-126, M-127, M-128, M-129, and M-130, is divided into two units, separated by an expansion joint. It houses two inline turbine-generator units, and auxiliary equipment including condensers, condensate pumps, moisture separators, air ejectors, feedwater heaters, reactor feed pumps, ASDs for reactor recirculation pumps, interconnecting piping and valves, switchgear, and heating and ventilating equipment.

Three 110 ton overhead cranes are provided above the operating floor for servicing both turbine-generator units. Two reinforced concrete tunnels, one for each unit, are provided for the offgas pipelines at the foundation level, running from the area around the control structure to the radwaste enclosure.

The turbine enclosure rests on a reinforced concrete mat foundation. The superstructure is framed with structural steel and reinforced concrete. Rigid steel frames support the two turbine enclosure cranes. They also resist all transverse (north-south) lateral loads. Steel bracings resist longitudinal (east-west) lateral loads above the operating floor. Below this level, reinforced concrete shear walls transfer all lateral loads to the foundations.

Seismic separation gaps are provided at the interface of the turbine enclosure with the reactor, control, and radwaste enclosures.

The floors of the turbine enclosure are of reinforced concrete supported by structural steel beams. They are designed as diaphragms for lateral load transfer to the shear walls. The roof is built-up roofing on metal decking.

Exterior walls are covered by nonstructural precast reinforced concrete panels.

Interior walls required for radiation shielding or fire protection are constructed of reinforced concrete block. These walls are not used as elements of the load resistant system.

The seismic Category II turbine enclosure may undergo some plastic deformation under seismic loading resulting from the SSE, but the plastic deformation is limited to a ductility factor of 2. Those portions of the turbine enclosure which support the main steam lines are designed so that the main steam lines and their supports maintain their integrity under the seismic loading resulting from the SSE. Furthermore, the Turbine Enclosure will maintain its integrity to ensure that the MSIV Leakage Alternate Drain Pathway will be capable of performing its function as described in Section 6.7 during and following an SSE.

The turbine-generator units are supported on freestanding reinforced concrete pedestals. The mat foundations for the pedestals are founded on rock at the same levels as the basemat for the turbine enclosure. Separation joints are provided between the pedestals, and the turbine enclosure floors and walls, to prevent transfer of vibration to the enclosure. The operating floor of the turbine enclosure is supported on vibration damping pads at the top edge of the pedestal.

3.8.4.2 Applicable Codes, Standards, and Specifications

The codes, standards, and specifications used in the design, fabrication, and construction of seismic Category I structures are listed in Table 3.8-1.

3.8.4.3 Loads and Load Combinations

Tables 3.8-9, 3.8-10, and 3.8-11 list the loading combinations considered in the design seismic Category I structures (other than the primary containment).

The reactor enclosure and control structure are also assessed for Mark II hydrodynamic loads resulting from MSRV discharge and LOCA phenomena. For a definition of loads and loading combinations including hydrodynamic loads, refer to Reference 3.8-1 and Appendix 3A. Hydrodynamic loads have no significant effect on other seismic Category I structures described in Section 3.8.4.

3.8.4.3.1 Description of Loads

3.8.4.3.1.1 Normal Loads

Normal loads are loads which are encountered during normal plant operation and shutdown. They include dead loads, live loads, thermal loads due to operating temperature, and other permanent loads contributing stress, such as hydrostatic loads. Dead and live loads are described in Sections 3.8.1.3.1 and 3.8.1.3.2, respectively.

3.8.4.3.1.2 Severe Environmental Loads

Severe environmental loads are loads that could infrequently be encountered during the plant life. They include those loads induced by the OBE and the design wind. Loads due to OBE are discussed in Sections 3.7 and 3.8.1.3.6. Wind loads are discussed in Section 3.3.

3.8.4.3.1.3 Extreme Environmental Loads

Extreme environmental loads are loads which are credible, but which are highly improbable. They include those loads induced by the SSE and the design tornado. Loads due to the SSE are discussed in Sections 3.7 and 3.8.1.3.6. Tornado loads are discussed in Section 3.3.

3.8.4.3.1.4 Abnormal Loads

Abnormal loads are loads generated by a DBA. Abnormal plant conditions generated by a DBA include the postulated rupture of high energy piping. Loads induced by such an accident include elevated temperatures and pressures within or across compartments, and jet impingement and impact forces associated with such ruptures. Loads due to postulated rupture of piping are discussed in Section 3.6. Other loads that may be generated by a DBA include railroad accident blast loads and tornado loading (including missiles generated from both), and loading from aircraft impact, as described in Sections 2.2.3 and 3.5.1.6.

3.8.4.3.2 Load Combinations

Table 3.8-9 describes the load combinations applicable to the reactor enclosure and control structure. Table 3.8-10 contains the load combinations applicable to seismic Category I structures, other than the primary containment, reactor enclosure, and control structure. Load combinations considered in the design of miscellaneous structural components of seismic Category I structures are listed in Table 3.8-11. Applicable load combinations from Table 3.8-10 are used in the design of the turbine enclosure.

3.8.4.4 Design and Analysis Procedures

The structures are designed to maintain elastic behavior for the load combinations defined in Section 3.8.4.3. All reinforced concrete components of the structure are designed by the ultimate strength method, in accordance with ACI 318. Design of major reinforced and prestressed concrete structural components was completed prior to the formal adoption of ACI 349, and this standard was therefore not used for LGS. All structural steel components are designed by the working stress method, in accordance with the AISC Specification.

Roof and floor diaphragms transfer the horizontal loads to the shear walls, which then transfer the loads to the foundation. Standard analytical procedure is used to analyze the distribution of lateral loads from the diaphragms to the shear walls. Vertical loads are transferred to the foundation by bearing walls and steel columns.

Seismic analysis of the structures uses the techniques described in Section 3.7. The enclosures are analyzed dynamically.

Design of structures for missile and aircraft protection is covered in Section 3.5.3.

The design and analysis of the fuel pool girders of the reactor enclosures are in accordance with ACI 318.

Design and analysis of concrete block walls are by the working stress method, in accordance with the UBC.

The stainless steel liner plates, used on the inside of the spent fuel pool, the cask loading pit, the reactor well, and the steam dryer and separator storage pool, are designed to function as a leak-tight membrane and to facilitate decontamination. The liner plate is designed to accommodate stresses due to the combined effect of long-term shrinkage of the underlying structural concrete and temperature effects inside the pools.

The spray pond is basically a soil-structure. Its design is discussed in Sections 2.5.4 and 2.5.5.

The reactor enclosure and control structure are also assessed for Mark II hydrodynamic loads resulting from MSRV discharge and LOCA phenomena. Assessment procedures that consider the effect of hydrodynamic loads are described in Appendix 3A.

3.8.4.5 Structural Acceptance Criteria

3.8.4.5.1 Reinforced and Prestressed Concrete

The reinforced and prestressed concrete structural components are designed by the strength method, in accordance with ACI 318, for the loads and load combinations described in Section 3.8.4.3. The stresses resulting from the load combinations are within allowable limits.

3.8.4.5.2 Structural Steel

The structural steel components are designed by the working stress method in accordance with the AISC Specification for the loads and load combinations described in Section 3.8.4.3. The allowable stresses for different load combinations are indicated therein. The stresses developed in the structural components are within the allowable limits.

3.8.4.5.3 Concrete Masonry Block Walls

The design of masonry walls was performed in accordance with the criteria in the UBC. The masonry wall design was re-evaluated for conformance with the criteria in NRC IE Bulletin 80-11. A comparison of the LGS design criteria for masonry walls with the SEB criteria shows that the two are in good agreement except in the following two areas:

a. Under extreme environmental loading conditions, the LGS criteria (appendix I, part 1, Reference 3.8-21) lists the following combinations:

1. $D + L + T_a + H_a + R + P + E'$

2. $D + L + W' + T_o + H_o$

where:

D	=	Dead load of structure and equipment plus any other permanent loads
L	=	Live loads expected to be present when the plant is operating
E'	=	Design basis earthquake loads
W'	=	Tornado loads
T_o	=	Operating temperature loads
T_a	=	Accident temperature loads
H_o	=	Thermal operating condition pipe load
H_a	=	Thermal accident condition pipe load
R	=	Jet impingement load and/or pipe whip
P	=	Pressurization load due to HELB

The SEB criteria, under similar conditions, lists the following load combinations:

1. $D + L + T_o + R_o + E'$

2. $D + L + T_o + R_o + W_t$

3. $D + L + T_a + R_a + 1.5P_a$

4. $D + L + T_a + R_a + 1.25P_a + 1.0\ (Y_r + Y_j + Y_m) + 1.25E$

5. $D + L + T_a + R_a + 1.0P_a + 1.0\ (Y_r + Y_j + Y_m) + 1.0E'$

where:

R_o	=	H_o of the LGS criteria
E	=	OBE loads
W_t	=	W' of the LGS criteria
R_a	=	H_a of the LGS criteria
P_a	=	P of the LGS criteria
Y_r	=	Equivalent static load on the structure generated by the reaction on the broken high energy pipe during the postulated break, and including an appropriate dynamic load factor to account for the dynamic nature of the load.
Y_j	=	Jet impingement equivalent static load on a structure generated by the postulated break, and including an appropriate dynamic load factor to account for the dynamic nature of the load, = R of the LGS criteria.
Y_m	=	Missile impact equivalent static load on a structure generated by or during the postulated break, as from pipe whipping, and including an appropriate dynamic load factor to account for the dynamic nature of the load, = R of the LGS criteria.

Comparing the load combination under the two criteria, it is seen that combinations (1) and (2) of the LGS criteria are the same as combinations (v) and (ii) of the SEB criteria. Further, combination (v) of SEB is clearly more severe than combination (k) of SEB which is, therefore, not used in the LGS criteria. A study made on the masonry walls and their applicable load combinations indicated that load combination (v) of SEB will, in most cases, be more severe than combinations (iii) and (iv). For the cases where this was not true, the difference in loadings was not significant and additional calculations were made to ensure that the walls would be structurally adequate under SEB load combinations (iii) and (iv).

The results of the re-evaluation contained in Reference 3.8-21 are unaffected if the SEB load combinations (iii) and (iv) are considered in addition to the LGS load combinations in the re-evaluation of the masonry walls.

b. The LGS criteria permits the use of an increase factor for shear and bond of 1.67 which, when multiplied with the allowable stresses for normal operating conditions, gives that allowables for extreme environmental conditions. The increase factors under the SEB criteria are 1.5 for tension parallel to bed joint and shear in the reinforcement and 1.3 for tension normal to the bed joint and masonry shear.

All block walls at LGS contain steel tension reinforcement which takes all tension normal or parallel to the bed joint. Therefore the LGS allowable masonry tension stress normal or parallel to the bed joint was not used, i.e., zero psi was assumed for the reanalysis of block walls. Because the masonry tension strength at the bed joint was assumed to be zero, the increase factor stated in the question was also not used in the reanalysis of the block walls. Further, the factor 1.67 has also not been used for shear or bond calculations for the masonry wall re-evaluation; instead a factor of 1.0 has been used.

Because the code allowable stresses (Reference 3.8-23, chapter 10.1 of the commentary) are generally associated with a safety factor of 3, the 1.67 increase factor for stresses for masonry load combinations involving abnormal and/or extreme environmental conditions, provides a factor of safety against failure of 1.8 (3 divided by 1.67). The factor of safety of 1.8 is conservative and allows sufficient margin for abnormal and/or extreme conditions.

Reference 3.8-21 provides additional details and results of this evaluation.

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

3.8.4.6.1 Reinforced Concrete, Masonry, and Prestressed Concrete

The reinforced concrete, masonry, and prestressed concrete materials are discussed in Section 3.8.6.

3.8.4.6.2 Structural Steel

3.8.4.6.2.1 Materials

The various structural steel components conform to the following ASTM Specifications:

Item	ASTM Specification
Beams, girders, and plates	A36 or A441
High strength bolts	A325 or A490
Anchor bolts	A36 or A307

3.8.4.6.2.2 Welding and Nondestructive Testing

All welding and nondestructive testing is performed in accordance with the AWS Structural Welding Code.

3.8.4.6.2.3 Fabrication

The fabrication of structural steel conforms to the AISC Specification.

3.8.4.6.3 Quality Control

Quality control requirements during construction are discussed in the document "Limerick Generating Station Units 1 and 2; Summary Description of the Quality Assurance Program for Design and Construction," referenced in FSAR Section 17.1.

3.8.4.6.4 Special Construction Techniques

No special construction techniques are involved in the construction of seismic Category I structures.

3.8.4.7 Testing and Inservice Inspection Requirements

Testing and inservice inspections are not required for seismic Category I structures (other than the primary and secondary containment). The operational test, to verify the inleakage requirements of SGTS in the reactor enclosure, is covered in Sections 6.5.3 and 9.4.2.

3.8.5 FOUNDATIONS

This section describes foundations for all seismic Category I structures, other than the primary containment and the spray pond. The primary containment foundation is described in Section 3.8.1. The spray pond is basically a soil-structure; its design is discussed in Sections 2.5.4 and 2.5.5. A description of the foundation for the turbine enclosure, which is a nonseismic Category I structure, is also included in this section.

3.8.5.1 Description of the Foundations

Reinforced concrete foundations resting on sound rock, or lean concrete on sound rock, are provided for the structures.

Bearing walls of the structures are rigidly connected to the foundation. Steel columns are attached to the foundation by base plates and anchor bolts. The bearing walls and the steel columns carry all the vertical loads; they also carry the vertical axial loads due to the overturning effect of lateral forces, from the structure to the foundation. Horizontal shears due to wind, tornado, and seismic loads are transferred to the shear walls by the roof and floor diaphragms. The shear walls transfer the horizontal shears to the foundation, and from there, to the foundation medium through friction and/or direct bearing. The sides of all structure foundations are keyed to the foundation rock by poured concrete. This helps transfer the horizontal shears to the foundation rock.

The foundation for each structure is separated from each adjacent foundation by a 1 inch minimum seismic gap, except along the east side of the radwaste enclosure foundation, which is supported by the reactor enclosure foundation (Figure 3.8-59).

Peripheral subterranean walls are designed to resist lateral pressures due to backfill and surcharge loads, in addition to dead loads, live loads, and seismic loads.

3.8.5.1.1 Reactor Enclosure and Control Structure

The foundation for the reactor enclosure and control structure is a single integral unit. The foundation plan and representative sections are shown in Figure 3.8-60. The reactor enclosure and control structure walls are supported on continuous wall footings. Columns are supported on spread footings. The wall footings and spread footings vary in dimension and reinforcing.

All of these elements of the foundation for the reactor enclosure and control structure are joined together by a continuous 3'-0" thick mat in the reactor structure area, and by a continuous 2'-0" thick mat in the control structure. The 3'-0" thick mat surrounds the primary containment basement, with a 1 inch seismic gap separating the two.

3.8.5.1.2 Diesel Generator Enclosure

Figure 3.8-61 shows the foundation plan and typical sections of the diesel generator enclosure. The wall foundations extend well below the diesel generator base slab to competent bedrock. The base slab is supported on fill, which for design purposes is considered to be both yielding and nonyielding. In addition, the base slab is designed to span between the wall foundations, without support from the fill.

3.8.5.1.3 Spray Pond Pump Structure

Figure 3.8-62 shows the foundation plan and typical sections of the spray pond pump structure. The walls are founded on competent bedrock. The walls do not have independent footings, but are connected monolithically by a 2'-0" thick slab at the foundation level.

3.8.5.1.4 Radwaste Enclosure

Figure 3.8-59 shows the foundation plan and typical sections of the radwaste enclosure. The walls of the radwaste enclosure are founded on competent bedrock and are supported on continuous wall footings. Columns are supported on spread footings. The wall footings and spread footings of the main portion of the radwaste enclosure are joined together by a 1'-10" thick slab at the foundation level.

The walls of the offgas portion of the radwaste enclosure are supported on continuous wall footings, connected by a 1'-6", or 2'-2" thick slab at the foundation level.

3.8.5.1.5 Turbine Enclosure

Figure 3.8-63 shows the foundation plan and typical sections of the turbine enclosure. The walls are supported on continuous wall footings. Columns are supported on spread footings. The wall footings and spread footings vary in dimension and reinforcing. All of these elements of the foundation are joined together by a continuous 1'-3" to 1'-6" thick mat.

3.8.5.1.6 Miscellaneous Structures

Figure 3.8-64 shows typical foundation details of some safety-related miscellaneous structures. These structures are founded on competent bedrock, compacted fill, or lean concrete which can safely withstand the superimposed loads. The walls of the subgrade pits, manholes, and trenches are connected monolithically with the floor slabs at the foundation level. The minimum thicknesses of the walls and the floor slabs are 1'-6" and 1'-0", respectively.

3.8.5.2 Applicable Codes, Standards, and Specifications

The codes, standards, and specifications used in the design, fabrication, and construction of structure foundations are listed in Table 3.8-1.

3.8.5.3 Loads and Load Combinations

The loads and load combinations used in the design of the primary containment foundation are described in Section 3.8.1.3. The loads and load combinations used in the design of other seismic Category I structure foundations are discussed in Section 3.8.4.3.

3.8.5.4 Design and Analysis Procedures

The foundations of all seismic Category I structures are designed in accordance with the same codes and standards as used in the design of the superstructure. The design and analysis procedures are discussed in Section 3.8.1.4 and Section 3.8.4.4 for the primary containment and other seismic Category I structures, respectively. The design loads and load combinations are discussed in Section 3.8.5.3.

The bearing walls and steel columns carry all the vertical loads from the structure to the foundation. The lateral loads are transferred to the shear walls by the roof and floor diaphragms, which then transmit them to the foundation. Determination of overturning moment due to seismic loads is discussed in Section 3.7.

Settlement of all seismic Category I structure foundations is considered negligible, as the foundations are supported by sound rock, or lean concrete on top of sound rock, except those of some yard facilities, such as valve pits, manholes, pipe trenches, electrical ducts, etc, which are supported on select fill and yard fill on dense residual fill, or on sound rock.

The sliding of seismic Category I structure foundations cannot occur because, as explained in Section 3.8.5.1, the sides of the foundations are keyed to the foundation rock by poured concrete.

Detailed description of the foundation rock and soil is contained in Section 2.5.4.

3.8.5.5 Structural Acceptance Criteria

The seismic Category I structure foundations are designed to meet the same structural acceptance criteria as the structures themselves. These criteria are discussed in Sections 3.8.1.5 and 3.8.4.5. In addition, for the load combinations described in Section 3.8.5.3, the minimum factor of safety against overturning is 1.5.

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

The seismic Category I structure foundations are constructed of reinforced concrete. The reinforced concrete materials are discussed in Section 3.8.6. Cement for the ESW/RHRSW cross tie valve pits in the spray pond pump house yard is Type II. Concrete for these pits is commercial grade material dedicated for nuclear safety related use. The material characteristics identified in Section 3.8.6 were used as a basis for the dedication plan.

No special construction techniques are involved in the construction of these foundations.

3.8.5.7 Testing and Inservice Inspection Requirements

Foundation testing and inservice inspection are not considered necessary, since all seismic Category I structure foundations are supported on sound rock, or lean concrete on sound rock. The only testing carried out is as described in Section 3.8.1.7.

3.8.6 CONCRETE, REINFORCING, PRESTRESSED CONCRETE, AND MASONRY MATERIALS

Materials, workmanship, and quality control associated with the construction of reinforced concrete, prestressed concrete, and masonry features of seismic Category I structures are based on the codes, standards, recommendations, and specifications listed in Table 3.8-1. These documents are modified as required to suit the particular conditions associated with nuclear power plant design and construction while maintaining structural adequacy. The extent of application and principal exceptions are indicated herein, and as follows:

a. ACI 301-66

The provisions of ACI 301-66, "Specification for Structural Concrete for Buildings", are modified as follows:

1. The requirements of section 7.3.2 of ACI 318-71 apply in lieu of section 504(b).

2. The following requirements apply in lieu of section 804(b)2:

   (b)2 Hot weather - Concrete deposited in hot weather shall have a placing temperature which shall not cause difficulty from loss of slump, flash set, or cold joints and not greater than 90°F.

3. Vertical construction joints are cleaned and roughened by waterblasting, sandblasting, or bush hammering after the concrete has reached its final set. Prior to receiving additional concrete, vertical construction joints are wetted in lieu of the use of neat cement grout as specified in section 805(c).

4. The following requirements apply in lieu of section 901(a) and the fifth sentence of section 902(d):

   (a) Surface defects, such as form blowholes, honeycomb, etc., are repaired as soon as practicable, but no later than 28 days after form removal.

   (b) Form tie holes on the structures below grade level and in concrete which is coated with a special protective coating are patched as soon as practicable, but no later than 28 days after form removal. Form tie holes in areas other than the above need not be patched. Form tie holes which are patched need not be cured or kept moist.

5. The following requirements apply in lieu of the requirements specified in section 1202(a) and (c) and 1405(d):

The requirements of ACI 306-66 apply during cold weather. In addition, when heated enclosures are used for curing and freezing protection for all thicknesses of concrete, the temperature at the surface of the concrete is maintained between 50°F and 70°F. Any housing, covering, or other protection remains in place and intact for at least 24 hours after the artificial heating is discontinued. The maximum allowable gradual drop in temperature throughout the first 24 hours after the end of the 7 day curing period is 30°F for concrete thicknesses greater than 36 inches.

6. The following requirements apply in lieu of the requirements of sections 1401(a), 1404(a), 1404(c), 1405(a), and 1405(c), respectively:

   (a) Concrete sections more than 3 feet in the least dimension are termed mass concrete.

   (b) The maximum working limit slump of the concrete is 3 inches, with an inadvertency margin of +2 inches, except for starter mixes, which have a maximum slump of 5 inches.

   (c) Concrete is placed in layers not more than 24 inches, with vibrator heads extending into the previously placed layer.

   (d) The minimum curing period is 7 days.

   (e) The concrete is either moist cured or liquid membrane cured for 7 days. Liquid membrane compounds are applied at the time that the free water on the surface has disappeared, and no water sheen is seen, but not so late that the liquid curing compound is absorbed into the surface pores of the concrete.

   (f) The following requirements apply in lieu of section 601(c):

   The surface of the concrete at all joints is thoroughly cleaned, and all laitance removed, except where a mechanical joint is used, in which case all laitance need not be removed. The use of a mechanical joint without the removal of laitance is permitted in areas not exposed to excessive moisture.

b. <u>ACI 318-71</u>

The provisions of ACI 318-71, "Building Code Requirements for Reinforced Concrete", are modified as follows:

1. Section 3.5.1(a) of the 1974 Supplement to ACI 318-71 applies in lieu of section 3.5.1(a) of ACI 318-71.

2. Starter mixes, defined as concrete with a 1 inch maximum size aggregate 3 inches and 5 inches, are used as an alternate to mortar in section 5.4.4 and grout in section 6.4.1.

3. The following requirements apply in place of the requirements specified in section 5.5.1:

   The concrete is moist cured or liquid membrane cured for 7 days. Liquid membrane compounds are applied at the time that the free water on the surface has disappeared and no water sheen is seen, but not so late that the liquid curing compound is absorbed into the surface pores of the concrete.

4. The following requirements apply in place of the requirements specified in section 6.3.2.2:

   The temperature of the liquid, gas or vapor shall not exceed 150°F except in local areas such as penetrations through walls and slabs where the temperature shall not exceed 200°F.

5. The following requirements apply in place of the requirements specified in section 6.3.2.4:

   Piping and fittings are tested in accordance with the requirements of the code governing that piping system (e.g., ASME B&PV Code, ANSI B31.1, state or local plumbing codes, etc.), as specified in other sections of the UFSAR.

   Whenever the piping system is not governed by such applicable codes or code cases, such systems are tested for leaks prior to concreting. The testing pressure above atmospheric pressure is 50% above the pressure to which the piping and fittings may be subjected in service; but the minimum testing pressure is not less than 150 psig. The pressure test is held for 4 hours, with no drop in pressure, except that which may be attributable to changes in ambient air temperatures; or the system is maintained at the hydrostatic test pressure for a minimum of 10 minutes, while all joints are visually examined for leakage.

6. The following requirements apply in place of the requirements specified in section 6.3.2.6:

   Piping systems not governed by applicable codes or code cases, carrying liquid, gas, or vapor, which is potentially explosive or injurious to health, are retested in accordance with item b.4 above, subsequent to the hardening of the concrete.

7. The following requirements apply in place of the requirements specified in section 6.3.2.7:

   Piping systems may be energized with water exceeding neither 50 psig, nor 90°F at any time, if approved by the responsible field engineer.

   Piping systems, including systems governed by piping system codes, may be energized above either 50 psig or 90°F, or energized with fluids other than water during the period beyond 7 days after the concrete placement, provided that the temperature of the fluid does not exceed 150°F, and the pressure does not exceed 200 psig. Piping systems may be energized prior to, and remain energized during, placement of concrete, provided that: the above temperature and pressure restrictions are applied, and if the pressure in the energized system drops during concrete placement, the lowest pressure reached becomes the limiting pressure until the 7 day postplacement time limit has elapsed. However, if the pressure drops further within 24 hours after completion of the concrete placement, the system can be re-energized only up to the limiting pressure at any time after the 24 hour period has elapsed.

8. Vertical construction joints are cleaned and roughened by waterblasting, sandblasting, or bush hammering after the concrete reaches its final set. Prior to receiving additional concrete, vertical construction joints are wetted, in lieu of the use of neat cement grout specified in section 6.4.1.

9. The following requirements apply in lieu of the second sentence of section 6.4.1:

   Where a joint is to be made, the surface of the concrete is thoroughly cleaned. All standing water and laitance are removed, except where a mechanical joint is used, in which case all laitance need not be removed. The use of a mechanical joint without removing the laitance is permitted only in areas not exposed to excessive moisture.

10. The following requirements apply in lieu of the first sentence of section 4.14:

    f'_c for 5000 psi design strength concrete shall be based on tests of 90 days. f'_c for all other concrete design strengths shall be based on 28 day tests.

11. The following requirements apply in lieu of the last sentence of section 4.3.1:

    Each strength test result shall be the average of two cylinders from the same sample tested at 28 days for concrete with a design strength of 4000 psi or less and at 90 days for concrete with a design strength of 5000 psi.

c. <u>ACI 613-54</u>

The provisions of ACI 613-54, "Recommended Practice for Selecting Proportions for Concrete", are adhered to, except that the following recommended laboratory tests, mentioned on pages 211-14 through 211-16, are not used:

1. Fineness of Portland-Cement by Air Permeability Apparatus - ASTM Designation C204

2. Specific Gravity of Hydraulic Cement - ASTM Designation C188

3. Percentage of Shale in Aggregate - AASHTO Designation T10.

4. Alkali Reactivity, Potential, of Cement-aggregate Combinations - ASTM Designation C227

5. Air Content (volumetric) of Freshly Mixed Concrete - ASTM Designation C173

6. Air Content of Fresh Concrete by Pressure Methods (Washington-type meter) - Bureau of Reclamation Concrete Manual, Designation 24

7. Air Content of Freshly Mixed Concrete - U.S. Army Corps of Engineers Handbook for Concrete and Cement, Designation CRD-C 41

8. Laboratory Concrete Mixing - Bureau of Reclamation Concrete Manual, Designation 28

9. Flow of Portland-cement Concrete by Use of the Flow Table - ASTM Designation C124

10. Compressive Strength of Concrete Using Portions of Beams Broken in Flexure (modified cube method) - ASTM Designation C116

11. Flexural Strength of Concrete (using beam with third-point loading) - ASTM Designation C78

12. Fundamental Transverse and Torsional Frequencies of Concrete Specimens - ASTM Designation C215

13. Hardened Concrete, Securing, Preparing, and Testing Specimens from, for Compressive and Flexural Strengths - ASTM Designation C42

14. Cement Content of Hardened Portland-cement Concrete - ASTM Designation C85

15. Volume Change of Cement, Mortar and Concrete - ASTM Designation C157

16. Absorption of Concrete - AASHTO Designation T25

d. ACI 614-59

The provisions of ACI 614-59, "Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete", are modified as follows:

1. In addition to the requirements of ACI 614-59, storage of concrete materials shall be in accordance with ACI 304-73.

2. The following requirement applies in lieu of the second sentence of part II, item 6:

   The fineness modulus does not vary more than 0.20 from the value assumed in selecting proportions for the concrete.

3. The following requirements apply in lieu of the fourth sentence of part V, item 2:

   Concrete is not allowed or caused to flow to a distance, within the mass, more than the amount required for proper consolidation, and in no case more than 5 feet from the point of deposition.

4. The following requirements apply in lieu of the requirements of part VI, item 6:

   Immediately before the concrete is placed, all horizontal surfaces are covered with a nominal ½ inch of flowable grout mix thoroughly broomed into horizontal surfaces. For congested areas where brooming is impossible, the grout mix is forced ahead of the concrete. If the congestion may cause segregation, the grout thickness is increased to a nominal 1 inch thickness. Starter mixes, as defined above in item b.2, may be used as an alternate to the ½ inch or 1 inch grout. Starter mixes may be used to cover the bottom mat of reinforcing steel to a nominal depth of 2 inches above the reinforcing steel, or 6 inches above existing concrete on construction joints where no reinforcing mat exists.

5. The following requirement applies in lieu of the requirement of the fourth sentence of part VI, item 7:

   The maximum working limit slump of the concrete is 3 inches, with an inadvertency margin of +2 inches, except for starter mixes, which have a maximum slump of 5 inches.

e. <u>ACI SP-2, Fifth Edition</u>

The provisions of ACI SP-2, "Manual of Concrete Inspection", are modified as follows:

1. All samples are taken in accordance with ASTM C172.

2. Samples for cylinder strength tests of central mixed concrete are taken at the point of discharge from the central mixer plant. Samples for cylinder strength tests of truck mixed concrete, and slump, air content, and concrete temperature tests for all concrete are taken at the point of discharge from the delivery truck or at the end of the pump line for pumped concrete.

3. In order to establish a correlation between concrete properties at the central mixer plant and at the end of the pump line, pumped concrete is sampled at both of these locations to determine slump, air content, and concrete temperature. These correlation samples are taken from the same batch of concrete that is sampled for concrete cylinder strength tests at the central mixer plant, and again from this same batch of concrete as it reaches the end of the pump line. The correlation samples are taken at the following frequencies:

(a) Samples for slump and concrete temperature are taken from the first batch of each class of concrete produced per day, and for every 200 cubic yards of each class of concrete produced thereafter.

(b) Samples for air content are taken from every 400 cubic yards of each class of concrete produced, with a minimum of once per day for each class of concrete produced.

4. Vertical construction joints are cleaned and roughened by waterblasting, sandblasting, or bush hammering after the concrete has reached its final set. Prior to receiving additional concrete, vertical construction joints are wetted in lieu of using neat cement grout.

3.8.6.1 Concrete and Concrete Materials

3.8.6.1.1 Concrete Material Qualifications

3.8.6.1.1.1 Cement

Cement is Type II, Type I after June 1983, Portland-cement conforming to ASTM C150 except for RPV pedestal repair where the cement used is an expansive cement Type E-1(K) complying with ASTM C845. The following initial user tests are performed to ascertain conformance with ASTM Specification C-150:

Test	Designation
Chemical Analysis of Hydraulic Cement	ASTM C114
Fineness of Portland-Cement by the Turbidimeter	ASTM C115
Autoclave Expansion of Portland-Cement	ASTM C151
Time of Setting of Hydraulic Cement by Gillmore Needles	ASTM C266
Compressive Strength of Hydraulic Cement Mortars	ASTM C109
Air Content of Hydraulic Cement Mortar	ASTM C185

In addition, these tests are repeated during construction to check storage environmental effects on cement characteristics. The tests supplement visual inspection of material storage procedures.

Certified copies of material test reports showing chemical composition of the cement and verification that the cement being furnished complies with requirements are furnished by the manufacturer for each batch or lot.

3.8.6.1.1.2 Normal Weight Aggregate

Fine and coarse aggregates conform to ASTM C33. The following initial user tests are performed to ascertain conformance with ASTM C33:

Test	Designation
Organic Impurities in Sand for Concrete	ASTM C40
Effect of Organic Impurities in Fine Aggregate on Strength of Mortar	ASTM C87
Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate	ASTM C88
Materials Finer Than No. 200 Sieve in Mineral Aggregates by Washing	ASTM C117
Lightweight Pieces in Aggregate	ASTM C123
Specific Gravity and Absorption of Coarse Aggregate	ASTM C127
Specific Gravity and Absorption of	ASTM C128
Fine Aggregate	
Resistance to Abrasion of Small Size Coarse Aggregate by Use of the Los Angeles Machine	ASTM C131
Sieve or Screen Analysis of Fine and Coarse Aggregates	ASTM C136
Clay Lumps and Friable Particles in Aggregates	ASTM C142
Scratch Hardness of Coarse Aggregate Particles	ASTM C235
Potential Reactivity of Aggregates	ASTM C289
Petrographic Examination of Aggregates for Concrete	ASTM C295
Test for Flat and Elongated Particles in Coarse Aggregate (except that the acceptance criteria for the ratio of flatness and elongation shall be 4 in lieu of 3)	CRD C119

Coarse aggregate grading is in accordance with size Nos. 4, 57, and 67, as defined in ASTM C33. The quantity of flat and elongated particles in the coarse aggregate is limited to 15% by weight.

3.8.6.1.1.3 High Density Aggregates

The requirements specified for normal density aggregates apply for high density aggregates, except as noted below.

Fine and coarse aggregates have a minimum bulk specific gravity of 4.4, as determined by ASTM C127 and ASTM C128. The high density aggregates are combined to produce a workable mix which meets all shielding requirements.

Certified test reports are prepared by the supplier for each material shipment, attesting to aggregate conformance to cleanliness requirements when tested per ASTM C117, and specific gravity requirements when tested per ASTM C127 and ASTM C128.

Metal aggregate consists of commercial S-930 and S-780 chilled iron or steel shot. This shot conforms to the appropriate SAE gradation presented in table 1 of the SAE Handbook.

3.8.6.1.1.4 Mixing Water and Ice

Water and ice used in mixing concrete is free of injurious amounts of oil, acid, alkali, organic matter, or other deleterious substances, as determined by AASHTO T26. Such water and ice contains no injurious impurities that would cause either a change in the setting time of portland-cement of more than 25%, as determined in accordance with ASTM C266, or a reduction in compressive strength of mortar of more than 5%, compared with results obtained with distilled water, as determined in accordance with ASTM C109 (using 2 inch cube specimens).

3.8.6.1.1.5 Admixtures

Air entraining admixtures conform to ASTM C260. Water reducing and retarding admixtures conform to ASTM C494, Type A and/or D. Types A and D are used in accordance with the manufacturer's recommendations. Certificates stating conformance to the applicable ASTM specification are furnished with each shipment. The use of any other admixture is not permitted.

3.8.6.1.2 Concrete Mix Design

3.8.6.1.2.1 Concrete Properties

Concrete properties required for each type of mix design are verified by testing for the applicable properties indicated below:

Property	Test Designation
Compressive strength	ASTM C39
Unit weight	ASTM C138
Slump	ASTM C143
Air content	ASTM C231

Samples for property testing are taken in accordance with ASTM C172.

The following additional properties of selected mix designs have been determined to ascertain material compatibility with design assumptions:

Property	Test Designation
Modulus of elasticity	ASTM C469
Poisson's ratio	ASTM C469
Thermal diffusivity	CRD C36
Thermal coefficient of expansion	CRD C39

3.8.6.1.2.2 Concrete Mix Proportions

Concrete mix designs are prepared in accordance with paragraphs 4.2.2.2 and 4.2.3 of ACI 318-71. Trial batches are prepared in accordance with ACI 613-54.

Four compressive strength cylinders are cast at each time of sampling for all normal density concrete. Two of these cylinders are tested at 7 days. The two remaining cylinders are tested at 28 days for concrete with a design strength of 4000 psi or less and at 90 days for concrete with a design strength of 5000 psi. For high density concrete, three compressive strength cylinders are cast at each time of sampling; one is tested at 7 days, and the other two are tested at 28 days.

Concrete design strengths for the various structures are in accordance with the following table:

Item	Strength (psi)
Fuel pool girders, critically stressed walls in the reactor enclosure	5000
Reactor enclosure, primary containment, control structure, spray pond pump structures, diesel generator enclosure, radwaste enclosure, cooling towers, turbine enclosure foundation and walls below el 217'	4000
All other structural concrete	3000
Mass concrete fill	2000

The total air content of the concrete, except for grout, varies with the concrete mix and aggregate size as follows:

Concrete mixes with a 1 inch maximum aggregate size (except for D-1 (5000 psi) concrete mix) have a total air content of not less than 4%, nor more than 7% of the concrete volume.

D-1 (5000 psi) concrete mix with a 1 inch maximum aggregate size has a total air content of not less than 4%, nor more than 6% of the concrete volume for onsite concrete, 7% for offsite concrete.

Concrete mixes with a maximum aggregate size of 1½ inches have a total air content of not less than 3%, nor more than 6% of the concrete volume.

Testing for air content is in accordance with ASTM C231.

In lieu of establishing limits on the water-cement ratio, the concrete is proportioned, mixed, and placed at specified slumps. The average slump at the point of placement is less than the "working limit", which is the maximum slump for estimating the quantity of mixing water used in the concrete. An "inadvertency margin" is the allowable deviation from the "working limit" for those occasional batches which may inadvertently exceed the "working limit." Job-site tests indicate that concrete with slumps at the "inadvertency margin" produce acceptable quality concrete.

3.8.6.1.3 Grout

3.8.6.1.3.1 Construction Grout

Construction grout for use at horizontal construction joints and similar applications is proportioned from the same materials as for concrete. Such grouts have a sand/cement ratio comparable to the concrete with which it is to be used. Grout strength is determined in accordance with ASTM C109.

3.8.6.1.3.2 Starter Mix

Starter mixes are used in applications, such as at the bottom of foundation slabs, and in lieu of construction grout. They are defined as concrete with a 1 inch maximum size aggregate, with a slump between 3 inches and 5 inches.

3.8.6.1.3.3 Nonshrink Grout

Nonshrink grout is prepared from proprietary materials such as Embeco 636 by Master Builders Company, or Five Star Grout by US Grout Corporation. Such grouts are proportioned in accordance with the manufacturer's recommendations, and are tested prior to use for expansion, compressive strength, and flow characteristics, with maximum water content recommended by the manufacturer.

3.8.6.1.4 Batching, Placing, Curing, and Protection

3.8.6.1.4.1 Storage

Storage of aggregates, cement, and admixtures is in accordance with the recommendations of ACI 614-59.

3.8.6.1.4.2 Batching, Mixing, and Delivering

Concrete for principal structures is provided as central mixed concrete from an onsite batch plant, or as truck mixed concrete from an offsite plant after June 1983. Concrete blockout grout, however, may be batched by volume and provided from a mortar mixer. Batch plant facilities are certified by the National Ready Mix Concrete Association. Measuring devices are calibrated at required intervals, and more frequently where deemed appropriate.

The measuring of materials, batching, mixing, and delivery of all concrete conforms to ASTM C94, except as otherwise noted.

3.8.6.1.4.3 Placing

Placing of concrete is in accordance with the recommendations of ACI 301-66, ACI 306-66, ACI 318-71, ACI 605-59, and ACI 614-59.

3.8.6.1.4.4 Consolidation

Consolidation of concrete is in accordance with the recommendations of ACI 614-59, and as specified below.

Concrete is placed with the aid of mechanical vibrating equipment, and supplemental hand spading and tamping. The vibrating equipment is of the internal type, with a minimum frequency of 7000 revolutions per minute.

Surface vibrators, or manual jitterbugs are permitted to consolidate concrete in slabs 8 inches thick or less.

3.8.6.1.4.5 Curing

Curing of concrete is in accordance with the recommendations of ACI 301-66, ACI 306-66, ACI 318-71, and ACI 605-59.

3.8.6.1.4.6 Hot and Cold Weather Concreting

Methods and means of placing and curing concrete in cold and hot weather comply with the recommended practices of ACI 306-66 and ACI 605-59, respectively.

3.8.6.1.5 Construction Testing of Concrete and Concrete Materials

A concrete and concrete materials testing laboratory operated by Bechtel QC personnel is established at the project site to monitor the quality of such work and materials, and to promptly report any deviations from specified conditions. Such testing personnel are qualified to meet the guidelines of Regulatory Guide 1.58. Qualifications and procedures in use by Bechtel QC personnel conform with Regulatory Guide 1.94. During operation, if repairs to concrete or concrete materials are required, the Company and contractor personnel performing NDE are trained, tested, qualified, or certified in accordance with a company procedure that meets applicable requirements of 10 CFR 50.55a and ASME Section XI with specific exceptions and clarifications as discussed in the QATR.

Production testing for concrete and concrete materials is as shown in Table 3.8-12. Production testing for concrete conforms with Regulatory Guide 1.94, with the following clarification.

For central mixed concrete, samples for concrete cylinder strength and unit weight tests are taken at the point of discharge from the central mixing plant. The concrete temperature is taken at the point of discharge from the central mixer when obtaining compressive strength samples. In addition, for each load of concrete represented by compressive strength samples, the concrete temperature is taken at the point of discharge from the delivery trucks, or at the end of the pump line for pumped concrete. For truck mixed concrete furnished from an offsite source, samples for concrete cylinder strength and unit weight tests, together with temperature readings, are taken at the point of discharge from the delivery truck or at the end of the pump line for pumped concrete. Materials that do not meet test requirements are not used in the construction.

Concrete cylinder test results are reviewed for compliance with section 4.3.3 of ACI 318-71, and are evaluated in accordance with ACI 214-65. Materials or portions thereof that do not meet the above criteria, but may inadvertently be used are handled as described in the document "Limerick Generating Station Units 1 and 2; Summary Description of the Quality Assurance Program for Design and Construction," referenced in FSAR Section 17.1.

3.8.6.2 Concrete Reinforcement Materials

3.8.6.2.1 Qualification

Reinforcing steel for concrete structures conforms to ASTM A615, Grade 60, including section S1 for bar sizes 14 and 18. Certified copies of material test reports indicating chemical composition, physical properties, and dimensional compliance are furnished by the manufacturer for each heat. Spiral reinforcing steel is plain wire conforming to ASTM A82.

Prior to installation at the job-site, all reinforcing steel is subjected to a testing program meeting the guidelines of Regulatory Guide 1.15. Any reinforcing steel which does not meet these requirements is not used in the construction.

Sleeves for reinforcing steel mechanical splices conform to ASTM A519 for Grades 1018 and 1026. Certified copies of material test reports indicating chemical composition and physical properties are furnished by the manufacturer for each sleeve lot.

3.8.6.2.2 Fabrication

3.8.6.2.2.1 Bending of Reinforcement

Hooks and bends are fabricated in accordance with ACI 318-71, section 7.1. Bars partially embedded in concrete are bent subject to the following conditions:

a. Bending Partially Embedded Reinforcement

Bending of reinforcement partially embedded in concrete is performed in accordance with the following description. The diameter of the bend measured on the inside of the bar, and the distance between the beginning of the bend and the existing concrete surface is not less than "D", shown below:

Bar Size	"D"
Nos. 3 through 8	6 bar diameters
Nos. 9, 10, and 11	8 bar diameters
Nos. 14S and 18S	10 bar diameters

Bar Nos. 3 to 5 inclusive may be bent once, and straightened once, cold.

Bar Nos. 6 to 9 inclusive may be bent once, and subsequently straightened. Heating is required for both bending and straightening, as described below.

However, when the bending does not exceed 30°, bar Nos. 6 to 9 *inclusive may be bent once, and straightened once, cold.

When only cold bending is applied, the distance between the beginning of the bend and the existing concrete may be decreased to a minimum of 0.5 "D" as shown above.

When heat is used, it is applied as uniformly as possible over a length of bar equal to 10 bar diameters, and is centered at the middle of the arc of the completed bend. The maximum bar temperature is between 1100°F and 1200°F, and is maintained at that level until bending (or straightening) is complete.

Temperature-measuring crayons, or a contact pyrometer is used to determine the temperature. Heat is applied with care so as to avoid damage to the concrete. Care is taken to prevent rapid quenching of heated bars.

The bars, which are heated for straightening and bending, are visually inspected to determine whether they are cracked, reduced in cross-section, or otherwise damaged. Any damaged portions are removed and replaced.

3.8.6.2.2.2 Splicing of Reinforcement

Lap Splices

In general, lapped splices are used for No. 11 and smaller bars. Such lap splices are in accordance with sections 7.5, 7.6, and 7.7 of ACI 318-71.

Mechanical Splices

Cadweld splices conform with Regulatory Guide 1.10, except for the alternate approaches to tensile testing frequency, and the procedure for substandard tensile test results, discussed below.

Cadweld splices are used most frequently for Nos. 14 and 18 main bars in the category I concrete structures. Cadweld splices are used most frequently for Nos. 14 and 18 main bars in other Category I concrete structures. Most completed splices are visually inspected for the presence of slag, excessive porosity, or voids in the filler metal, and for proper alignment and centering of the reinforcing bar. However, there are some instances where longitudinal centering of the splice sleeve on the splice ends is off center, and probing for voids in the filler metal cannot be performed due to interferences. In these instances, a test program simulating these conditions is performed. The test results conform with the tensile strength guidelines of Regulatory Guide 1.10.

For purposes of quality control, splices representing the work of each splicing crew are tensile tested for each position, bar size, and series (T or B). Bar-to-bar splices are designated as T-series splices. Bar-to-embedment splices are designated as B-series splices.

A crew is defined as the unique combination of persons preparing, assembling, and igniting the splice. Each unique combination of persons on a crew is considered as a separate crew. In the event that one or more members of the crew is unavailable for completing a production splice lot, or performing a sister splice, the remaining member or members of the crew complete the production splice lot, or perform the sister splice. For purposes of the tensile testing program, the remaining member or members are considered as comprising the crew.

A T-series "sister splice" consists of two 14 inch (+1 inch) bars spliced in sequence with, at the same location, under the same condition, and in an otherwise identical manner as the corresponding production splice. Straight sister splice samples are taken in lieu of production splice test samples for curved reinforcing bars, and for the splices in locations outside the primary containment where space limitations prohibit taking production splice test samples.

A B-series "sister splice" is made in the configuration shown on the design drawings, and is spliced in sequence with, at the same location, under the same condition, and in an otherwise identical manner as the corresponding production splice.

The number and frequency of Cadweld splice tensile tests performed by each crew for each position, bar size, and series are as follows:

a. One sister splice is tested, prior to the start of any production splicing. Qualification splices meet this requirement if performed by each member of the crew within the previous three calendar days, and the crew has made no production splices with visual defects or which fail tensile testing since making the qualification splices.

   Deviation from this criteria was allowed in some cases for the Cadweld splice installation. This deviating condition was documented in nonconformance report NCR#14244. The Cadweld splices installed without prior sister splice testing are acceptable per engineering review documented in the nonconformance report. The omission of sister splice testing prior to production splicing does not affect the performance safety-related structures because LGS meets the intent of Regulatory Guide 1.10. For the Cadweld installation without prior sister splice testing, the test frequency as stated in paragraph (b) below was implemented and completed satisfactorily. Therefore, the satisfactory completion of the testing frequency of paragraph (b) in conjunction with the crew qualification requirements, and the visual inspection requirement for production splices, combined to assure that the level of safety required for mechanical splices discussed in Regulatory Guide 1.10 is provided.

b. One test sample out of each lot of 25 consecutive production splices is tested, including the first group of 25.

   For T-series splices, each consecutive group of three such samples consists of one production splice and two sister splices. All test samples are sister splices for the B-series splices, and for splices of No. 18 bars to square Cadweld bars of the flange skirt assembly.

   Production splice test samples are selected at random by the inspector from any within the lot of 25 splices represented. Sister splice samples are made upon completion of the lot of 25 production splices. Horizontal sister splices are side or top filled to conform to the fill location of the majority of the splices.

   If fewer than a complete lot of 25 splices are made prior to the date of a concrete placement containing any of the splices in the incomplete lot, the test splices for that lot are made prior to the placement to determine the acceptability of the splices in that lot, and the lot is then treated as a complete lot.

Welded Splices

Whenever both lap and mechanical splices have been determined to be impractical, welded splices are used on a case-by-case approval basis. Such welding is performed by qualified welders using a procedure conforming to the basic recommendations of AWS D12.1.

3.8.6.2.2.3 Placing of Reinforcement

Reinforcement is securely tied with wire, and held in position by spacers, chairs, and other supports to maintain placement accuracy within the tolerances established for reinforcement protection, and the design requirements.

3.8.6.2.2.4 Spacing of Reinforcement

Spacing of reinforcement is in accordance with ACI 318-71.

3.8.6.2.2.5 Surface Condition of Reinforcement

Reinforcement surface conditions at the time of concrete placement are in compliance with section 7.2 of ACI 318-71.

3.8.6.2.3 Construction Testing of Concrete Reinforcement Materials

Inspection of reinforcement materials to ensure that bending, placing, splicing, spacing, and surface condition requirements are met is in accordance with Regulatory Guide 1.94, except as described in Section 3.8.6.2.2.2.

3.8.6.2.4 Formwork and Construction Joints

Formwork is constructed in accordance with the applicable provisions of ACI 347-68, so that the finished concrete surfaces do not exceed the tolerances of ACI 347-68.

Construction joints are made in accordance with ACI 301-66, ACI 318-71, and ACI 614-59. In addition, all horizontal construction joints are covered, immediately before the concrete is placed, with a nominal ½ inch of flowable grout mix, or a nominal ¾ inch of starter mix.

Concrete is placed in accordance with Regulatory Guide 1.55, except as discussed below.

Regulatory positions 2 and 3 of the Regulatory Guide state the presumed functional responsibilities of the "designer" and the "constructor." Under the designer's role, are listed the responsibilities for checking shop drawings and locations of construction joints. On this project, the former responsibility is fully delegated to the Bechtel field, although the design engineering office may check significant portions, and may advise the field accordingly. The responsibility for construction joint location is partly delegated to the field, in the sense that the field has to follow the guidelines set out in the design drawings and specifications prepared by the design engineering office.
Also, Regulatory Guide 1.55 references ACI 301-72 and ACI 305-72. However, concrete at LGS is placed in accordance with ACI 301-66 and ACI 605-59 respectively.

3.8.6.3 Prestressed Concrete

3.8.6.3.1 Post-Tensioning System for Prestressed Concrete

The post-tensioning system used for prestressed concrete is the VSL E5-55 post-tensioning system, described in the 1976 Edition of the Post-Tensioning Manual, published by the Post-Tensioning Institute.

3.8.6.3.2 Concrete for Prestressed Members

The concrete used for prestressed members is a 5000 psi concrete, designed and tested in accordance with Section 3.8.6.1.

3.8.6.3.3 Prestressing Steel

Prestressing steel consists of 7 wire, stress-relieved prestressing strands meeting the requirements of ASTM A416, and having a guaranteed minimum ultimate tensile strength of 270 ksi. Stress relaxation tests in accordance with ASTM E328 are also performed. The tests are performed for each 50 tons of a heat or fraction thereof. Any prestressing steel which does not meet the test requirements is not used in the construction.

3.8.6.3.4 Sheathing

Sheathing is spiral-wrapped, semirigid corrugated tubing, fabricated from a galvanized sheet, 24 gauge minimum split strip of lock-forming quality. The sheathing meets the requirements of ASTM A527, Coating Designation G90. It is not chemically treated, or oiled.

3.8.6.3.5 Anchorage Assembly

The anchorage assembly is the system of components consisting of the anchor head, wedge, bearing plate, and trumpet. The anchor head, wedge, and bearing plate materials meet the requirements of AISI 1026, AISI 86L20, and ASTM A537, respectively. After fabrication, bearing plates are galvanized in accordance with ASTM A123. Trumpets are fabricated from galvanized sheet meeting the requirements of ASTM A569.

3.8.6.3.6 Grout

The prestressing steel is bonded to the concrete by completely filling the entire void space between the sheathing and the tendon with grout. The grout consists of a mixture of Portland-cement, water, and admixture. The cement and water used to make the grout meet the testing requirements stipulated in Section 3.8.6.1. The grout is designed to have a minimum 7 day compressive strength of 2500 psi, in accordance with ASTM C109, and a minimum 28 day compressive strength of 3500 psi, in accordance with U.S. Army Corps of Engineers CRD-C589. Three samples for each test are taken each day grouting is performed for every 5 cubic yards, or fraction thereof produced.

3.8.6.3.7 Post-Tensioning System Performance Tests

Prior to actual post-tensioning operations, the post-tensioning system is subjected to static and dynamic tests in accordance with the recommendations of section 3.1.8 of the 1976 Edition of the Post-Tensioning Manual, published by the Post-Tensioning Institute.

3.8.6.4 Concrete Unit Masonry and Masonry Material

3.8.6.4.1 Material Qualification

3.8.6.4.1.1 Concrete Unit Masonry and Building Brick

Concrete unit masonry conforms to either ASTM C90, Type I, Grade N for hollow masonry units, or ASTM C145, Type I, Grade N for solid masonry units. Concrete building brick conforms to ASTM C55, Type I, Grade N. Certified test reports are prepared for each material.

3.8.6.4.1.2 Masonry Mortar

Masonry mortar conforms to ASTM C476, Type PL.

3.8.6.4.1.3 Masonry Grout

The three types of masonry grout described below have been used for filling cavities in masonry.

a. Chemtree Grout

Chemtree grout is a premixed grout manufactured by Chemtree Corporation. It has a minimum plastic unit weight of 206 lb/ft^3, and a minimum compressive strength of 1500 psi. Tests for unit weight and compressive strength are in accordance with Section 3.8.6.1.2.1.

Chemtree grout is used to fill the cavity and cells of a small number of multiple wythe walls in the radwaste enclosure and in the auxiliary bay portion of the turbine enclosure.

b. 2000 psi Masonry Grout

The second masonry grout used for filling cavities in masonry is a 2000 psi grout designed and tested in accordance with Section 3.8.6.1, as modified below:

1. Aggregate

Coarse and fine aggregate are combined as a single aggregate, graded to produce a workable mix that meets all shielding requirements.

2. User Tests

The following user tests are performed every 6 months during production:

Test	Designation
Potential Reactivity of Aggregates	ASTM C289
Specific Gravity and Absorption of Fine Aggregate	ASTM C128

Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate	ASTM C88
Materials Finer than No. 200 Sieve in Mineral Aggregates by Washing	ASTM C117
Organic Impurities in Sand for Concrete	ASTM C40

3. Compressive Strength, Unit Weight, and Slump

Three compressive strength cylinders are cast. One cylinder is tested at 7 days, and two are tested at 28 days. The minimum compressive strength, determined at 28 days, is 2000 psi.

The minimum plastic unit weight for the masonry grout is 152 lb/ft^3.

The masonry grout is allowed a slump of 9 inches ±1 inch.

c. 2000 psi Masonry Grout Using High Density Aggregates

The third masonry grout used for filling cavities is a 2000 psi grout using high density and normal weight aggregates, designed and tested in accordance with Section 3.8.6.1, as modified below:

1. Aggregate

Normal weight and high density fine aggregate are used to produce a workable mix that meets all shielding requirements.

2. User tests

The user tests for normal weight fine aggregate are defined in Section 3.8.6.1.1.2. The user tests for high density fine aggregate are defined in Section 3.8.6.1.1.3

3. Compressive Strength, Unit Weight, and Slump

These requirements are the same as those discussed in Section 3.8.6.4.1.3.b.3

3.8.6.4.1.4 <u>Concrete Infill</u>

Concrete infill is used as an alternative to masonry grout for filling cavities in masonry. Concrete infill conforms to the requirements described in Section 3.8.6.1.

3.8.6.4.1.5 <u>Steel Reinforcement</u>

Steel reinforcement conforms to the requirements described in Section 3.8.6.2.

3.8.6.4.2 Construction Testing

Testing to monitor the quality of masonry work and masonry materials is in accordance with the program described in the document "Limerick Generating Station Units 1 and 2; Summary Description of the Quality Assurance Program for Design and Construction," referenced in FSAR Section 17.1.

Test frequency during the construction of concrete unit masonry is as shown in Table 3.8-12.

Materials that are inadvertently used in the construction, but do not meet test requirements, are handled as described in the document "Limerick Generating Station Units 1 and 2; Summary Description of the Quality Assurance Program for Design and Construction," referenced in FSAR Section 17.1.

3.8.7 COMPUTER PROGRAMS FOR STRUCTURAL ANALYSIS

This section describes the computer programs used for the structural analysis of all seismic Category I structures. Each program description includes a statement of the program's area of application, and a discussion of the modifications and assumptions made. In addition, the descriptions of the computer programs not in the public domain include a selection of the sample problems used to verify their solution accuracy.

3.8.7.1 3D/SAP (Finite-Element Analysis of Three-Dimensional Elastic Solids)

3.8.7.1.1 Application

Finite-element structural analysis program 3D/SAP is capable of performing static analyses of three-dimensional solid structures subjected to concentrated or distributed loadings, thermal expansion, and/or arbitrarily directed static body forces. It is a modern version of SAP (Reference 3.8-4) written as a general purpose structural analysis computer program.

3.8.7.1.2 Program Background

Program 3D/SAP was developed by the Control Data Corporation, and is in the public domain.

3.8.7.2 ASHSD (Axisymmetric Shell and Solid)

3.8.7.2.1 Application

ASHSD is a special purpose program which can be used in the elastic, static, or dynamic analysis of structural systems capable of being represented as axisymmetric shells and/or solids.

This program allows a useful study of the interaction between a typical nuclear containment structure, modeled as an axisymmetric shell, and the subsoil, modeled as an axisymmetric solid.

3.8.7.2.2 Program Background

This program is a refinement of the original ASHSD code developed at the University of California at Berkeley. The present program is highly modified for the special purpose of static and dynamic analysis of nuclear containment structures. The modified program has the following features:

a. The code has a shell finite-element which uses an interaction stiffness, allowing analysis of layered shells.

b. Since shell layers may be bonded or unbonded from each other, it is possible to describe concrete shells in their actual geometric form. For example, it is possible to describe liner plate, concrete, reinforcing steel, and post-tensioning steel in their real spatial locations.

c. Post-tension forces may be applied to the shell by subjecting only the unbonded post-tensioning elements to a pseudothermal loading.

d. Isotropic or orthotropic elastic constants are possible for both shell and solid elements. The orthotropic material properties may be used to describe the different stiffness of reinforcing steel in the hoop and meridional directions, for example.

e. Nonuniform thermal gradients through the wall thickness may be imposed.

f. Eigenvalues and eigenvectors may be computed by the program.

g. Three dynamic response routines are available in the program. They are:

    1. Arbitrary dynamic loading, or earthquake-based excitation using an uncoupled (modal) technique.

    2. Arbitrary dynamic loading, or earthquake-based excitation using a coupled (direct integration) technique.

    3. Response spectrum modal analysis for ABS and SRSS displacements and element stresses.

h. The coupled time history solution has the capability to allow an arbitrary damping matrix.

i. The stiffness and mass matrices may be obtained as punched output for input into other programs.

The version of this program currently used by Bechtel is maintained by the Control Data Corporation.

3.8.7.2.3 <u>Sample Problems</u>

This program is verified by comparing the computer results with hand-calculated solutions and published references. Three sample problems are presented as examples of verification.

a. Sample Problem A: Closed Cylinder Under Internal Pressure

This problem demonstrates the membrane state of stress in a closed cylinder subjected to a uniformly distributed internal pressure. Hand calculations are used to verify this aspect of the program.

The selected problem is a cylinder with closed ends subjected to internal pressure. Only one-half of the cylinder is required in the model because of symmetry. Furthermore, it is assumed that the closed ends are distant from the section being analyzed, and they are therefore excluded.

Two models of the cylinder are actually analyzed. One model uses the thin-shell elements, and the other uses the axisymmetric solid elements. These models are shown in Figures 3.8-65 and 3.8-66 with their key dimensions.

The problem parameters for both test cases are as follows:

<u>Boundary Conditions</u>

Node 1: Z displacement = 0
θ displacement = 0
Rotation in R-Z plane = 0
(free to move radially)

Node 16: θ displacement = 0
(free to move axially, radially, and to rotate about the θ axis)

<u>Numerical Data</u>

Material: concrete

Modulus of elasticity (E) = 4.031×10^6 psi

Thickness (t) = 36 inches

Radius (R) = 900 inches

Poisson's ratio (ν) = 0.17

Pressure (p) = 60 psi

Length (L) = 1800 inches

N = 27,000 lb/in (an equivalent node load applied at Node 16)

The theoretical values for the membrane force resultants are calculated to be pR/2 (= 27,000 lb/in) axial force, and pR (= 54,000 lb/in) for the circumferential force (hoop stress).

The results obtained from the ASHSD program are presented in Table 3.8-13, both for the thin-shell and the layered shell models. Analytical computations indicate maximum errors at Node 16 of 0.4% for the longitudinal force and 3.2% for the circumferential force.

b. Sample Problem B: Cylindrical Shell Subjected to Internal Pressure and Uniform Temperature Rise

This test example demonstrates the use of a combined static load and thermal load condition. A short circular cylindrical shell clamped at both ends is subjected to an internal pressure and a uniform temperature rise.

The theoretical solutions given in Reference 3.8-4 are used to verify this analysis.

The general arrangement of the cylinder is shown in Figure 3.8-67. Only one-half of the cylinder is used for the finite-element model because of symmetry. This is shown in Figure 3.8-68, with Node 1 located at the middle of the cylinder.

The problem parameters are as follows:

Boundary Conditions

At center of cylinder, Node 1:	Z displacement = 0 θ displacement = 0 Rotation in the R-Z plane = 0
At end of cylinder, Node 26:	R displacement = 0 Z displacement = 0 θ displacement = 0 (tangential) Rotation in the R-Z plane = 0

Numerical Data

Material: concrete

Modulus of elasticity (E) = 4.031×10^6 psi

Poisson's ratio (υ) = 0.17

Thermal coefficient of expansion (α) = 55×10^{-7} in/in/°F

Thickness (t) = 30 inches

Radius (R) = 600 inches

Length (L) = 1200 inches

Height (Z) = 600 inches

Pressure (p) = 60 psi

Temperature (T) = 150°F

$R/t = 20$

$L/R = 2$

The theoretical results are shown in Figure 3.8-69. These values are obtained by using the following equations from Reference 3.8-4:

Axial Moment: $M_x = 2\mu^2 D_x \left(\frac{PR^2}{Et} + R\alpha T\right)$ (EQ. 3.8-1)

where:

$$\mu^2 = \left[\frac{3(1-\upsilon^2)}{R^2t^2}\right]$$

$$D_x = \frac{Et^3}{12(1-\upsilon^2)}$$

Normalized length: Ln = (Z/R)(L/2R)

Figure 3.8-69 compares the results obtained from the ASHSD program and the theoretical solution. The results of ASHSD agree well with those of the reference.

c. Sample Problem C: Asymmetric Bending of a Cylindrical Shell

This sample problem illustrates the use of higher harmonics for asymmetric loading cases. As a comparison to the computer output, results for this problem are taken from Reference 3.8-5.

The cylindrical shell analyzed is a short, wide cylinder as shown in Figure 3.8-70. The finite-element idealization of the cylinder and the pertinent data are illustrated in Figure 3.8-71. At each end of the cylinder, moments of the form $M = M_o \cos\eta\theta$ were input for harmonics η = 0, 2, 5, 20.

The problem parameters are:

Material: steel

Modulus of elasticity (E) = 29×10^6 psi

Thickness (t) = 1.25 inches

Radius (R) = 60.0 inches

Poisson's ratio (υ) = 0.3

Length (L) = 60.0 inches

$L/R = 1$

$R/t = 48$

$$M_o = \frac{Et^2}{100(1-\upsilon^2)} = 497939.56 \text{ in-lb/in} \quad \text{(EQ. 3.8-2)}$$

The comparison results are taken directly from Reference 3.8-5. Those results are plotted in Figure 3.8-72.

The comparison of the computer results to the reference results are shown in Figure 3.8-72. (Note that the longitudinal moments and radial displacements are expressed as nondimensional ratios).

The reference and computer program results show good agreement. This verifies the accuracy of the program for this type of analysis.

3.8.7.3 CECAP (Concrete Element Cracking Analysis Program)

3.8.7.3.1 Application

CECAP computes stresses in a concrete element under thermal and/or nonthermal (real) loads, considering effects of concrete cracking. The element represents a section of a concrete shell or slab, and may include two layers of reinforcing, transverse reinforcing, prestressing tendons, and a liner plate.

The program outputs stresses and strains at selected locations in the concrete, reinforcement, tendons, and liner plate, and resultant forces and moments for the composite concrete element.

3.8.7.3.2 Program Background

CECAP assumes linear stress-strain relationships for steel and for concrete in compression. Concrete is assumed to have no tensile strength. The solution is an iterative process, whereby tensile stresses found initially in concrete are relieved (by cracking) and redistributed in the element. The equilibrium of nonthermal loads is preserved. For thermal effects, the element is assumed free to expand inplane, but is fixed against rotation. The capability for expansion and cracking generally results in a reduction in thermal-stresses from the initial condition.

The version of this program currently used by Bechtel is maintained by the Control Data Corporation.

3.8.7.3.3 Sample Problems

Sample problems are analyzed by CECAP and compared with hand-calculated solutions. These sample problems consider a reinforced concrete beam as shown in Figure 3.8-73. The parameters for all sample problems are as follows:

Modulus of elasticity of concrete (E_c) = $3x10^6$ psi

Modulus of elasticity of reinforcing steel (E_s) = $30x10^6$ psi

Poisson's ratio for concrete (υ_c) = 0.22

Coefficient of thermal expansion of concrete
(α_c) = $6x10^{-6}$ in/in/°F

Temperature difference (ΔT) = 100°F

Coefficient of thermal expansion of reinforcing steel
(α_r) = α_c

Three sample problems are presented as examples of verification.

a. Sample Problem A: Beam with a Thermal Moment

The analysis of a reinforced concrete beam subjected to a linear thermal gradient is performed to test the redistribution of thermal-stresses due to the relieving effect of concrete cracking.

Figure 3.8-74 shows the reinforced concrete beam, and the corresponding CECAP concrete element used in the analysis. Boundary conditions, geometry, and applied loads are illustrated.

The following illustrates how thermal loads are treated in a cracked section analysis of a reinforced concrete beam. The main assumptions pertaining to thermal boundary conditions are:

1. The beam is allowed to freely expand axially.

2. There is no rotation of the initial thermal-stress slope.

The beam cross-section and initial thermal-stress distribution are shown in Figure 3.8-75. For ΔT = 100°F, the equivalent thermal moment and thermal-stresses in concrete and steel are:

Thermal moment (M) = $\Delta T \alpha_c E_c b t^2/12$ (EQ. 3.8-3)
= 3,175,000 in-lbs

Concrete stress (σ_c) = $\Delta T \alpha_c E_c/2$ (EQ. 3.8-4)
= 900 psi (compression)

Rebar stress (σ'_c) = $\dfrac{(t/2-2)}{t/2}\sigma_c$ (EQ. 3.8-5)
= 814 psi (tension)

where:

b = width of beam = 12 inches

t = depth of beam = 42 inches

E_c = modulus of elasticity = 3×10^6 psi

α_c = coefficient of linear expansion for concrete
= 6×10^{-6} in/in/°F

The stress diagram used for the cracked section analysis with thermal loading is shown in Figure 3.8-76. The assumptions of free movement axially and constant thermal-stress slope are maintained by a lateral translation of the initial reference axis to a final cracked position.

From force equilibrium:

$$F_{rebar} + F_{concrete} = 0 \qquad \text{(EQ. 3.8-6)}$$

where:

$F_{rebar} = f_s$ = total stress in steel

$= 1.0\,(814 + \Delta\sigma_c)\,10$ psi

$F_{concrete} = f_c$ = total stress in concrete

$$= -\,900\left(\frac{42}{2}\right)\left(\frac{12}{2}\right) + \frac{\Delta\sigma_c\,(12)}{2}\left[21 + \left(\frac{900-\Delta\sigma_c}{900}\right)2\right]$$

Solving for $\Delta\sigma_c$:

$\Delta\sigma_c = 582$ psi

From above, the rebar and concrete stresses are:

$f_s = (814+582)10 = 13{,}970$ psi (tension)

$f_c = 900-582 = 318$ psi (compression)

The location of the cracked neutral axis is:

$$kd = x = \frac{900-582}{900}\,(21) = 7.42 \text{ in} \qquad \text{(EQ. 3.8-7)}$$

The self-relieved thermal moment is:

$$M_T = \frac{f_s A_s (d - x/3)}{12} = 43{,}690 \text{ in-lb} \qquad \text{(EQ. 3.8-8)}$$

The rebar and concrete stresses, self-relieved thermal moment, and neutral axis location obtained from the CECAP program are compared with the hand calculations in Table 3.8-14. It can be seen that the CECAP results compare favorably with the hand calculation.

b. Sample Problem B: Beam with a Real Moment

The analysis of a reinforced concrete beam subjected to a real moment tests the CECAP program for nonthermal moments.

Figure 3.8-77 shows the loading and geometry for the reinforced concrete beam, and the corresponding CECAP concrete element model.

The following illustrates the working stress analysis of reinforced concrete beams. The beam cross-section, stress block, and transformed sections are shown in Figure 3.8-78. The resultant forces and moment are:

$$C = f_c\,(kd)(b)/2 \qquad \text{(EQ. 3.8-9)}$$

$$T = A_s(f_s) \qquad \text{(EQ. 3.8-10)}$$

$$M = Cjd = Tjd \qquad \text{(EQ. 3.8-11)}$$

Equating the first moments of the compression and tension areas about the neutral axis of the transformed section,

$$kd(b)\left(\frac{kd}{2}\right) = nA_s\,(d - kd) \qquad \text{(EQ. 3.8-12)}$$

where:

$$n = E_s/E_c = 10.0$$

which yields:

$$(kd)^2 + 1.67kd - 66.67 = 0$$

Solving for kd:

$$kd = 7.37 \text{ in}$$

For an applied moment (m) of 3,175,000 in-lbs, the resultant forces are:

$$C = T = \frac{M}{jd} = \frac{3{,}175{,}000}{\left(40 - \frac{7.37}{3}\right)} = 84{,}570 \text{ lb} \qquad \text{(EQ. 3.8-13)}$$

Rebar and concrete stresses, respectively, are:

$$f_s = \frac{T}{A_s} = 84{,}570 \text{ psi (tension)} \qquad \text{(EQ. 3.8-14)}$$

$$f_c = \frac{2C}{kd(b)} = \frac{2(84{,}570)}{(7.37)(12)} \qquad \text{(EQ. 3.8-15)}$$

$$= 1193 \text{ psi (compression)}$$

Table 3.8-14 shows a comparison of rebar and concrete stresses, and neutral axis locations obtained from the CECAP program and hand calculations. The CECAP results are shown to compare to hand calculations within the force accuracy limits in the program.

c. Sample Problem C: Beam with a Real Moment and a Real Axial Load

This sample problem involves the analysis of a reinforced concrete beam subjected to both a real moment, and a real axial compressive load.

The loading and geometry for the reinforced concrete beam and corresponding CECAP model are illustrated in Figure 3.8-79.

The following illustrates the working stress analysis of reinforced concrete beams subjected to both moments and axial compressive loads. The beam cross-section and stress block are shown in Figure 3.8-80. The analysis uses the equations presented in Reference 3.8-7, simplified as follows:

$$(kd)^3 + 3\left[\frac{M}{N} - \frac{t}{2}\right](kd)^2 + \frac{6nA_s}{b}\left[d - \frac{t}{2} + \frac{M}{N}\right](kd) - \frac{6nA_s d}{b}\left[d - \frac{t}{2} + \frac{M}{N}\right] = 0 \qquad \text{(EQ. 3.8-16)}$$

$$f = \frac{N}{A_s}\left[\frac{M}{N} + \frac{kd}{3} - \frac{t}{2}\right] \Big/ \left[d - \frac{kd}{3}\right] \qquad \text{(EQ. 3.8-17)}$$

$$f = \frac{f_s kd}{n(d - kd)} \quad \text{for } \frac{M}{N} \geq t/6 \qquad \text{(EQ. 3.8-18)}$$

where:

M = moment = 375,000 in-lbs

N = axial load = 101,000 lbs

t = depth of beam = 42 inches

b = width of beam = 12 inches

Equation 3.8-16 becomes:

$kd^3 + 55.8kd^2 + 293kd - 11720 = 0$

$M/N = \frac{3{,}175{,}000}{101{,}000} = 31.4 \geq t/6 = \frac{42}{6} = 7$

Solving the above by iteration for (kd) yields:

kd = 12.7 in

The resulting rebar and steel stresses are:

f_s = 41,320 psi (tension)

f_c = 1922 psi (compression)

The rebar and concrete stresses, and neutral axis location obtained from the CECAP program are compared with the hand calculations in Table 3.8-14. The results for the two solution methods agree very closely.

3.8.7.4 CE668 (Plate Bending Analysis)

3.8.7.4.1 Application

This program performs the linear-elastic analysis of a plate with arbitrary shape and support, stiffener beams, and elastic subgrade, under loads normal to the middle plane of the plate.

3.8.7.4.2 Sample Problems

Sample problems were analyzed by CE668, and compared with hand-calculated solutions.

a. Sample Problem A: Rectangular Plate with a Concentrated Load at the Center

The simply supported rectangular plate, shown in Figure 3.8-81, is subjected to a concentrated load of 300 pounds at its center. Only one-half of the plate is modeled by finite-elements, because of symmetry. The boundary conditions are zero-displacement with free normal rotation at the simply supported edges, and free displacement with zero normal rotation at the symmetry axis. The plate has isotropic structural properties.

The problem parameters are:

Poisson's ratio (υ) = 0.3

Modulus of elasticity (E) = $29x10^6$ psi

Plate thickness (h) = 0.5 in

Concentrated load (P) = 300 lb

Width of plate (a) = 10 inches

Length of plate (b) = 40 inches

The formulas for the deflections and moments are taken from Reference 3.8-8.

Deflection (at Center)

$$\omega = 0.01695 \frac{Pa^2}{D} \qquad \text{(EQ. 3.8-19)}$$

where:

$$D = \frac{Eh^3}{12(1-\upsilon^2)}$$

ω = 0.00153 in at Node 116

Moments

M_x: (for b >> a) (at x = 2, y = 0)

$$M_x = \frac{-P(1+\upsilon)}{8\pi} \ln \left[\frac{1 - \sin \frac{\pi x}{a}}{1 + \sin \frac{\pi x}{a}} \right] \quad \text{(EQ. 3.8-20)}$$

M_x = 20.92 in-lb at Node 113

M_y: (for b >> a) (at x = 6, y = 0)

$$M_y = -P\frac{(1+\upsilon)}{8\pi} \ln \left[\frac{1 - \sin \frac{\pi x}{a}}{1 + \sin \frac{\pi x}{a}} \right] \quad \text{(EQ. 3.8-21)}$$

M_y = 57.198 in-lb at Node 117

The hand-calculated values for deflections and moments are compared with the CE668 values in Table 3.8-15. The results for the two solution methods are in very close agreement, the largest difference being 1.55%.

b. Sample Problem B: Uniform Load on a Rectangular Plate with Various Edge Conditions

For this problem, the rectangular plate has one edge fixed, one edge free, and two edges simply supported, as shown in Figure 3.8-82. The plate is subjected to a uniformly distributed load (q) equal to 2.0 psi. Only one-half of the plate is modeled by finite-elements, because of symmetry. Boundary conditions are specified in accordance with the edge support conditions.

The problem parameters are as follows:

Poisson's ratio (υ) = 0.3

Modulus of elasticity (E) = 29×10^6 psi

Plate thickness (h) = 0.2 in

Uniformly distributed load (q) = 2.0 psi

Plate width (b) = 15 inches

Plate length (a) = 30 inches

The formulas for the deflections and moments were taken from Reference 3.8-8.

Deflection (at x = 15, y = 15)

$$\omega = 0.0582 \frac{qb^4}{D} \qquad \text{(EQ. 3.8-22)}$$

where:

$$D = \frac{Eh^3}{12(1-\upsilon^2)}$$

ω = 0.277 in at Node 11

Moments

M_x: (at x = 15, y = 15)

M_x = $0.0293\ qa^2$

= 52.74 in-lb at Node 11 (EQ. 3.8-23)

M_y: (at x = 15, y = 0)

M_y = $0.319\ qb^2$

= 143.55 in-lb at Node 121 (EQ. 3.8-24)

The hand-calculated values for deflections and moments are compared with the CE668 values in Table 3.8-15. The results for the two solution methods are in close agreement, the largest difference being 3.4%.

3.8.7.5 EASE (Elastic Analysis for Structural Engineering)

3.8.7.5.1 Application

EASE is a finite-element program which performs static analyses of two-dimensional and three-dimensional trusses and frames, plane elastic bodies, and plate and shell structures. The program library contains various types of elements, including truss, beam, plane stress and strain, plate bending, and shell elements. The EASE program accepts thermal loads, as well as pressure, gravity, and concentrated loads.

The program output includes joint displacements, beam forces and element stresses, and forces and moments.

3.8.7.5.2 Program Background

EASE was developed by the Engineering/Analysis Corporation, Redondo Beach, California, in 1969, and is in the public domain. The version currently used by Bechtel is maintained by the Control Data Corporation, Cybernet Service.

3.8.7.6 E0119

3.8.7.6.1 Application

This program analyzes a bolted flange. The program calculates allowable and actual bolt stresses, and longitudinal, radial, and tangential flange stresses.

3.8.7.6.2 Program Background

The symbols, terms, and analysis procedures used in this program are in accordance with ASME Section III, Appendix XI.

The version of this program currently used by Bechtel is maintained by the Control Data Corporation.

3.8.7.6.3 Sample Problems

Sample problems were analyzed by E0119 and compared with the solutions published in Reference 3.8-9.

The problem parameters for the two sample problems are as follows:

Design pressure = 400 psi

Design temperature = 500°F

Atmospheric temperature = 75°F

Poisson's ratio = 0.30

Corrosion allowance = 0

Gasket width = 0.75 inches

Effective gasket width = 0.306 inches

Gasket factor = 2.75

Gasket seating strength = 3700 psi

a. Sample Problem A: Welding Neck Flange

   Figure 3.8-83 details the dimensions of the welding neck flange. Table 3.8-16 compares the results obtained using E0119 with the results published in Reference 3.8-9.

b. Sample Problem B: Slip-on Flange

Figure 3.8-84 details the dimensions of the slip-on flange. Table 3.8-16 compares the results obtained using E0119 with the results published in Reference 3.8-9.

In both problems, the results obtained using E0119 compare favorably with the published results.

3.8.7.7 E0781 (Shells of Revolution Program)

3.8.7.7.1 Application

This program calculates the stresses and displacements in thin-walled elastic shells of revolution when subjected to static edge, surface, and/or temperature loads with arbitrary distribution over the surface of the shell. The geometry of the shell must be symmetric, but the shape of the median is arbitrary. It is possible to include up to three branch shells with the main shell in a single model. In addition, the shell wall may consist of different orthotropic materials, and the thickness of each layer and the elastic properties of each layer may vary along with the median.

3.8.7.7.2 Program Background

The Shells of Revolution Program is based on the analysis described in Reference 3.8-10.

The program numerically integrates the eight ordinary first-order differential equations of thin-shell theory. The equations are derived so that the eight variables chosen are those appearing on the boundaries of the axially symmetric shell; thus the entire problem is expressed in these fundamental variables.

The program has been altered so that a 4x4 force-displacement relation is used as a boundary condition, as an alternative to the usual procedure of specifying forces or displacements. This force-displacement relation describes the forces at the boundary in terms of displacements at the boundary, or the displacements at the boundary in terms of forces, or some compatible combination of the two. In this manner, it is possible to study the behavior of a large complex structure. It is also possible to introduce a "Spring Matrix" at the end of any part of the stress model. This matrix must be expressed in the form, Force = Spring Matrix · Displacement. In addition to the above changes, the program has been modified to increase the size of the problem that can be considered and to improve the accuracy of the solution.

3.8.7.7.3 Sample Problems

This program is verified by comparing the computer results with experimental measurements and published references. Two sample problems are presented as examples of verification.

a. Sample Problem A: Comparison of 2:1 Ellipsoidal and Torispherical Heads Subjected to an Internal Pressure Load

   This problem illustrates the program's ability to generate cylindrical, torispherical, and ellipsoidal shapes. The E0781 program's "solution" is compared to the experimental investigation of 2:1 ellipsoidal heads subjected to internal pressure discussed in Reference 3.8-11.

   The problem compares a 2:1 ellipsoidal head, to an equivalent torispherical head subjected to the same uniformly distributed internal pressure. An equivalent torisphere is defined as one having the same height above the tangent line as the ellipsoid, and a minimal L/b ratio (thus having the least possible discontinuity between the torus and the sphere). For the geometry shown in Figure 3.8-85:

$$(L\text{-}b)\sin\phi_o = A\text{-}r \qquad \text{(EQ. 3.8-25)}$$

$$(L\text{-}b)\cos\phi_o = L\text{-}B \qquad \text{(EQ. 3.8-26)}$$

Minimizing L/b using equations 3.8-25 and 3.8-26:

$$\tan\phi_o = B/A = 0.5019 \qquad \text{(EQ. 3.8-27)}$$

$$\phi_o = 26.653^\circ$$

$$L/A = \frac{c \pm (c^2 - 2c)^{½}}{2} \qquad \text{(EQ. 3.8-28)}$$

where:

$c = B/A + A/B = 2.494$

$L = \frac{18.19}{2}[2.5 + (6.22 - 4.99)^{½} = 32.778$ in

$b = B\,[B/A - L/A] + A = 6.32$ in

and:

A = radius of cylinder = 18.19 inches

B = height of torisphere or ellipsoid above the base (tangent line) = 9.31 inches

L = major radius of torisphere

b = minor radius of torisphere

t = thickness of the shell

Segment lengths used are:

<u>cylinder</u>

$$(rt)^{½} = 2.37 \text{ inches} \qquad \text{(EQ. 3.8-29)}$$

where:

r = distance to pole = 18.16 inches

t = thickness of shell = 0.31 inches

<u>torisphere</u>

5° to 10° - 4 angular segments @ 1.25°

10° to 26.567° - 4 angular segments @ 4.13°

26.567° to 90° - 6 angular segments @ 10.57°

ellipsoid

5° to 10° - 2 4 angular segments @ 1.25°

10° to 30° - 4 angular segments @ 5°

30° to 90° - 6 angular segments @ 10°

Boundary Conditions:

It is assumed that at 5° from the pole, a membrane stress exists in both the ellipsoid and the torisphere:

$$Q = M\phi = 0 \qquad \text{(EQ. 3.8-30)}$$

$$N\phi = \frac{pr}{2 \sin \phi} \qquad \text{(EQ. 3.8-31)}$$

where:

r = distance to pole = 32.778 in

Q = transverse shear in ϕ direction, lb/in

$M\phi$ = moment resultant in ϕ direction, in-lb/in

$N\phi$ = membrane force in ϕ direction, lbs/in

Letting (p) = pressure = 680 psi, then for the torisphere:

$N\phi$ = (680/2)(32.778) = 11,144.5 lb/in

If $N\phi$ = 11,144.5 lb/in, a preliminary run yields:

Q = 95.202 lb/in, so a new value for $N\phi$ for the torisphere is calculated:

$$\Delta N = \frac{\Delta Q}{\tan \phi} \qquad \text{(EQ. 3.8-32)}$$

$N\phi$ = 11,144.5 + ΔN = 10056.3 lb/in and an appropriate membrane state was generated.

For the ellipsoid:

$$r = \frac{A \sin \phi}{R} \qquad \text{(EQ. 3.8-33)}$$

where:

$R = [C_1 + (1 - C_1)\sin^2\phi]^{1/2}$

$C_1 = (B/A)^2 = 0.2519$

$R = 0.5075$

$$N\phi = \frac{A \sin\phi}{R} \frac{P}{2\sin\phi} = 12{,}185.78 \text{ lb/in}$$

To better compare the heads, it seems desirable to have the longitudinal displacement at the center of the cylinder equal to zero ($\mu\phi = \phi$). Thus the problem is run twice; the first run yielding the radial displacement, W, required for zero-displacement at the center (W = 0.0966 in).

Start	W = 0.0966 in
	$N\phi$ = 10,056 lb/in
	$M\phi$ = N = 0
End	Q = N = M0 = 0
	$N\phi$ = 12,186 lb/in

Figure 3.8-86 shows the analytical model, with boundary conditions.

In order to check the results, the answers at the boundaries are examined first. It is assumed that there is a membrane state of stress at the boundaries. Therefore, at the edges Q and $M\phi$ must be approximately zero.

	Q (lb/in)	$M\phi$ (lb-in/in)
Start	-0.08636	0.0
End	-0.0009252	0.0001487

To satisfy equilibrium in the cylinder:

$N\phi \approx 0.5pr = 6169$ lb/in

Plots of the hoop force and longitudinal bending from E0781 results compare the ellipsoidal and torispherical heads. Even though the change in radii is minimized, the disturbance at the junction of the sphere and torus is considerable (Figure 3.8-87).

A comparison with the experimental ellipsoidal head shows a good correlation of stress values. Plots of $\nabla\phi$ and $\nabla\theta$ on the inside, outside, and meridian of the head are shown in Figures 3.8-88 through 3.8-92. Any deviations are caused by the changes in thickness, and by the experimental head's variation from a true 2:1 ellipsoidal head. These variations are shown in Figures 3.8-93 and 3.8-94.

b. Sample Problem B: Cylindrical Water Tank with Tapered Walls

This problem illustrates the program's capability to analyze a pressure load with one fixed boundary and one free boundary.

The problem used for this verification is "Shell of Variable Thickness," taken from Reference 3.8-12 (pp. 289-295).

The problem consists of a tapered shell filled with water. The shell has a radius of 9 feet, and is 12 feet high. The shell thickness varies from 11 inches at the bottom, to 3 inches at the top. These dimensions, and the location of the Z axis, are shown in Figure 3.8-95. The length of a segment is 18 inches $[(rt)^{1/2}]$, where r and t are the same as defined in Sample Problem A.

Since the pressure varies linearly, only two points are needed in the function generator in order to fully describe the function.

It is known that the pressure at the top of the tank, exposed to the atmosphere, is zero. Assuming that the density of the water is 62.5 lb/ft^3, the pressure at the bottom of the tank is:

$$p = \frac{(12\ ft)(62.5\ lb/ft^3)}{(144\ in^2/ft^2)} = 5.21\ psi$$

The boundary conditions for this problem are as follows:

W = displacement normal to surface

Uϕ = displacement component in ϕ direction

$\beta\phi$ = rotation of reference surface in ϕ direction

Q = transverse shear in ϕ direction

Nϕ = membrane force in ϕ direction

Mϕ = moment resultant in ϕ direction

Fixed at start	W = Uϕ = $\beta\phi$ = 0
Free at end	Q = Nϕ = Mϕ = 0

Table 3.8-17 lists the Program E0781 results, and compares them with the theoretical solutions from Reference 3.8-12 at two locations.

Program E0781 yields a maximum hoop force, Nϕ, equal to 346.8 lb/in (4160 lb/ft), 54 inches from the base. This value differs from the theoretical solution of 4180 lb/ft by 0.48%.

Program E0781 yields a maximum moment at the base, M, equal to -1539 in-lb/in (-1539 ft-lb/ft). This value differs from the theoretical solution of -1470 ft-lb/ft by 4.69%.

3.8.7.8 FINEL (Finite-Element Program for Cracking Analysis)

3.8.7.8.1 Application

This program performs a static analysis of stresses and strains in plane and axisymmetric structures by the finite-element method. The program computes the displacements of the corners of each element, and the stresses and strains within each element.

3.8.7.8.2 Program Background

In this program, the structure is idealized as an assemblage of two-dimensional finite-elements of triangular or quadrilateral shapes, having arbitrary material properties. Reinforcement of concrete materials is included by adjusting the element material properties. Bilinearity in compression, and bilinearity or cracking in tension are specially emphasized.

The version of this program currently used by Bechtel is maintained by Control Data Corporation.

3.8.7.8.3 Sample Problems

To verify this program, sample problems were analyzed by FINEL, and compared to experimental and/or hand-calculated solutions. Three sample problems are presented as examples of verification.

a. Sample Problem A: Simply Supported Beam with a Concentrated Load at the Center

The beam shown in Figure 3.8-96 is investigated experimentally and analytically. This investigation compares results obtained from the FINEL program with those obtained from References 3.8-13 and 3.8-14.

The finite-element mesh used in Reference 3.8-14, and in the FINEL analysis are shown in Figures 3.8-97 and 3.8-98, respectively. The FINEL analysis requires a finer mesh because it uses linear displacement elements. Reference 3.8-14 uses quadratic displacement elements.

The material properties of the concrete and reinforcing steel, and the loading history used in the FINEL analysis are given in Table 3.8-18.

The cracking patterns obtained from Reference 3.8-14 and FINEL are shown in Figure 3.8-99. The load-deflection curves from References 3.8-13 and 3.8-14 and the FINEL analysis are shown in Figure 3.8-100. The load-deflection curve obtained from the FINEL analysis shows very good agreement with the experimental results. The cracked region grows faster in the FINEL analysis, and more slowly in Reference 3.8-14, since the FINEL and Reference 3.8-14 load-deflection curves show different gradients (stiffnesses).

The results of analytical, experimental, and FINEL solutions are shown in Figure 3.8-100. The FINEL analysis agrees well with the experimental results, up to the point where the reinforcing steel in the beam yields. After the yield point, the FINEL analysis incorrectly calculated the effective stiffness of elements which have yielded. Therefore, the solution is not valid for further loadings. However, since all reinforcing steel remains elastic for the containment analysis, the FINEL program is verified for that application.

b. Sample Problem B: Axially Constrained Hollow Cylinder with a Distributed Pressure Loading

This problem investigates the response of an axially constrained hollow cylinder to internal pressure. A hand-calculated solution yields values of tangential, axial, and radial stresses at various radii from the center of the cylinder, which are then compared to the FINEL values.

The finite-element model is illustrated in Figure 3.8-101. Nodal points are free to move only in the radial direction, representing the conditions of axisymmetry and plane strain.

The problem parameters are as follows:

Poisson's ratio (υ) = 0.25

Modulus of elasticity (E) = 4.32×10^5 ksf

Number of nodal points = 22

Number of elements = 10

Internal pressure (p) = 1.0 ksf

From References 3.8-15 and 3.8-16, the following equations were used:

Hoop or tangential stress, T :

$$T = p \frac{a^2 (b^2 + r^2)}{r^2 (b^2 - a^2)} \qquad \text{(EQ. 3.8-34)}$$

Axial stress, T_2:

$$T_2 = \frac{2\upsilon P a^2}{b^2 - a^2} = \frac{P}{2} \frac{a^2}{b^2 - a^2} \qquad \text{(EQ. 3.8-35)}$$

Radial stress, T_R:

$$T_R = -p \frac{a^2 (b^2 - r^2)}{r^2 (b^2 - a^2)} \qquad \text{(EQ. 3.8-36)}$$

where:

a = 65.0 feet

b = 68.75 feet

p = 1.0 ksf

$a \leq r \leq b$

The results from FINEL for the hollow cylinder tangential, axial, and radial stresses are compared with the hand-calculated values in Table 3.8-19. The results are exactly the same, except for one value, where there is a 4.17% difference.

c. Sample Problem C: Axially Constrained Hollow Cylinder with a Linear Temperature Gradient

This problem evaluates the response of an axially constrained hollow cylinder, to a radially varying linear temperature gradient. The tangential, axial, and radial stresses are determined by hand calculations, and compared to the FINEL results.

Figure 3.8-102 illustrates the finite-element mesh. The conditions of axisymmetry and plane strain are imposed by using the axisymmetric quadrilateral element, and restraining all nodes against axial displacement.

The temperature profile is shown in Figure 3.8-103.

The problem parameters are as follows:

Poisson's ratio (υ) = 0.25

Modulus of elasticity (E) = 4.32×10^5 ksf

Coefficient of thermal expansion (α) = 6×10^{-6} ft/ft/°F

Number of nodal points = 22

Number of elements = 10

From References 3.8-16 and 3.8-17, the following equations are used:

Hoop or tangential stress, ∂_θ :

$$\partial_\theta = \frac{\alpha E}{1-\upsilon}\frac{1}{r^2}\left[\frac{(r^2+a^2)}{b^2-a^2}\int_a^b Trdr + \int_a^r Tr'dr' - Tr^2\right] \qquad \text{(EQ. 3.8-37)}$$

Axial stress, ∂_z :

$$\partial_z = \frac{\alpha E}{1-\upsilon} \left[\frac{2\upsilon}{b^2 - a^2} \int_a^b Trdr - T \right] \quad \text{(EQ. 3.8-38)}$$

Radial stress, ∂_R :

$$\partial_R = \frac{\alpha E}{1-\upsilon} \frac{1}{r^2} \left[\frac{(r^2 - a^2)}{b^2 - a^2} \int_a^b Trdr - \int_a^r Tr'dr' \right] \quad \text{(EQ. 3.8-39)}$$

where:

a	=	65.0 feet
b	=	68.75 feet
T	=	T(r) = temperature above reference, °F (reference temperature = 100°F)

Expressions for the temperature field are:

$T(r) = C_2r + C_1$ (EQ. 3.8-40)

$T(a) = 25 = C_1 + 65.0C_2$ (EQ. 3.8-41)

$T(b) = -25 = C_1 + 68.75C_2$ (EQ. 3.8-42)

Solving:

$$C_2 = \frac{-50}{68.75 - 65} = -13.33$$

$$C_1 = -25 - 68.75(-13.33) = 891.67$$

Then:

$$T(r) = -13.33r + 891.67$$

Evaluating the integral:

$$\int Trdr = \int (-13.33r + 891.67)rdr$$

$$= -\frac{13.33r^3}{3} + \frac{891.67r^2}{2} + C$$

$$= -4.44r^3 + 445.83r^2 + C$$

$$\int_a^b Trdr = -4.44(b^3 - a^3) + 445.83(b^2 - a^2)$$

$$\int_{s}^{b} Tr'dr' = -4.44(r^3 - a^3) + 445.83(r^2 - a^2)$$

The results from FINEL for the tangential, axial, and radial stresses are compared with the values obtained by hand calculations in Table 3.8-19. The results between the two methods of solution agree very closely.

3.8.7.9 ME620 (Transient Temperature Analysis of Plane and Axisymmetric Solids)

3.8.7.9.1 Application

The heat conduction program, ME620, is used to determine the temperature distribution as a function of time, within a plane or axisymmetric solid body subjected to step-function temperature or heat flux inputs. The program is also used for steady-state temperature analysis.

3.8.7.9.2 Program Background

The program utilizes a finite-element technique coupled with a step-by-step time integration procedure, as described in Reference 3.8-18.

The program was developed at the University of California, Berkeley, by Professor E. L. Wilson, and has been modified by Bechtel to incorporate the save and restart capabilities. The version of this program currently used by Bechtel is maintained by the Control Data Corporation.

3.8.7.9.3 Sample Problems

To verify this program, sample problems were analyzed by ME620 and compared with published data. Two sample problems are presented as examples of verification.

a. Sample Problem A: Heat Conduction in a Square Plate with One Edge Quenched

This problem tests the ability of the program to solve the temperature changes in a plane region subjected to conduction boundary conditions. The plate is brought to an equilibrium temperature, and one edge quenched while the other three edges are kept insulated.

A square plate is brought to equilibrium at a given initial temperature, T_o. Three edges are perfectly insulated, while a third edge is suddenly brought to a lower temperature, T_1. This quench is kept constant for the entire analysis. A temperature-time history is then obtained for the corner farthest from the quenched edge.

Figure 3.8-104 shows the actual plate arrangement, while Figure 3.8-105 shows a diagram of the finite-elements.

The problem parameters are as follows:

Nomenclature

L = length of longest heat flow path, inches

T_o = initial temperature of plate, °F

T_1 = quenching temperature of edge, °F

Data

The plate is 10 inches square

T_o = 100°F

T_1 = 0°F

Diffusivity α = 1.0 in^2/sec (chosen for convenience)

Time increment (ΔT) = 1 second for numerical solution

At any time, t, during the transient state, the time factor T (or characteristic time) is given by:

$$T = \alpha t/L^2$$

The time to reach steady-state is given when T = 1.0; hence the transient time is $t = L^2/\alpha$ = 100 seconds. The results derived from Reference 3.8-19 are plotted in Figure 3.8-106.

The temperature variation at point A is plotted in Figure 3.8-106 according to the results of ME620, and is compared with the theoretical transient change. The curves are seen to agree quite well. Deviations are due to the selected finite-element mesh size, and to the selected time step for the analysis.

b. Sample Problem B: Heat Conduction in a Surface Quenched Sphere

This problem tests the ability of ME620 to analyze the temperature distribution in an axisymmetric solid, with given temperature boundary conditions. The results of the program analysis are compared to a closed-form solution derived from Reference 3.8-20.

This problem considers a solid steel sphere (shown in Figure 3.8-107) which is brought to an equilibrium temperature, and suddenly quenched to a lower uniform temperature on its surface. The quenching environment is held at a constant temperature. A temperature-time history for three seconds is obtained from the program for all node points. The points used for the comparison are at a radius of 0.2 inches; only one time period is checked. The finite-element model is shown in Figure 3.8-108.

The problem parameters are as follows:

Nomenclature

L = length of the longest heat flow path (radius of sphere), inches

T_o = initial temperature of sphere, °F

T_1 = quenching temperature of outer surface, °F

Data

Radius of sphere (R) = 0.59 inches

T_o = 1472°F

T_1 = 68°F

Conductivity = $6.02\text{x}10^{-4}$ Btu/(in-sec-°F)

Diffusivity (α) = 0.0193 in^2/sec

Specific heat = 0.11 Btu/(lb-°F)

Density (p) = 0.284 lb/in^3

Time increment = 0.2 sec

At any time, t, during the transient state, the time factor T (or characteristic time) is given by:

$$T = \alpha t/L^2$$

The time to reach steady-state is given when T = 1.0; hence the transient time is $t = L^2/\alpha$ = 3.0 seconds.

The result from Reference 3.8-20 for the temperature at a radius of 0.2 inches and at time t = 1.8 seconds, is 933.8°F. The result obtained from ME620 is 923.4°F. The difference between these two values is 1.1%.

3.8.7.10 ANSYS

3.8.7.10.1 Application

The ANSYS engineering analysis system computer program is a large-scale general purpose computer program employing finite-element technology for the solution of several classes of engineering analysis problems. Analysis capabilities include static and dynamic; plastic, creep, and swelling; small and large deflections; steady-state and transient heat transfer; and steady-state fluid flow. A variety of finite-elements are available for use in the program. Structural loadings may be forces, displacements, pressures, temperatures, or response spectra. The program output includes joint displacements, element stresses, forces, and moments.

3.8.7.10.2 Program Background

ANSYS was developed by Swanson Analysis Systems, Inc, and is in the public domain. The version of the program currently used by Bechtel is maintained by the Control Data Corporation, Cybernet Service. Version 5.0 was used to evaluate the Spent Fuel Pool for increase storage capacity. This evaluation was performed by Gilbert/Commonwealth and Version 5.0 is maintained by Gilbert/Commonwealth.

3.8.7.11 IMAGES 3D

3.8.7.11.1 Application

The IMAGES 3D computer program is a three-dimensional general purpose finite element program for use on an IBM-compatible personal computer. Analysis capabilities include static, modal, and dynamic analysis of structures composed of frame, plates, shell and solid elements. Structural loadings may be forces, displacements, pressures, temperatures, response spectra, or time history. The program output includes joint displacements, element stresses, forces and moments. It also calculates natural frequencies and mode shapes.

3.8.7.11.2 Program Background

IMAGES 3D was developed by Celestial Software, which is a division of Robert L. Cloud & Associates. The program is in the public domain. Version 3.0 of the program is currently used by the licensee. The documentation package for IMAGES 3D includes a verification manual which documents a series of test problems that demonstrates that the results from IMAGES 3D are similar to those results of other public domain programs or classical solutions.

3.8.8 REFERENCES

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3.8-2 INTENTIONALLY LEFT BLANK

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3.8-18 E. L. Wilson and R. E. Nickell, "Application of the Finite-Element Method to Heat Conduction Analysis", Journal of Nuclear Engineering and Design, (1966).

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3.8-20 Ingersol, Zobel, Ingersol, "Heat Conduction With Engineering, Geological, and Other Applications", Maple Press Co., Inc., York, Pennsylvania, p. 165, (1954).

3.8-21 "Report on the Re-evaluation of Concrete Masonry Walls for the Limerick Generating Station, Units 1 and 2", Bechtel Power Corporation, (April 1982).

3.8-22 "Visual Weld Acceptance Criteria for Structural Welding at Nuclear Power Plants, NCIG-01", Nuclear Construction Issues Group, Revision 2, (May 7, 1985).

3.8-23 "Building Code Requirements for Concrete Masonry Structures", ACI 531 and ACI 531-R, American Concrete Institute, Detroit, (1979).

Table 3.8-1

CODES, STANDARDS, RECOMMENDATIONS, AND SPECIFICATIONS USED IN DESIGN AND CONSTRUCTION OF SEISMIC CATEGORY I STRUCTURES(1)

DESIGNATION	TITLE	EDITION
American Concrete Institute (ACI)		
ACI 214	Recommended Practice for Evaluation of Compression Test Results of Field Concrete	1965
ACI 301	Specifications for Structural Concrete for Buildings	1966
ACI 306	Recommended Practice for Cold Weather Concreting	1966
ACI 307	Specification for the Design of Reinforced Concrete Chimneys	1959
ACI 315	Manual of Practice for Detailing Reinforced Concrete Structures	1965
ACI 318	Building Code Requirements for Reinforced Concrete	1971
ACI 347	Recommended Practice for Concrete Formwork	1968
ACI 605	Recommended Practice for Hot Weather Concreting	1959
ACI 613	Recommended Practice for Selecting Proportions for Concrete	1954
ACI 614	Recommended Practice for Measuring, Mixing, Transporting and Placing Concrete	1959
ACI SP-2	Manual of Concrete Inspection	5th Ed.
American Welding Society (AWS)		
AWS D1.0	Code for Welding in Building Construction	1969
AWS D1.1	Structural Welding Code(2)	1972, 1973 (Rev 1), 1974 (Rev 2), 1975, 1976 (Rev 1), 1977 (Rev 2)
AWS D12.1	Recommended Practice for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction	1961

Table 3.8-1 (Cont'd)

DESIGNATION	TITLE		EDITION
American Society of Mechanical Engineers (ASME)			
ASME	ASME Boiler and Pressure Vessel Code		
	a.	Section II, Part C: Welding Rods, Electrodes, and Filler Metals	1971, 1974 with Addenda through 1976
	b.	Section III, Subsection B: Requirements for Class B Vessels	1968, 1971, 1974 with Addenda through 1976
	c.	Section III, Subsection NB: Class 1 Components	1977 with Addenda through Summer 1979
	d.	Section III, Subsection NC: Class 2 Components	1968, 1971, 1974, 1977 with Addenda through Summer 1978
	e.	Section III, Subsection NE: Class MC Components	1977 with Addenda through Winter 1978
	f.	Section V, Nondestructive Examination	1968, 1971, 1974, 1977 with Addenda through Summer 1978
	g.	Section VIII, Parts UW and UG, and Division 1	1968, 1971, 1974, 1977 with Addenda through Summer 1978
	h.	Section IX, Welding and Brazing Qualification	1968, 1971, 1974 with Addenda through 1976
American Society for Testing and Materials (ASTM)[(3)]			
ASTM A20	General Requirements for Steel Plates for Pressure Vessels		1970, 1974a
ASTM A36	Structural Steel		1969, 1970, 1970a, 1974, 1975, 1977, 1979
ASTM A53	Welded and Seamless Steel Pipe		1970, 1972a, 1973, 1976
ASTM A82	Cold-Drawn Steel Wire for Concrete Reinforcement		1976

Table 3.8-1 (Cont'd)

DESIGNATION	TITLE	EDITION
ASTM A106	Seamless Carbon Steel Pipe for High Temperature Service	1970, 1972a, 1974
ASTM A108	Steel Bars, Carbon, Cold Finished, Standard Quality	1969, 1973
ASTM A123	Zinc (Hot-Galvanized) Coatings on Products Fabricated from Rolled, Pressed and Forged Steel Shapes, Plates, Bars and Strip	1973
ASTM A153	Zinc Coating (Hot-Dip) on Iron and Steel Hardware	1967
ASTM A167	Stainless and Heat-Resisting Chromium-Nickel Steel Plate, Sheet, and Strip	1974
ASTM A193	Alloy-Steel and Stainless Steel Bolting Materials for High Temperature Service	1977
ASTM A240	Heat-Resisting Chromium and Chromium-Nickel Stainless Steel Plate, Sheet, and Strip for Fusion Welded Unfired Pressure Vessels	1977
ASTM A242	High Strength, Low Alloy Structural Steel	1975
ASTM A262	Detecting Susceptibility to Intergranular Attack on Stainless Steels	1970
ASTM A276	Stainless and Heat-Resisting Steel Bars and Shapes	1976
ASTM A285	Pressure Vessel Plates, Carbon Steel, Low and Intermediate Tensile Strength	1969, 1972
ASTM A306	Carbon Steel Bars Subject to Mechanical Property Requirements	1964
ASTM A307	Carbon Steel Externally and Internally Threaded Standard Fasteners	1968
ASTM A312	Seamless and Welded Austenitic Stainless Steel Pipe	1974
ASTM A325	High Strength Bolts for Structural Steel Joints, Including Suitable Nuts and Plain Hardened Washers	1970a, 1971a, 1974
ASTM A386	Zinc Coating (Hot-Dip) on Assembled Steel Products	1967
ASTM A403	Wrought Austenitic Stainless Steel Piping Fittings	1973a

Table 3.8-1 (Cont'd)

DESIGNATION	TITLE	EDITION
ASTM A416	Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete	1974
ASTM A441	High Strength Low Alloy Structural Manganese Vanadium Steel	1970, 1970a, 1975
ASTM A449	Quenched and Tempered Steel Bolts and Studs	1976c
ASTM A479	Stainless and Heat-Resisting Steel Bars and Shapes for Use in Boilers and Other Pressure Vessels	1974, 1977
ASTM A480	Delivery of Flat-Rolled Stainless and Heat-Resisting Steel Plate, Sheet, and Strip	1974a, 1975
ASTM A490	Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints	1975, 1976
ASTM A500	Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes	1974, 1976
ASTM A501	Hot-Formed Welded and Seamless Carbon Steel Structural Tubing	1976
ASTM A516	Pressure Vessel Plates, Carbon Steel for Moderate- and Lower-Temperature Service	1971, 1972, 1973, 1974, 1974a, 1975, 1976
ASTM A519	Seamless Carbon and Alloy Steel Mechanical Tubing	1973, 1977b, 1979
ASTM A527	Steel Sheet, Zinc-Coated (Galvanized) by the Hot-Dip Process	1971
ASTM A570	Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality	1972
ASTM A575	Merchant Quality Hot-Rolled Carbon Steel Bars	1971, 1973
ASTM A606	Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High Strength, Low Alloy, with Improved Corrosion Resistance	1975
ASTM A537	Pressure Vessel Plates, Heat Treated, Carbon-Manganese-Silicon	1967, 1974a
ASTM A569	Steel, Carbon (0.15%, maximum). Hot-Rolled Sheet and Strip, Commercial Quality	1972
ASTM A588	High Strength Low Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 inches Thick	1980a

Table 3.8-1 (Cont'd)

DESIGNATION	TITLE	EDITION
ASTM A615	Deformed and Plain Billet Steel Bars for Concrete Reinforcement	1968, 1972, 1975
ASTM A668	Steel Forgings, Carbon and Alloy for General Industrial Use	1972
ASTM A675	Special Quality Hot-Rolled Carbon Steel Bars Subject to Mechanical Property Requirements	1972
ASTM B209	Aluminum-Alloy Sheet and Plate	1974, 1977
ASTM B308	Aluminum-Alloy Standard Structural Shapes, Rolled or Extruded	1973
ASTM C31	Making and Curing Concrete Test Specimens in the Field	1969
ASTM C33	Concrete Aggregates	1971a, 1978
ASTM C39	Compressive Strength of Molded Concrete Cylinders	1971, 1972
ASTM C40	Organic Impurities in Sand for Concrete	1966, 1973
ASTM C55	Concrete Building Brick	1975
ASTM C70	Surface Moisture in Fine Aggregate	1966, 1973
ASTM C87	Effect of Organic Impurities in Fine Aggregate on Strength of Mortar	1969
ASTM C88	Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate	1971, 1971a, 1973, 1976
ASTM C90	Hollow Load-Bearing Concrete Masonry Units	1975
ASTM C94	Ready-Mixed Concrete	1969, 1972, 1978a
ASTM C109	Compressive Strength of Hydraulic Cement Mortars	1970T, 1970, 1973, 1977
ASTM C114	Chemical Analysis of Hydraulic Cement	1969, 1973, 1977
ASTM C115	Fineness of Portland-Cement by the Turbidimeter	1970, 1973, 1977, 1978a
ASTM C117	Materials Finer than No. 200 Sieve in Mineral Aggregates by Washing	1969, 1976

Table 3.8-1 (Cont'd)

DESIGNATION	TITLE	EDITION
ASTM C123	Lightweight Pieces in Aggregate	1969
ASTM C127	Specific Gravity and Absorption of Coarse Aggregate	1968, 1977
ASTM C128	Specific Gravity and Absorption of Fine Aggregate	1968, 1973
ASTM C131	Resistance to Abrasion of Small Size Coarse Aggregate by Use of the Los Angeles Machine	1969, 1976
ASTM C136	Sieve or Screen Analysis of Fine and Coarse Aggregates	1971, 1976
ASTM C138	Unit Weight, Yield, and Air Content of Concrete	1971T, 1971, 1977
ASTM C140	Sampling and Testing of Concrete Masonry Units	1970
ASTM C142	Clay Lumps and Friable Particles in Aggregates	1971, 1978
ASTM C143	Slump of Portland-Cement Concrete	1971, 1974
ASTM C144	Standard Specification for Masonry Mortar	1966T, 1970
ASTM C145	Solid Load-Bearing Concrete Masonry Units	1975
ASTM C150	Portland-Cement	1968, 1970, 1971, 1972, 1973a, 1977, 1978a
ASTM C151	Autoclave Expansion of Portland-Cement	1971, 1977
ASTM C171	Standard Specification for Sheet Materials for Curing Concrete	1969
ASTM C172	Sampling Fresh Concrete	1971
ASTM C183	Sampling Hydraulic Cement	1971, 1973, 1976
ASTM C185	Air Content of Hydraulic Cement Mortar	1971, 1975
ASTM C186	Heat of Hydration of Hydraulic Cement	1968, 1977, 1978
ASTM C190	Tensile Strength of Hydraulic Cement Mortars	1970, 1972, 1977

Table 3.8-1 (Cont'd)

DESIGNATION	TITLE	EDITION
ASTM C192	Making and Curing Concrete Test Specimens in the Laboratory	1969, 1976
ASTM C207	Standard Specification for Hydrated Lime for Masonry Purposes	1976
ASTM C231	Air Content of Freshly Mixed Concrete by the Pressure Method	1971, 1972T, 1978
ASTM C233	Testing Air Entraining Admixtures for Concrete	1969, 1978
ASTM C235	Scratch Hardness of Coarse Aggregate Particles	1968
ASTM C260	Air Entraining Admixtures for Concrete	1969, 1977
ASTM C266	Time of Setting of Hydraulic Cement by Gillmore Needles	1971, 1977
ASTM C289	Potential Reactivity of Aggregates	1971
ASTM C295	Petrographic Examination of Aggregates for Concrete	1965
ASTM C451	False Set of Portland-Cement	1972, 1975
ASTM C452	Potential Expansion of Portland-Cement Mortars Exposed to Sulfate	1968, 1975
ASTM C469	Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression	1965
ASTM C476	Mortar and Grout for Reinforced Masonry	1971
ASTM C494	Chemical Admixtures for Concrete	1971
ASTM C566	Total Moisture Content of Aggregate by Drying	1967
ASTM C617	Capping Cylindrical Concrete Specimens	1971a, 1976
ASTM C845	Expansive Hydraulic Cement	1980
ASTM D75	Sampling Aggregates	1959, 1971
ASTM E109	Dry Powder Magnetic Particle Inspection	1963, 1971
ASTM E328	Stress-Relaxation Tests for Materials and Structures	1972

Table 3.8-1 (Cont'd)

DESIGNATION	TITLE	EDITION
ASTM E437	Industrial Wire Cloth and Screens (Square Opening Series)	1971
American Association of State Highway and Transportation Officials (AASHTO)		
AASHTO T26	Quality of Water to be Used in Concrete	1970
US Army Corps of Engineers		
CRD C36	Coefficient of Thermal Diffusivity	1948
CRD C39	Thermal Coefficient of Expansion	1955
CRD C79	Method of Test for Flow of Grout Mixtures	1958
CRD C119	Test for Flat and Elongated Particles in Coarse Aggregate	1953
CRD C589	Methods of Sampling and Testing Expensive Grouts	1970
American National Standards Institute (ANSI)		
ANSI N45.2.6	Qualifications of Inspection, Examination, and Testing of Personnel for the Construction Phase of Nuclear Power Plants	1973
American Institute of Steel Construction		
AISC	Specification for the Design, Fabrication and Erection of Structural Steel for Buildings	1969
	Supplement #1	1970
	Supplement #2	1971
	Supplement #3	1974
AISC	Code of Standard Practice for Steel Buildings and Bridges	1970
International Conference of Building Officials		
UBC	Uniform Building Code	1967, 1970

Table 3.8-1 (Cont'd)

DESIGNATION	TITLE	EDITION
American Iron and Steel Institute		
AISI	Specification for the Design of Cold-Formed Steel Structural Members	1968
Society of Automotive Engineers		
SAE J444a	Cast Shot Specification for Shot Peening or Blast Cleaning	1976
American Petroleum Institute		
API 5L	Specification for Line Pipe	1973
API 5LX	Specification for High Test Line Pipe	1973
American Waterworks Association		
AWWA M11	Steel Pipe Manual	1964
Regulations of the Commonwealth of Pennsylvania		
	Regulations for Boilers and Unfired Pressure Vessels	-

(1) For exceptions to the above codes, standards, recommendations and specifications, see Section 3.8.6.
(2) The contractor or fabricator may also qualify its welding procedures and welders in accordance with the latest edition of AWS D1.1 code at the time of qualification.
(3) Principal editions used for design are listed; later editions may be used for specific plant modifications.

Table 3.8-2

LOAD COMBINATIONS FOR PRIMARY CONTAINMENT WALL,
DIAPHRAGM SLAB, AND REACTOR PEDESTAL

The primary containment, diaphragm slab, and reactor pedestal are designed for the following load combinations:

1. Normal Operating Conditions

   a. $U=1.5D+1.8L+1.0T_o+1.25H_o$

   b. $U=1.25(D+L+E+H_o)+1.0T_o$

   c. $U=0.9D+1.25(E+H_o)+1.0T_o$

   d. $U=1.0D+1.0L+1.8E+1.0T_o+1.25H_o$

2. Design Accident and Extreme Environmental Conditions

   a. $U=1.05D+1.05L+1.0T_A+1.0H_A+1.0R+1.5P$

   b. $U=1.05D+1.05L+1.0T_A+1.0H_A+1.0R+1.25P+1.25E$

   c. $U=1.0D+1.0L+1.0T_A+1.0H_A 1.0R+1.0P+1.0E'$

   d. $U=0.95D+1.25E+1.0T_A+1.0H_A+1.0R$[1]

   e. $U=1.05D+1.0B+1.25P'+1.25E$

The pressure-retaining steel elements are designed for the following load combinations[2]:

		Stress Limits
a.	$D+L+1.15P$	1.15 times ASME Section III, Class B for "Normal Operating Conditions"
b.	$D+L+T_A+P$	ASME Section III, Class B for "Normal Operating Conditions"
c.	$D+L+T_A+P+H_A+R+E$	ASME Section III, Summer 1970 Addenda, Figure N-414, "For Emergency Conditions"
d.	$D+L+T_A+P+H_A+R+E'$	ASME Section III, Summer 1970 Addenda, Figure N-414, "For Faulted Conditions"

Table 3.8-2 (Cont'd)

Definitions:

U = Required ultimate load capacity, as defined in ACI 318-71

D = Dead load of structure and major equipment, plus any other major permanent loads contributing stress, such as soil or hydrostatic loads or operating pressures

L = Live loads expected to be present when the plant is operating. For loading conditions that include vertical seismic, replace L by $L_p + L_o$, where:

> L_p = permanent live load to include minor piping, minor equipment and the weight of equipment in lay-down areas: to be accelerated vertically
>
> L_o = 100 psf (except 50 psf for grating and platforms): not to be accelerated vertically

T_o = Thermal effects due to temperature gradient through the wall, under operating conditions

T_A = Thermal effects due to temperature gradient through the wall, under accident conditions

P = Design basis accident pressure load

R = Steam/water jet forces or reactions resulting from the rupture of process piping

E = Load due to the OBE resulting from a horizontal ground acceleration of 0.075 g, and a vertical ground acceleration of 0.05 g

E' = Load due to the design basis earthquake resulting from a horizontal ground acceleration of 0.15 g, and a vertical ground acceleration of 0.10 g

B = Hydrostatic loading due to postaccident flooding of the primary containment to the level of the reactor core

P' = Pressure of the atmosphere in the primary containment, with the containment flooded to the level of the reactor core

Table 3.8-2 (Cont'd)

H_o	=	Force on the structure due to thermal expansion or contraction of hanger critical pipes
H_A	=	Force on the structure due to thermal expansion of pipes, under accident conditions
T	=	Thermal loads resulting from temperature increases under accident conditions.

(1) Where overturning forces cause net tension in the absence of live load
(2) Bending from jet forces are considered as primary stresses.

Table 3.8-3

PRIMARY CONTAINMENT DESIGN PRESSURE AND TEMPERATURE CRITERIA

PORTION OF CONTAINMENT	TIME AFTER START OF ACCIDENT	CONTAINMENT ATMOSPHERE PRESSURE (psig)	CONTAINMENT ATMOSPHERE TEMPERATURE (°F)
Drywell	0 sec[(2)]	0.75	135[(3)]
Suppression Chamber	0 sec[(2)]	0.75	95
Drywell	0-45 sec	55	340
Suppression Chamber	0-45 sec	55	175
Suppression Chamber[(1)]	0-45 sec	25	175
Drywell	45 sec-60 min	35	340
Suppression Chamber	45 sec-60 min	35	175
Drywell	1-3 hrs	35	340
Suppression Chamber	1-3 hrs	35	220
Drywell	3-6 hrs	35	320
Suppression Chamber	3-6 hrs	35	220
Drywell	6-24 hrs	20	220
Suppression Chamber	6-24 hrs	20	220
Drywell	1-4 days	20	200
Suppression Chamber	1-4 days	20	200
Drywell	4-30 days	10	175
Suppression Chamber	4-30 days	10	175

(1) Drywell/suppression chamber maximum differential pressure condition

(2) Just before start of accident (i.e., at operating condition)

(3) This information is based on original design basis conditions. Further evaluation has validated the primary containment design for an initial drywell containment atmosphere temperature of 150°F. The stress indicated in UFSAR Figures 3A-362 to 3A-380, Design Assessment Report for Containment Structures reflects current plant conditions.

Table 3.8-4

LOAD COMBINATIONS AND ALLOWABLE STRESSES FOR
ASME CLASS MC COMPONENTS[(1)]

The drywell head assembly, equipment hatches, personnel lock, suppression chamber access hatches, and CRD removal hatch are designed for the following loading combinations and allowable stresses:

	Stress Limits
D+L+1.15P	1.15 times ASME Section III, Class B for "Normal Operating Conditions"
$D+L+T_A+P$	ASME Section III, Class B for "Normal Operating Conditions"
$D+L+T_A+P+H_A+R+E$	ASME Section III, Summer 1970 Addenda, Figure N-414, "For Emergency Conditions"
$D+L+T_A+P+H_A+R+E'$	ASME Section III, Summer 1970 Addenda, Figure N-414, "For Faulted Conditions"

Piping and electrical penetrations are designed for the following load combinations and allowable stresses:

a. The loads used in the design are as follows:

   1. Moments and forces transmitted by the piping to the penetration due to thermal expansion, weight, earthquake (including inertial effects and anchor movements), and other dynamic loads
   2. Pressures
   3. Thermal transients
   4. Number of operating cycles
   5. Pipe failure loads for faulted condition

b. The loading combinations are specified in Section 3.9.

c. Stress limits specified in ASME Section III, Article NB-3220 are used as the design criteria for Class I flued heads for design, normal and upset, and emergency condition. The rules contained in ASME Section III, Appendix F are used in evaluating the faulted condition for Class I and II flued heads.

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(1) Definitions of symbols are given in Table 3.8-2.

Table 3.8-5

LOAD COMBINATION FOR THE REACTOR SHIELD WALL[1]

The reactor shield wall is designed for the following loading combination:

Condition

Abnormal/Extreme	$D+L+T_A+R+P+E'$

(1) Symbols are defined in Table 3.8-2.

Table 3.8-6

LOAD COMBINATIONS FOR THE SUPPRESSION CHAMBER COLUMNS[1]

The suppression chamber columns are designed for the following loading combinations:

Condition	
Abnormal	$1.05D+1.05L+1.0T_A+1.0R+1.5P$
Abnormal/Severe	$1.05D+1.05L+1.0T_A+1.0R+1.25P+1.25E$
Abnormal/Extreme	$1.0D+1.0L+1.0T_A+1.0R+1.0P+1.0E'$

(1) Symbols are defined in Table 3.8-2.

Table 3.8-7

LOAD COMBINATIONS FOR THE PIPE WHIP RESTRAINTS
AND DRYWELL PLATFORMS[1]

The pipe whip restraints and drywell platforms are designed for the following loading combinations:

Condition	
Normal	D+L
Abnormal[2,3]	D+L+R+T

(1) Symbols are defined in Table 3.8-2, except as noted below.

(2) For the design of pipe whip restraints which also support other pipe support(s), either pipe support SSE reaction load or the pipe rupture load (R) is considered.

(3) Thermal loads can be neglected when it can be shown that they are secondary and self-limiting in nature and where the material is ductile.

Table 3.8-8

LOAD COMBINATION FOR THE SEISMIC TRUSS[1]

The seismic truss is designed for the following loading combination:

Condition	
Abnormal/Extreme	D+R+E'

(1) Symbols are defined in Table 3.8-2.

Table 3.8-9

LOAD COMBINATIONS FOR THE REACTOR ENCLOSURE[6]

A. Reinforced Concrete

1. Normal Operating and Severe Environmental Conditions

a. $U=1.5D+1.8L+1.0T_o+1.25H_o$

b. $U=1.25(D+L+H_o+E)+1.0T_o$

c. $U=1.25(D+L+H_o+W)+1.0T_o$

d. $U=0.9D+1.25(H_o+E)+1.0T_o$[1]

e. $U=0.9D+1.25(H_o+W)+1.0T_o$[1]

f. $U=1.0D+1.0L+1.8E+1.0T_o+1.25H_o$[2]

2. Design Accident and Extreme Environmental Conditions

a. $U=1.05D+1.05L+1.0T_A+1.0H_A+1.0R+1.5P$

b. $U=1.05D+1.05L+1.0T_A+1.0H_A+1.0R+1.25P+1.25E$

c. $U=1.0D+1.0L+1.0T_A+1.0H_A+1.0R+1.0P+1.0E'$

d. $U=0.95D+1.25E+1.0T_A+1.0H_A+1.0R$[1]

e. $U=1.0D+1.0L+1.0E'+1.0T_o+1.25H_o+1.0R$

f. $U=1.0D+1.0L+1.0W'+1.0T_o+1.0H_o$

g. $U=1.0D+1.0L+1.0B_o+1.0E'$

h. $U=1.0D+1.0L+1.0R_o$

B. Steel Structures

1. Normal Operation and Severe Environmental Conditions	Stress Limits
a. $D+L+T_o+H_o$	F_s
b. $D+L+T_o+H_o+E$	$1.25F_s$
c. $D+L+T_o+H_o+W$	$1.33F_s$
d. $D+L+T_o+H_o+E$[2]	F_s

Table 3.8-9 (Cont'd)

2.	Design Accident and Extreme Environmental Conditions		Stress Limits
	a.	$D+L+R+T_o+H_o+P+E'$	See note (3)
	b.	$D+L+R+T_A+H_A+P+E'$	See note (3)
	c.	$D+L+A+T_o+H_o$	See note (3)
	d.	$D+L+T_o+H_o+W'$(4)	See note (3)
	e.	$D+L+R_o$	

C. Post-tensioned Concrete Fuel Pool Girders

1.	Normal Operating Conditions (Including Refueling)		Stress Limits
	For elastic analysis by straight line theory:		
	a.	$D+L+T_o+F$	F_c
	b.	$D+L+E+T_o+F$	$1.25F_c$
	c.	$D+E+T_o+F$(5)	$1.25F_c$
	d.	$D+E'+T_o+F$(5)	$1.50F_c$
	For ultimate strength method:		
	e.	$U=1.5D+1.8L+1.0T_o$	
	f.	$U=1.25(D+L+E)+1.0T_o$	

2. Design Accident and Extreme Environmental Conditions

a. $U=1.05D+1.05L+1.0T_A$

b. $U=1.05D+1.05L+1.25E+1.0T_A$

c. $U=1.0D+1.0L+1.0E'+1.0T_o$

Definitions:

W = Wind load

W' = Tornado wind load

Table 3.8-9 (Cont'd)

B_o = Hydrostatic loading due to flooding of the ECCS and RCIC compartments in the reactor enclosure, to suppression pool equilibrium water level

A = Hydrostatic load due to probable maximum flood

f_s = Calculated stress in structural steel

F_s = Allowable stress for structural steel

F_y = Yield strength of structural steel

F = Prestressing force

F_c = Allowable flexural concrete stress, by straight-line theory

R_o = Blast pressure due to railroad accident or pipeline explosion as discussed in Section 2.2.3.1.1 and defined in Reference 2.2-1. (The effect of ground shock due to blast load is a subject of separate investigation.)

For all other definitions, see Table 3.8-2

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(1) Where overturning forces cause net tension in the absence of live load
(2) For structural elements carrying mainly earthquake forces
(3) The allowable stress in structural steel does not exceed 0.9 F_y in bending, 0.85 F_y in axial tension or compression, and 0.5 F_y in shear. Where F_s is governed by requirements of stability (local or lateral buckling), f_s does not exceed 1.5 F_s. Thermal loads can be neglected when it can be shown that they are secondary and self-limiting in nature and where the material is ductile.
(4) Where protection against tornado forces is required
(5) When effects of upward vertical acceleration under earthquake conditions are considered
(6) Unless they are shown, stress limits comply with the ACI codes and AISC specifications. Deviations shown are consistent with typical nuclear power plant design practices.

Table 3.8-10

LOAD COMBINATIONS APPLICABLE TO SEISMIC CATEGORY I STRUCTURES OTHER THAN PRIMARY CONTAINMENT AND REACTOR ENCLOSURE(5)

A. Reinforced Concrete

1. Normal Operating and Severe Environmental Conditions

a. $U=1.5D+1.8L+1.0T_o+1.25H_o$

b. $U=1.25(D+L+H_o+E)+1.0T_o$

c. $U=1.25(D+L+H_o+W)+1.0T_o$

d. $U=0.9D+1.25(H_o+E)+1.0T_o$(2)

e. $U=0.9D+1.25(H_o+W)+1.0T_o$(2)

For structural elements carrying mainly earthquake forces:

f. $U=1.0D+1.0L+1.8E+1.0T_o+1.25H_o$

2. Design Accident and Extreme Environmental Conditions

a. $U=1.05D+1.05L+1.25E+1.0T_A+1.0H_A+1.0R$

b. $U=0.95D+1.25E+1.0T_A+1.0H_A+1.0R$(2)

c. $U=1.0D+1.0L+1.0E'+1.0T_o+1.25H_o+1.0R$

d. $U=1.0D+1.0L+1.0E'+1.0T_A+1.0H_A+1.0R$

e. $U=1.0D+1.0L+1.0A+1.0T_o+1.25H_o$

f. $U=1.0D+1.0L+1.0W'+1.0T_o+1.0H_o$

g. $U=1.0D+1.0L+1.0R_o$

Table 3.8-10 (Cont'd)

B. Steel Structures

1.	Normal Operating and Severe Environmental Conditions		Stress Limits
	a.	$D+L+T_o+H_o$	F_s
	b.	$D+L+T_o+H_o+E$	$1.25F_s$
	c.	$D+L+T_o+H_o+W$	$1.33F_s$
	d.	$D+L+T_o+H_o+E$[3]	F_s
2.	Design Accident and Extreme Environmental Conditions		Stress Limits
	a.	$D+L+R+T_o+H_o+E'+P$	See note [4]
	b.	$D+L+R+T_A+H_A+E'+P$	See note [4]
	c.	$D+L+A+T_o+H_o$	See note [4]
	d.	$D+L+T_o+H_o+W'$	See note [4]
	e.	$D+L+Ro$	

(1) Symbols are defined in Table 3.8-9.
(2) Where overturning forces cause net tension in the absence of live load
(3) For structural elements carrying mainly earthquake forces
(4) The allowable stress in structural steel does not exceed 0.9 F_y in bending, 0.85 F_y in axial tension, and 0.5 F_y in shear. Where F_s is governed by requirements of stability (local or lateral buckling), f_s does not exceed 1.5 F_s. Thermal loads can be neglected when it can be shown that they are secondary and self-limiting in nature and where the material is ductile.
(5) Unless they are shown stress limits comply with the ACI codes and AISC specifications. Deviations shown are consistent with typical nuclear power plant design practices.

Table 3.8-11

LOAD COMBINATIONS APPLICABLE TO MISCELLANEOUS STRUCTURAL COMPONENTS OF SEISMIC CATEGORY I STRUCTURES[3]

A. Anchor Bolts

		Stress Limits
1.	Normal Operating and Severe Environmental Conditions	
a.	$D+L+H_o$	F_s
b.	$D+L+H_o+(E \text{ or } W)$	$1.25F_s$
2.	Design Accident and Extreme Environmental Conditions	
a.	$D+L+R+H_o+P+E'$	[4]
b.	$D+L+R+H_A+P+E'$	[4]

B. Masonry

		Stress Limits
1.	Normal Operating and Severe Environmental Conditions	
a.	D+L	1.0xUBC[2]
b.	D+L+E	1.25xUBC[2]
2.	Design Accident and Extreme Environmental Conditions	
a.	D+L+P	1.0xUBC[2]
b.	D+L+P+E'	1.33xUBC[2]

(1) Symbols are defined in Table 3.8-9.
(2) As specified in UBC, but without the customary increase in normal allowable working stress due to earthquake
(3) Stress limits shown are deviations from the ACI codes and AISC specifications; these values are consistent with typical nuclear power plant design practices.
(4) Strength method is used. Stress limit in steel shall not exceed 0.85 F_y for tension and 0.50 F_y for shear. The allowable stress in concrete shall be in accordance with ACI 318-71.

Table 3.8-12

MINIMUM TESTING FREQUENCY FOR CONCRETE, CONCRETE MATERIALS, CONCRETE MASONRY, AND MASONRY MATERIALS

MATERIAL	REQUIREMENT	TEST	FREQUENCY
Cement properties	Standard physical and chemical	ASTM C150	Once for each 5000 cubic yards of concrete production
Aggregate	Organic impurities	ASTM C40	Once per 8 hour shift during concrete production
	Soundness of aggregate	ASTM C88	Once for each 5000 cubic yards of concrete production
	Material finer than No. 200 sieve	ASTM C117	Once for each 5000 cubic yards of concrete production
	Lightweight pieces in aggregates	ASTM C123	Once for mix qualification
	Specific gravity and absorption	ASTM C127/C128(1)	Once for mix qualification
	Abrasion of coarse aggregate	ASTM C131	Once for each 5000 cubic yards of concrete production
	Gradation of coarse aggregate	ASTM C136	Once for each size of coarse aggregate during each 8 hour shift of production
	Gradation of fine aggregate	ASTM C136	Twice during each 8 hour shift of production
	Potential reactivity	ASTM C289	Once for each 5000 cubic yards of concrete production
	Petrographic examination	ASTM C295	Once for mix qualification
	Flat and elongated particles	CRD C119	Once per week on each size of coarse aggregate
Water and ice	Quality of water to be used in concrete (to meet the requirements herein)	AASHTO T26	Once every 3 months
Admixtures	Air entraining agent	ASTM C260	Upon delivery at job-site, prior to approval for use
	Water reducing and retarding agent(s)	ASTM C494	Upon delivery at job-site, prior to approval for use

Table 3.8-12 (Cont'd)

MATERIAL	REQUIREMENT	TEST	FREQUENCY
Concrete	Mixer uniformity	National Ready Mix Concrete Association (NRMCA)	Initially, and then once every 2 years
	Slump	ASTM C143	Once for every 35 cubic yards of each class of concrete produced for concrete used in seismic Category I structures. For all other structures, once for every 50 cubic yards of each class of concrete.
	Air content, temperature, unit weight	ASTM C231/C138	Once for each 100 cubic yards of each class of concrete produced
	Compressive strength	ASTM C31/C39	The specified number of cylinders to be tested for design strength at the date specified are cast for each 100 cubic yards of each class of concrete, or at least once per day for each class of concrete
Masonry units	Compressive strength, absorption, weight, and moisture content	ASTM C140	Test frequency is in accordance with ASTM C140
Mortar aggregate	Physical properties	ASTM C144	The first load and each tenth successive load of masonry aggregate is tested
Cement for mortar	Standard physical and chemical properties	ASTM C150	Initially, and thereafter for every 10,000 pounds of cement delivered
Hydrated lime	Physical and chemical properties	ASTM C207, Type S or SA	Initially, and thereafter for every 5000 pounds of lime delivered
Chemtree grout	Compressive strength	ASTM C39	Two cylinders cast each day that grout is placed
	Unit weight	ASTM C138	Three times, at approximately equal time intervals, each day that grout is placed
Mortar	Compressive strength	UBC 24.23	Once per shift per mixer

(1) ASTM C128 is performed more frequently for medium and high density grout/concrete and masonry grout

Table 3.8-13

MEMBRANE FORCE RESULTANTS FROM "ASHSD" PROGRAM

	THIN-SHELL		LAYERED SHELL	
NODE POINT	LONGITUDINAL FORCE (lb/in)	CIRCUMFERENTIAL FORCE (lb/in)	LONGITUDINAL FORCE (lb/in)	CIRCUMFERENTIAL FORCE (lb/in)
1[(1)]	27,000	54,004	27,000	54,004
2	27,000	54,005	27,000	54,005
3	27,000	54,008	27,000	54,008
4	27,000	54,012	27,000	54,012
5	27,000	54,015	27,000	54,015
6	27,000	54,012	27,000	54,012
7	27,001	53,999	27,001	53,999
8	27,001	53,968	27,001	53,968
9	27,001	53,912	27,001	53,912
10	27,000	53,829	27,000	53,829
11	26,999	53,731	26,999	53,731
12	26,997	53,654	26,997	53,654
13	26,994	53,674	26,994	53,674
14	26,989	53,912	26,989	53,912
15	26,984	54,532	26,984	54,532
16	27,111	55,724	27,111	55,724

(1) Node point 1 represents the center of the cylinder.

Table 3.8-14

COMPARISON OF RESULTS FROM "CECAP" PROGRAM AND HAND CALCULATIONS

Sample Problem A: Beam with a Thermal Moment

PARAMETER	CECAP	HAND CALCULATION	% ERROR
f_s	13,150 psi	13,790 psi	5.9
f_c	-331 psi	-318 psi	4.1
kd	7.55 in	7.42 in	1.8
M_T	43,760 in-lb/in	43,690 in-lb/in	0.2

Sample Problem B: Beam with a Real Moment

PARAMETER	CECAP	HAND CALCULATION	% ERROR
f_s	79,170 psi	84,570 psi	6.4
f_c	-1845 psi	-1913 psi	3.6
kd	7.6 in	7.4 in	2.7

Sample Problem C: Beam with a Real Moment and Real Compressive Load

PARAMETER	CECAP	HAND CALCULATION	% ERROR
f_s	41,620 psi	41,320 psi	0.7
f_c	-1908 psi	-1922 psi	0.7
kd	12.2 in	12.7 in	3.9

Table 3.8-15

COMPARISON OF RESULTS FROM "CE668" PROGRAM AND HAND CALCULATIONS

Sample Problem A: Rectangular Plate with a Concentrated Load at the Center

	PARAMETER	HAND CALCULATION	CE668
1.	Deflection (in)		
	@ Node 116	0.00153	0.00151
2.	Moments (in-lbs)		
	M_x @ Node 113	20.92	21.24
	M_y @ Node 117	57.198	56.377

Sample Problem B: Uniform Load on a Rectangular Plate

	PARAMETER	HAND CALCULATION	CE668
1.	Deflection (in)		
	@ Node 11	0.277	0.278
2.	Moments (in-lbs)		
	M_x @ Node 11	52.72	50.92
	M_y @ Node 121	143.55	142.28

Table 3.8-16

COMPARISON OF RESULTS FROM "E0119" PROGRAM AND PUBLISHED RESULTS

Sample Problem A: Welding Neck Flange

1. Design (Operating) Condition

STRESS COMPONENT	ALLOWABLE STRESS (psi)	ACTUAL STRESS (psi) REFERENCE E0119	3.8-9
Bolts	25,000	21,801	-
Longitudinal flange	26,250	22,856	22,865
Radial flange	17,500	10,981	10,982
Tangential flange	17,500	6,799	6,800

2. Bolt-up Condition

STRESS COMPONENT	ALLOWABLE STRESS (psi)	ACTUAL STRESS (psi) REFERENCE E0119	3.8-9
Bolts	25,000	6,077	-
Longitudinal flange	26,250	20,278	20,288
Radial flange	17,500	9,743	9,744
Tangential flange	17,500	6,032	6,033

Sample Problem B: Slip-on Flange

1. Design (Operating) Condition

STRESS COMPONENT	ALLOWABLE STRESS (psi)	ACTUAL STRESS (psi) REFERENCE E0119	3.8-9
Bolts	25,000	20,971	-
Longitudinal flange	26,250	21,160	21,163
Radial flange	17,500	11,128	11,128
Tangential flange	17,500	13,763	13,764

Table 3.8-16 (Cont'd)

2. Bolt-up Condition

STRESS COMPONENT	ALLOWABLE STRESS (psi)	ACTUAL STRESS (psi) REFERENCE E0119	3.8-9
Bolts	25,000	5,671	-
Longitudinal flange	26,250	15,644	15,648
Radial flange	17,500	8,227	8,228
Tangential flange	17,500	10,175	10,177

Table 3.8-17

COMPARISON OF RESULTS FROM "E0781" PROGRAM AND PUBLISHED RESULTS[1]

PROGRAM E0781 RESULTS			PUBLISHED RESULTS
DISTANCE FROM BASE (in)	$N\phi$ (lb/in)	$M\phi$ (in-lb/in)	(Solution At Maximums)
0.0	5.919×10^{-6}	-1539.0	$M\phi$ = -1470 in-lb/in
6.0	21.15	-903.9	-
12.0	71.29	-440.5	-
18.0	134.0	-124.8	-
24.0	194.3	71.47	-
30.0	253.3	177.1	-
36.0	297.2	218.3	-
42.0	327.3	217.6	-
48.0	343.3	192.8	-
54.0	346.8	157.1	$N\phi$ = 348 lb/in
60.0	339.6	119.5	-
66.0	324.2	85.46	-
72.0	303.0	57.80	-
78.0	277.9	36.29	-
84.0	250.8	23.41	-
90.0	222.9	15.00	-
96.0	195.1	10.58	-
102.0	167.8	8.685	-
108.0	141.4	8.075	-
114.0	115.9	7.754	-
120.0	91.45	7.032	-
126.0	68.13	5.584	-
132.0	46.29	3.453	-
138.0	26.50	1.177	-
144.0	94.53	-1.481×10^{-3}	-

[1] Published results are from Reference 3.8-12.

Table 3.8-18

PARAMETERS ASSUMED FOR "FINEL" PROGRAM VERIFICATION

A. Material Properties - Concrete and Reinforcing Steel

PROPERTY	CONCRETE	STEEL
E	4.3×10^6 psi	29×10^6 psi
ν	0.15	0.29
T_{yield}	-4820 psi	±44,900 psi
E_{yield}	0.0	0.0
T_{crack}	+546 psi	-
E_{crack}	1.0	-
Shear stiffness reduction factor for once-cracked concrete	0.5	-

B. Loading History

LOAD, P (lb)	NUMBER OF CYCLES AT LOAD FOR CONVERGENCE
1	1
87,000	4
20,000	4
28,000	1
31,200	4
31,300	1[(1)]

(1) Reinforcing steel yielded

Table 3.8-19

COMPARISON OF RESULTS FROM "FINEL" PROGRAM AND HAND CALCULATIONS

Sample Problem B: Hollow Cylinder with Distributed Pressure Loading

		TANGENTIAL STRESS (ksf)		AXIAL STRESS (ksf)		RADIAL STRESS (ksf)	
ELEMENT	r (ft)	HAND CALCULATION	FINEL	HAND CALCULATION	FINEL	HAND CALCULATION	FINEL
1	65.19	17.79	17.79	4.212	4.212	-0.95	-0.95
2	65.56	17.69	17.69	4.212	4.212	-0.84	-0.84
3	65.94	17.58	17.58	4.212	4.212	-0.73	-0.73
4	66.31	17.48	17.48	4.212	4.212	-0.63	-0.63
5	66.69	17.38	17.38	4.212	4.212	-0.53	-0.53
6	67.06	17.28	17.28	4.212	4.212	-0.43	-0.43
7	67.44	17.18	17.18	4.212	4.212	-0.33	-0.33
8	67.81	17.08	17.08	4.212	4.212	-0.24	-0.23
9	68.19	16.99	16.99	4.212	4.212	-0.14	-0.14
10	68.56	16.89	16.89	4.212	4.212	0.05	-0.05

Sample Problem C: Hollow Cylinder with a Linear Temperature Gradient

		TANGENTIAL STRESS (ksf)		AXIAL STRESS (ksf)		RADIAL STRESS (ksf)	
ELEMENT	r (ft)	HAND CALCULATION	FINEL	HAND CALCULATION	FINEL	HAND CALCULATION	FINEL
1	65.19	-78.34	-78.33	-77.96	-77.96	-0.22	-0.23
2	65.56	-60.67	-60.66	-60.68	-60.68	-0.62	-0.62
3	65.94	-43.10	-43.09	-43.40	-43.40	-0.91	-0.91
4	66.31	-25.63	-25.62	-26.12	-26.12	-1.10	-1.10
5	66.69	- 8.26	- 8.25	- 8.84	- 8.84	-1.19	-1.19
6	67.06	9.01	9.02	8.44	8.44	-1.18	-1.18
7	67.44	26.19	26.20	25.72	25.72	-1.08	-1.08
8	67.81	43.27	43.28	43.00	43.00	-0.88	-0.88
9	68.19	60.26	60.27	60.28	60.28	-0.58	-0.59
10	68.56	77.16	77.17	77.56	77.56	-0.21	-0.21

Table 3.8-20

COMPARISON OF CONTAINMENT LOAD COMBINATIONS WITH SRP 3.8.3

LGS UFSAR TABLE	TYPE OF STRUCTURE	CONDITION	OMISSION	JUSTIFICATION
3.8-5 thru 3.8-8	Concrete and Steel internal structures of containment	All accident conditions	Thermal pipe (R_a)[1] were not included	Thermal pipe reactions are checked locally for structural adequacy. These loads are generally small compared to pipe loads and negligible in combination with other loads.
3.8-7	Drywell Platforms	1) Abnormal	1) Seismic not included	1) Seismic is small in comparison with pipe restraints and jet force loads which have been considered.
		2) Normal	2) Seismic not included	2) Combining seismic with dead and live loads gives higher allowable stresses and thus does not control the design.
3.8-7	Drywell Platforms	Normal Abnormal	1) T_a Neglected	1) Expansion of structural members under thermal loadings are permitted; therefore, thermal-stresses are negligible.
	Seismic Truss	Abnormal/ Extreme	2) P_a Neglected	2) The pressure differential along the supporting beams is negligible.
3.8-8	Seismic Truss	Abnormal/ Extreme	Live load (L) not included	Live loads are negligible in comparison with main steam pipe jet forces (R)[1]

[1] "R" as defined in the UFSAR is equivalent to the expression ($Y_r + Y_j + Y_m$) as defined in the SRP

Table 3.8-21

COMPARISON OF CONTAINMENT LOAD COMBINATIONS WITH ASME CODE, ARTICLE CC-3000

ASME LOAD	DIFFERENCES	ASME CATEGORY	JUSTIFICATION
F	Prestress not applicable	All	LGS has a reinforced concrete containment and no prestress loads exist.
W & W_t	Wind and tornado loads not considered	Construction Severe environmental Abnormal/Severe Environmental Extreme Environmental	Tornado or wind loadings do not apply. The primary containment is protected by the reactor enclosure.
T_t	Thermal not considered for structural integrity test.	Test	LGS design criteria addresses the structural integrity test. Pressure has been increased 15% over ASME. The ambient air temperature during testing is close to that of construction; therefore, the effects of thermal gradient are negligible.
E_o, P_a	Load Factors for E_o and P_a have been slightly reduced.	Abnormal Abnormal/Severe Environmental	Load factors for loading combination are based on probable occurrence. By introducing the hydrodynamic event, a slight load factor reduction for these loads (E_o, P_a) is justifiable.