# 3.8 DESIGN OF SEISMIC CATEGORY I STRUCTURES

## *3.8.1 CONTAINMENT*

### 3.8.1.1 General Description

#### 3.8.1.1.1 Containment Structure

The reactor containment structure is a reinforced-concrete vertical right cylinder with a flat base and a hemispherical dome. A welded steel liner is attached to the inside face of the concrete shell to ensure a high degree of leaktightness. On the inside of the liner every weld seam has a leak test channel welded over it. The channels can be pressurized to design pressure for liner leak testing whenever the containment vessel is open. Exceptions were taken during the 1996 Steam Generator Replacement where two construction openings were created in the dome. The perimeter closure welds for both liner plate opening repairs have leak test channels on the outside of the liner plate. The thickness of the liner in the cylinder and dome is 3/8 in. and in the base is 1/4 in. The containment structure is 99 ft high to the spring line of the dome and has an inside diameter of 105 ft. The containment provides a minimum free volume of approximately 997,000 $ft^3$. An elevation and details of the containment structure are shown in Figures 3.8-1 through 3.8-5.

The cylindrical reinforced concrete walls are 3 ft 6 in. thick, and the concrete hemispherical dome is 2 ft 6 in. thick. These shell thicknesses are established to satisfy the requirements of the structural criteria as well as the shielding requirements. These thicknesses are nominal values. The true relevant engineering values are dependent on the specific location in the structure and the loading condition that is present. The concrete base slab is 2 ft thick with an additional thickness of concrete fill of 2 ft over the bottom liner plate. The containment cylinder is founded on rock (sandstone) by means of post-tensioned rock anchors which ensure that the rock then acts as an integral part of the containment structure. The hemispherical dome is reinforced concrete designed for all moments, axial loads, and shears resulting from the loading conditions described in this section. The cylinder wall is prestressed vertically and reinforced circumferentially with mild steel deformed bars. The base is a reinforced-concrete slab. The rock anchors are used for all vertical axial loads in the cylinder walls and thereby avoid the transfer of an imposed shear to the base slab. The structural systems for the containment structure are summarized as follows:

- Hemispherical dome - mild steel-reinforced concrete.
- Cylindrical walls.
    1. Vertical direction - prestressed concrete.
    2. Circumferential direction - mild steel reinforced concrete.
- Rock anchors - prestressed.

The design ensures that the structure has an elastic response to all loads and that it strains within such limits that the integrity of the liner is not prejudiced. The liner participates with the shell as it reacts to these loads and is designed to ensure the vessel vapor tightness.

The design of the structural elements are more fully described in Sections 3.8.1.2 and 3.8.1.3.

#### 3.8.1.1.2 Waterproofing

No drainage system was provided under the containment structure. The maximum ground water elevation considered during the design of Ginna Station in the vicinity of the containment structure was 252 ft. The design-basis water level has since been revised to 265 ft msl (see Section 2.4.10.1). This compares with an elevation at the underside of the base slab of 231 ft 8 in. It is unlikely that tensile stresses will produce cracks in the outside concrete face because significant constraint is afforded by the irregular surface of the founding rock material. This rock has significant structural characteristics as described in Section 2.5. However, the concrete is not totally impermeable. For this reason, the design of the liner, test channels, backup bars (structural tees), anchors on test channels (refer to Figure 3.8-6) and the concrete cover were based upon accommodating the full hydrostatic head of water. A significant corrosion potential for embedded steel does not exist due to the close contact between the alkaline concrete and steel which provides a highly corrosive-resistant environment for the liner.

The basement floor elevation of the containment vessel is 235 ft 8 in. The exterior of the cylinder walls are covered from the edge of the ring girder to elevation 253 ft 0 in. with a membrane waterproofing. No waterproofing was placed between the foundation material (rock) and the base slab. The liner and liner anchorage at the base of the vessel were designed to withstand a theoretical pore pressure equal to the hydrostatic head of water, 7.7 psi. The site is not subject to significant fluctuations in the ground water elevations. Consequently, if the base liner is subject to the assumed water pressure, this pressure should remain essentially constant.

The net buoyant force due to the hydrostatic pressure acting on the containment base is transmitted by the base slab to the cylinder walls.

#### 3.8.1.1.3 Rock Anchors

The side walls of the containment are anchored to the foundation rock with prestressed rock anchors. The anchors place a preload between the foundation rock and a ring beam at the base of the side wall. The tendons in the side walls are coupled to the rock anchors and extend to a location 12 ft 6 in. above the spring line to provide accessibility to the upper anchorage and to permit tensioning following the completion of the dome concrete work. A removable cover is placed over the top anchorage head for protection and to provide an expansion reservoir for the tendon protection system. Refer to Figure 3.8-7 for details of this enclosure.

The outer surface of the containment can be inspected, except in those limited areas where roofs, floors, and walls of adjacent buildings preclude access.

#### 3.8.1.1.4 Construction Sequence

The sequence for the construction of the shell of the containment structure was as follows:

A. Excavation was completed and the exposed rock examined by a soils engineer to ensure its competence.

B. The concrete for the portion of the ring girder at the base of the cylindrical wall was placed. Sleeves and bearing places for the rock anchors were embedded in this concrete pour.

C. The holes for rock anchors were drilled through the embedded sleeves and into the rock. The anchor, which was completely fabricated in the shop, was inserted and the first stage grout placed. Following the required curing period the anchor was tensioned and the second stage grout inserted under pressure.

D. The concrete for the base mat was placed with embedded bars for the backup of liner welds. The outer concrete pour contains the tension bars (dowel at base of cylindrical walls). The base slabs for the sumps and pit also were installed with embedded bars for backup of liner welds. The liner for the walls of the sumps was then erected and used as an inner form for the placement of concrete.

E. The liner was erected starting on the base and continuing to the knuckle, the cylindrical wall, and the dome. All electrical and mechanical penetrations (i.e., sleeves for penetrations) were installed as liner erection progressed. Essentially all electrical and mechanical penetrations were shop assembled in the cylindrical wall plates. Provision was made to install the equipment access hatch and personnel air locks at a later stage of construction. Temporary openings were provided in the liner cylindrical wall for construction access requirements.

   The closure procedure for temporary openings in the liner was similar to that for steel tank construction. Initially, special reinforcement was provided around the periphery of the temporary openings. A sufficient width of plate extended beyond the limits of the concrete placement to preclude detrimental heatup of the concrete due to the welding of the closure plate. The welding procedures were identical to that used for all liner weld seams.

   The preparation of construction joints and placement of concrete in temporary openings is as described in Appendix 3B Attachment 1.

F. The tendon conduit was embedded in the second ring girder pour with provision made for installing the tendon and completing the coupling of rock anchor to the sidewall tendon. The enclosure about the coupling was welded to the anchor plate and a window removed to permit making-up the coupling. An expansion bellows was provided where differential motion will occur at the level of the elastomer pads.

G. The elastomer pads were installed and tendon conduit plus mild steel reinforcing placed. The mild steel reinforcing was temporarily supported from the tendon conduit and the stiffeners on the liner. Concrete placement at temporary openings was delayed and provision made to stagger reinforcement splices at these locations as well as elsewhere on the structure. For the cylindrical wall and dome, the liner was used as an inner form.

H. Where grade is adjacent to the structure, a retaining wall was erected to ensure no earth was placed against the cylindrical wall.

I. The concrete cylindrical wall was completed and temporary openings closed after they no longer were required for construction. The reinforcement rings about the equipment access hatch and personnel air locks were installed. The reinforcement about the equipment access hatch was not placed until after major components were installed.

The cylinder walls were placed with horizontal joints spaced at approximately 11-ft centers. Vertical joints were spaced at approximately 42.5-ft centers (i.e., the cylinder was divided into approximately eight equal pours). The final six lifts were poured with the spacing of vertical joints increased to approximately 57 ft (i.e., six approximately equal pours). Form ties consisting of 0.5-in. diameter threaded studs spaced at approximately 2-ft centers were welded to the liner (both plate and channel anchors) for attaching the liner to the outer form. The outer form was supported by cantilever construction from the lower pour. No attempt was made to stagger vertical or horizontal joints. A minimum delay of 3 days was maintained before placing new concrete against abutting pours. Initial and final concrete curing were by the wet method as specified in ACI 301-66.

The dome concrete (i.e., all concrete above the ledge at elevation 343 ft 2 in.) was placed as continuous rings with a chord width of approximately 4.2 ft. The final pour (center "dollar" section) consisted of an approximately 8-ft diameter section. All concrete was placed in one pour for the full thickness of the concrete shell. A galvanized expanded metal mesh located 1 in. inboard of the exposed face was used as an outer form on the greater sloped portion of the dome (i.e., up to an angle of approximately 55 degrees from the spring line). Form ties in the form of 0.5-in. diameter studs were welded to the liner plate and attached to the cage of reinforcing bars. A minimum delay of 3 days was maintained before placing new concrete against the previous concrete ring.

During the 1996 Steam Generator Replacement outage two construction openings were created in the containment dome. The removed liner plate sections were reused. Stiffener angles welded to the liner plate sections for rigging removal replaced the original 0.5 in. diameter studs. The reinforcing bars were supported off the stiffener angles and support chairs. The dome openings were then boarded and poured monolithically. Board form box-outs were used to place and consolidate concrete.

J. The tendons were installed in the embedded conduits and the sidewall tendon and the rock anchor were coupled. The remaining concrete pour in the ring girders was completed and the wax inserted into the conduit.

K. The concrete dome was completed and the sidewall tendons tensioned.

L. Following the tensioning of the tendons, the equipment access hatch with inset personnel air lock plus the second personnel air lock were installed. The containment was then ready for structural and leakage testing.

### 3.8.1.1.5 Steel Reinforcement

The principal dome reinforcement is continuous except for the anchorage at elevation 366 ft 8 in. which is provided in the form of a mechanical connection to a continuous circumferential plate. Additional steel to control spalling on the outer face of the shell is provided in the form of welded wire fabric. At the dome to cylinder discontinuity additional reinforcement is provided on both faces with 180-degree hooks with total anchorage provided to satisfy the requirements of ACI 318-63. Details are shown on Figure 3.8-5.

In the anchorage zone of the prestressing steel, the major steel provided to withstand bursting forces consists of continuous spirals. Radial reinforcement is provided with 180-degree hooks around the vertical flexural steel for anchorage. Vertical (meridional) reinforcement used for flexural and temperature resistance is lap spliced in accordance with ACI 318-63 requirements on the basis of splice requirements at points of maximum tensile stress. Details are also shown on Figure 3.8-5.

All principal circumferential reinforcement is continuous except at the small penetrations where mechanical anchors are provided, as shown typically on Figure 3.8-4, and for a limited number of bars at the large openings, as described in Appendix 3B. Vertical (meridional) reinforcement is lap spliced (except for special large size bars which are Cadweld spliced) in accordance with ACI 318-63 requirements on the basis of splices at points of maximum tensile stress. At the base of the wall all vertical (meridional) reinforcement is provided with 90-degree hooks for anchorage. Details are shown on Figure 3.8-4.

### 3.8.1.2 Mechanical Design Bases

#### 3.8.1.2.1 General

The containment safety design basis and principal design criteria are contained in Section 6.2.1.1. The containment vessel is a steel-lined concrete shell designed to ensure that it responds elastically to all loads and strains within such limits that the integrity of the liner is not prejudiced. The liner is anchored so as to ensure composite action with the concrete shell.

The containment structure is designed based upon limiting load factors which are used as the ratio by which accident and earthquake loads are multiplied for design purposes to ensure that the load/deformation behavior of the structure is one of elastic, low-strain behavior. This approach places minimum emphasis on fixed gravity loads and maximum emphasis on accident and earthquake loads. Because of the refinement of the analysis and the restrictions on construction procedures, the load factors primarily provide for a safety margin on the load assumptions. Load combinations and load factors utilized in the design which provide an estimate of the margin with respect to all loads are tabulated in this section.

#### 3.8.1.2.2 Design Loads

The following loads were considered in the structural design:

- Internal pressure.
- Test pressure 69 psig.
- Live loads Roof loads plus pipe reactions.
- External pressure 2.5 psig.
- Wind load.
- Internal temperature.
    1. Accident.
    2. Operating - 120°F. (Reference 54 increased the maximum operating temperature to 125°F)
- Seismic ground accelerations.
- Dead loads.
- Prestressing loads.

The thermal loads on the containment vessel and their variation with time are developed on the basis of the transients discussed in Section 6.2.1.2. The seismic loads were evaluated as outlined in Section 3.8.1.3. The wind and snow loads used for the design of structures were those specified in State Building Code for the State of New York. The wind loads given in this code are as follows:

| Height Above Ground (ft) | Pressure Load (psf) |
|---|---|
| 0-15 | 12 |
| 16-25 | 15 |
| 26-40 | 18 |
| 41-60 | 21 |
| 61-100 | 24 |
| 101-200 | 28 |

The snow load specified in the code for the plant location is 40 psf for a flat roof. This value also was used in the design of the containment.

### 3.8.1.2.3 Design Stress Criteria

#### *3.8.1.2.3.1 Limiting Loads*

The design was based upon limiting load factors which were used as the ratio by which accident, earthquake, and wind loads were multiplied for design purposes to ensure that the load deformation behavior of the structure is one of elastic, low-strain response. The loads utilized to determine the required limiting capacity of any structural element on the containment were computed as follows:

- $C = 0.95\ D + 1.5\ P + 1.0\ T$
- $C = 0.95\ D + 1.25\ P + 1.0\ T' + 1.25\ E$
- $C = 0.95\ D + 1.0\ P + 1.0\ \underline{T} + 1.0\ E'$

Symbols used in the above equations were defined as follows:

C = Required load capacity of section.

D = Dead load of structure.

P = Accident pressure load - 60 psig.

T = Thermal loads based upon temperature transient associated with 1.5 times accident pressure.

T′ = Thermal loads based upon temperature transient associated with 1.25 accident pressure.

T = Thermal loads based upon temperature transient associated with accident pressure.

E = Seismic load based on 0.08g ground acceleration.

E′ = Seismic load based on 0.20g ground acceleration.

If the required resisting capacity on any structural component resulting from the wind load on any portion of the structure exceeded that resulting from the design earthquake, the wind load, W, was used in lieu of E in the second equation. The factor of 1.05 times dead load was used when it controlled in determining the required load capacity. All structural components were designed to have a capacity required by the most severe loading combination.

#### *3.8.1.2.3.2 Load Factors*

The load factors used in these equations make provision for safety of the containment structure in the same manner as does the ultimate strength design procedure in ACI 318. Because of the refinement of the analysis and the restrictions on construction procedures, the load factors in the design primarily provide for a safety margin on the load assumptions. The load factors utilized in the criteria were based upon the load factor concept employed in Part IV-B, Structural Analysis and Proportioning of Members - Ultimate Strength Design, of ACI 318-63. The load factor of 0.95 applied to the dead load represents the accuracy of dead load calculations (i.e., ±5%) considering the greater severity of reduced dead loads for tension members. The load factor applied to accident pressure loads was consistent with that suggested by Waters and Barrett (*References 1* and *2*) as the limit of low-strain behavior on prestressed concrete pressure vessels for nuclear reactors. This factor was also consistent with the proposed set of "French Regulations Concerning Concrete Reactor Pressure Vessels" wherein it was stated that: The design pressure shall not exceed 2/5 of the pressure calculated to bring about destruction of the structure by rupture of the cables. The load factor considering a tendon stress of 0.60 $f_u$ at factored load would therefore equal 0.6 divided by 2/5 or 1.5. The load factor of 1.25 applied to the design earthquake load is consistent with that utilized in ACI 318 Part IV-B, Chapter 15.

The containment design includes the consideration of both primary and secondary stresses. When a structure experiences only elastic strains there is only a minimal relief of restraints causing secondary stresses. If a structure experiences increased strains beyond the elastic range, the restraints at any point will cease to be as significant due to local yielding in these regions and, if increased loads were applied until collapse of the structure was imminent, all restraints would be effectively removed and only membrane forces (primary stresses) should be experienced, unless premature shear failure were to occur. The design limit for the containment structure was conservatively established to ensure elastic, low-strain behavior at design loads thereby requiring design consideration of all secondary stress effects.

#### *3.8.1.2.3.3 Maximum Thermal Load*

The maximum expected values of T (thermal load) at any section are based upon the following conditions:

a. The maximum operating temperature inside the containment is 125°F and the minimum ambient temperature outside the containment is -10°F. The analysis used the original 120°F in containment. Reference 54 increased the allowable temperature to 125°F. The peak temperature will only be reached during hot outside temperatures such that this gradient is very conservative. See Section 3.8.2.2.2 of UFSAR for SEP re-evaluation of containment.

b. The maximum temperature of the inner surface of the liner (inner face of insulation where the liner is insulated) will be that temperature associated with the factored load, 1.5 times accident pressure. This temperature is approximately 312°F. The design of the shell where the liner is insulated is based upon a maximum temperature rise of 10°F in the liner coincident with maximum pressure.

c. The maximum operating temperature at the basement floor elevation is 125°F and 5 ft below the floor elevation it is 50°F. The upper 2 ft of the basement slab were designed for a transient thermal gradient equal to 30°F. Thermal expansion of the basement slab approximately balances drying shrinkage.

The steady-state operating thermal transient considered in the design for winter conditions (external ambient temperature equals -10°F) is shown on Figure 3.8-8. The steady operating thermal transient for summer conditions was not developed in detail in that it was concluded that such a condition would not affect the reinforcement requirements because a lesser gradient would exist.

The transient thermal gradients through the containment shell for the insulated liner due to the design-basis accident (factored loads) was assumed for purposes of analysis to be the superposition of a liner increase of 10°F onto the operating thermal gradient described above. This is conservative as compared to the expected results described in Appendix 3E. The maximum concrete fiber temperature where the liner is uninsulated (dome) is 220°F in the region immediately in contact with the liner. The calculated shell elongation due to the pressure load exceeds the concrete fiber elongations due to the thermal load indicating that no restraint of concrete thermal growth occurs.

### 3.8.1.2.4 Load Capacity

#### *3.8.1.2.4.1 Reinforced Concrete*

The value of Young's modulus ($E_c$) for uncracked concrete was assumed to be 4.1 x $10^6$ psi calculated on the basis of the equation in Table 1002(a) of ACI 318-63. The $E_c$ and Poisson's ratio ($\nu_c$) for cracked concrete were assumed to be zero. This latter assumption is considered to be substantiated by test data (*References 3 through 5*) for reinforcement experiencing stresses in excess of the 20,000 to 30,000 psi. (Refer to Appendix 3B regarding similar assumptions regarding the analysis of large openings.)

This structure is prestressed vertically only and the liner is insulated in the prestressed portion. The liner stresses (meridional direction) were calculated to be 4500 psi compression based upon a prestress force of 0.70 fs. The concrete strain due to creep and shrinkage was established as being 320 x $10^{-6}$ in./in. This increases the liner stress to 14,100 psi compression at the end of plant life.

Concrete reinforcement is intermediate grade billet steel conforming to ASTM A15-64 and A408-62T with a guaranteed minimum yield strength of 40,000 psi. Replacement reinforcement used for the Steam Generator Replacement dome opening repairs is ASTM A615 Grade 0. This reinforcement exceeds the minimum yield strength requirements of the original reinforcement. The design limit for tension members (i.e., the capacity required for the factored loads) is based upon the yield stress of the reinforcing steel. No mild steel reinforcement will experience average strains beyond the yield point at the factored load.

The load capacity so determined is reduced by a capacity reduction factor, $\phi$, which provides for the possibility that small adverse variations in material strength, workmanship, dimensions, and control, while individually within required tolerances and the limits of good practice, occasionally may combine to result in any under capacity. The coefficient $\phi$ is 0.95 for tension, 0.90 for flexure, and 0.85 for diagonal tension, bond, and anchorage. The coefficient $\phi$ of 0.95 for tension members compares with a coefficient of 0.90 utilized in ACI 318 for ultimate strength design of flexured members. However, in a tension member, unlike the case of a flexural member, only the variation of steel strength and not concrete strength is of concern. Also, the effect of reinforcement misplacement is not as critical as it is for a flexural member. Therefore, the capacity reduction factor of 0.95 is considered to be conservative.

The two equations developed previously for the loss-of-coolant accident and the loss-of-coolant accident combined with the design (operating-basis) earthquake could be written as follows for the mild steel reinforced sections:

C = 0.95 Y.P. = 0.95 D + 1.0 T + 1.5 P

C = 0.95 Y.P. = 0.95 D + 1.25 P + 1.0 T' + 1.25 E

To compare these equations with a working strength design the following equations are developed:

$$f = \frac{(D + P + T)0.95}{(0.95 \cdot D + 1.5 \cdot P + 1.0 \cdot T)} = 63\%$$

(Equation 3.8-1)

$$f = \frac{(D + P + T + E)0.95}{(0.95 \cdot D + 1.25 \cdot P + 1.0 \cdot T' + 1.25 \cdot E)} = 74\%$$

(Equation 3.8-2)

The new symbol in the above equation is defined as follows:

*f* = Ratio of the working stress to yield stress.

#### *3.8.1.2.4.2 Prestressed Concrete*

The design for the containment provides for prestressing the concrete in the cylinder walls in the longitudinal (vertical) direction with a sufficient compressive force to ensure that upon application of the design load combinations there will be no tensile stresses in the concrete due to membrane forces. In addition to the membrane stresses, there are also flexural and shear stresses which result from discontinuity effects. On the basis of the design criteria, the concrete stresses and the stresses on the mild steel reinforcing upon application of the combined loads will then be produced by combined flexure and shear and/or compression. The structural elements are then acting in a manner similar to those tested as a basis for ACI 318- 63 Chapter 17, Shear and Diagonal Tension -Ultimate Strength Design, and there is a basis for designing shear reinforcement.

The steel tendons for prestressing consist of high tensile, bright, cold drawn and stress-relieved steel wires conforming to ASTM A 421-59 T, Type BA, Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete, with a minimum tensile stress of 240,000 psi.

The prestressed concrete is assumed to develop no tensile capacity in a direction normal to a horizontal plane. The design load capacity of tension elements is based upon a resultant condition of zero concrete stress due to the maximum combination of primary and secondary membrane forces. Any nominal secondary tensile stresses due to bending will be assumed to cause partial cracking. Mild steel reinforcing will be provided to control this cracking by limiting crack width, spacing, and depth. The load capacity so determined will be reduced by a capacity reduction factor, $\phi$, which will be conservatively established as 0.95 which compares with a coefficient of 0.90 utilized in ACI 318 for ultimate design of flexural members. In a prestressed tension member only variations in the field-applied tensioning loads are of any concern. Tendon location and concrete strength variations are not critical as they are for flexural members.

Generally, if no tension stresses can be developed in the concrete, prestressed concrete tension members have a relatively low reserve strength above the point of zero stress. If cracking is initiated as the very low tensile stresses are developed in the concrete, all additional loads will be carried by the steel alone. Since the prestressing steel has a relatively small area of cross-section, the strain at any section increases markedly after cracking begins. For this reason, the containment design was conservatively based upon no complete cracking of any prestressed wall section.

Tensile stresses in the concrete resulting from diagonal tension will be permitted. The nominal shear stresses as a measure of this diagonal tension will be less than the maximum value stipulated in Chapter 17 of ACI 318.

The steel tendons are stressed during the post-tensioning operation to a maximum of 80% of ultimate strength and locked-off for an initial stress of 70% of the ultimate strength. The maximum effective prestress is determined, taking into consideration allowances for the following losses which are deduced from the transfer prestress:

- Elastic shortening of concrete.
- Creeping of concrete.
- Shrinkage of concrete.
- Relaxation of steel stress.

- Frictional loss due to intended or unintended curvature of the tendons.

In no event does the effective prestress exceed 60% of the ultimate strength of the prestressing steel or 80% of the nominal yield point stress of the prestressing steel, whichever is smaller. The design of all prestressed concrete elements for shear, bond, and other design considerations is in accordance with ACI 318-63 Chapter 26, Prestressed Concrete.

The prestressing force applied in the field was determined by measuring tendon elongation and also by checking jack pressure on a calibrated gauge or by the use of an accurately calibrated dynamometer. The cause of any discrepancy which exceeded 5% was ascertained and corrected. Elongation requirements were taken from load-elongation curves for the steel used.

With the exception of the large openings (refer to Appendix 3B) reinforcing bars are not draped around openings. Consequently, the minimum radius is the radius of the cylinder. The reinforcement about small openings is shown typically on Figure 3.8-4. The horizontal reinforcement is concentrated near the hole to accommodate stress concentrations. The tendons are draped only if required for clearances. The magnitude of prestress under construction and operating conditions is well within accepted limits based on ACI 318 requirements. The initial average membrane stress is 640 psi. Even a stress concentration factor of 3 results in acceptable stresses. The requirements for anchoring reinforcing bars are discussed in Section 3.8.1.4.5.4, Anchorage Stresses.

#### *3.8.1.2.4.3 Liner*

The liner is carbon steel plate conforming to ASTM A442-60T Grade 60 with a minimum yield of 32,000 psi. The liner plate thickness is 1/4 in. for the base and 3/8 in. for the cylinder and dome. Original liner welds in general were made from both sides of the plate and therefore backup strips were not used. In the base where the liner was welded to structural tees, the tees were continuous at all plate intersections.

During the 1996 Steam Generator Replacement outage, construction openings were created in the dome. Liner plate sections were removed during the replacement, prepped on the ground, then lifted and welded back in place. As required, ASTM A516 Grade 60 plate with a minimum yield of 32,000 psi was used in the liner repair. All seam welds for removed liner sections were made from the exterior only with the use of backing bars. The backing bars were left in place.

The load capacity is based upon the yield stress of the liner as reduced by the capacity reduction factor, $\phi$, previously described. Sufficient anchorage is provided to ensure elastic stability of the liner. Anchorages are in the form of stagger welded channels on the cylinder and studs on the dome. Liner plate stiffener angles were used in lieu of studs at the locations where dome openings were repaired following the Steam Generator Replacement in 1996.

Insulation is provided for the side walls to a point 15 ft 0 in. above the spring line so as to limit the maximum liner temperature due to the loss-of-coolant accident and thereby avoid excessive compressive stresses in the steel plate.

All weld seams in the liner plate are covered with a test channel to permit testing of leaktightness. Except for the equipment access hatch, all penetrations provide a double barrier against leakage and can be pressurized to permit testing of leaktightness. The equipment access hatch contains weld seams with no test channels. The liner plate on the base of the containment is welded to backup bars. These bars are continuous, as shown in Figure 3.8-6.

##### *3.8.1.2.4.4 Rock*

The containment is founded on rock (sandstone) for which the soils consultant recommended an allowable bearing pressure of 35 tons per square foot. The maximum bearing pressure occurs under the ring girder where the maximum bearing pressure was limited to 30 tons per square foot. This bearing pressure occurs under operating conditions and is reduced under incident conditions. The soils consultant also recommended a limit on the lateral resistance of the rock of 25,000 psf. The maximum lateral pressure, occurring at the ring girder under the combination of operating and incident loads is 24,000 psf. A detailed description of subsurface conditions is found in Section 2.5.

#### 3.8.1.2.5 Codes and Standards

The design, materials, fabrication, inspection, and proof testing of the containment complied with the applicable parts of the following:

1. ASME Boiler and Pressure Vessel Code, Section III - Nuclear Vessels, Section VIII - Unfired Pressure Vessels, Section IX - Welding Qualifications.
2. Building Code Requirements for Reinforced Concrete (ACI 318-63).
3. American Institute of Steel Construction Specifications:
    a. Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings, adopted April 17, 1963.
    b. Code of Standard Practice for Steel Buildings and Bridges, revised February 20, 1963.
4. USAS N 6.2 - 1965, Safety Standard for Design, Fabrication, and Maintenance of Steel Containment Structures for Stationary Nuclear Power Reactors.
5. ACI 306-66, Specifications for Structural Concrete for Buildings.
6. ASTM C 150-64, Specifications for Portland Cement.
7. State of New York Department of Public Works Specification.
8. ASTM C 260-63T, Specifications for Air-Entrained Admixtures for Concrete.
9. ASTM A 15-64T, Specifications for Billet-Steel Bars for Concrete Reinforcement.
10. ASTM A 305-56T, Specifications for Minimum Requirements for Deformation of Deformed Bars for Concrete Reinforcement.
11. ASTM A408-64T, Specifications for Special Large Size Deformed Billet-Steel Bars for Concrete Reinforcement.
12. ASTM C 94-65, Recommended Practice for Winter Concreting.
13. ACI 306-66, Recommended Practice for Winter Concreting.
14. ACI 605-59, Recommended Practice for Hot Weather Concreting.

15. ASTM A 421-65, Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete.
16. ASTM C29-60, Method of Test for Unit Weight of Aggregate.
17. ASTM C 40-66, Method of Test for Organic Impurities in Sands for Concrete.
18. ASTM C 127-59, Method of Test for Specific Gravity and Absorption of Coarse Aggregate.
19. ASTM C 128-59, Method of Test for Specific Gravity and Absorption of Fine Aggregate.
20. ASTM C 136-63, Method of Test for Sieve or Screen Analysis of Fine and Coarse Aggregate.
21. ASTM C 39-64, Method of Test for Compressive Strength of Molded Concrete Cylinders.
22. ASTM C 192-66, Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Laboratory.
23. ASTM A 15-62T, Specifications for Billet-Steel Bars for Concrete Reinforcement.
24. ASTM A408-64, Specifications for Special Large Sized Deformed Billet-Steel Bars for Concrete Reinforcement.
25. ASTM A 432-64, Specification for Deformed Billet-Steel Bars for Concrete Reinforcement with 60,000 psi Minimum Yield Strength.
26. ASTM C 31-65, Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Field.
27. ASTM C33-64, Specifications for Concrete Aggregates.
28. ASTM C42-64, Methods of Securing, Preparing, and Testing Specimens from Hardened Concrete for Compressive and Flexural Strengths.
29. ASTM C 131-64T, Method of Test for Abrasion of Coarse Aggregate by Use of the Los Angeles Machine.
30. ASTM C 138-63, Method of Test for Weight per Cubic Food, Yield, and Air Content (Gravimetric) of Concrete.
31. ASTM C 143-58, Method of Test for Slump of Portland Cement Concrete.
32. ASTM C 150-65, Specifications for Portland Cement.
33. ASTM C 172-54, Method of Sampling Fresh Concrete.
34. ASTM C 231-62, Method of Test for Air Content of Freshly Mixed Concrete by the Pressure Method.
35. ASTM C 260-65T, Specifications for Air-Entrained Admixtures.
36. ASTM C 494-62T, Specifications for Chemical Admixtures for Concrete.
37. ASTM C 173-58, Method of Test for Air Content of Freshly Mixed Concrete by the Volumetric Method.
38. ACI 214-57, Recommended Practice for Evaluation of Compression Test Results of Field Concrete.
39. ACI 315-65, Manual of Standard Practice for Detailing Reinforced Concrete Structures.

40. ACI 347-63, Recommended Practice for Concrete Formwork.
41. ASTM D 287-64, Method of Test for API Gravity of Crude Petroleum and Petroleum Products (Hydrometer Method).
42. ASTM D 97-66, Method of Test for Pour Points.
43. ASTM D 92-66, Method of Test for Flash Point by Cleveland Open Cup.
44. ASTM D 88-56, Method of Test for Saybolt Viscosity.
45. ASTM D 937-58, Method of Test for Cone Penetrations of Petroleum.
46. ASTM D 512-62T, Methods of Test for Chloride Ion in Industrial Water and Industrial Waste Water.
47. ASTM D 1255-65T, Method of Test for Sulfides in Industrial Water and Industrial Waste Water.
48. ASTM D 992-52, Method of Test for Nitrate Ion in Industrial Water.
49. ASTM A 442-60T, Tentative Specifications for Carbon Steel Plates with Improved Transition Properties.
50. ASTM A 300-63T, Specifications for Steel Plates for Pressure Vessels for Service at Low Temperature.
51. ASTM A 36-63T, Specifications for Structural Steel.
52. SSPC-SP6-63, Commercial Blast Cleaning.
53. SSPC-SP8-63, Pickling.
54. SSPC-PA1-64, Shop, Field, and Maintenance Painting.
55. ASTM A 322-64A, Specification for Hot-Rolled Alloy Steel Bars.
56. ASTM A 29-64, Specification for General Requirements for Hot-Rolled and Cold-Finished Carbon and Alloy Steel Bars.
57. ASTM D 624-54, Methods of Test for Tear Resistance of Vulcanized Rubber.
58. ASTM D 676-59T, Method of Test for Indentation of Rubber by Means of a Durometer.
59. ASTM B 412-66T, Method of Tension Testing of Vulcanized Rubber.
60. ASTM D 573-53, Method of Test for Accelerated Aging of Vulcanized Rubber by the Oven Method.
61. ASTM D 395-61, Method of Test for Compression Set of Vulcanized Rubber.
62. ASTM D 746-64T, Method of Test for Brittleness Temperature of Plastics and Elastomers by Impact.
63. ASTM D 1149-64, Method of Test for Accelerated Ozone Cracking of Vulcanized Rubber.
64. ASTM D 471-66, Method of Test for Change in Properties of Elastometric Vulcanizates Resulting from Immersion in Liquids.
65. ASTM A 514-65, Specification for High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding.

66. ASTM A 441-66T, Specification for High-Strength Low Alloy Structural Manganese Vanadium Steel.
67. ASTM A 53-65, Specification for Welded and Seamless Steel Pipe.
68. ASTM A 435-65, Method and Specification for Ultrasonic Testing and Inspection of Steel Plates of Firebox and Higher Quality.
69. ASTM C 177-63, Method of Test for Thermal Conductivity of Materials by Means of the Guarded Hot Plate.
70. ASTM C 165-54, Method of Test for Compressive Strength of Performed Block-Type Thermal Insulation.
71. ASTM C 355-64, Methods of Test for Water Vapor Transmission of Thick Materials.
72. ASTM C 273-61, Method of Shear Test in Flatwise Plane of Flat Sandwich Constructions or Sandwich Cores.
73. ASTM D 1622-63, Method of Test of Apparent Density of Rigid Cellular Plastics.

The structural design also met the requirements established by the State Building Construction Code, State of New York, 1961.

#### 3.8.1.2.6 Code and Standards Steam Generator Replacement (Dome Opening Repairs)

The design, materials, fabrication, inspection, and testing of the Steam Generator Replacement dome opening repairs complied with the applicable parts of the following. (The latest revision of the code in effect at the time of construction is applicable.)

1. ACI 211.1, Standard Practice for Selecting Proportions of Normal, Heavyweight, and Mass Concrete.
2. ACI 301, Specification for Structural Concrete for Buildings.
3. ACI 304, Guide for Measuring, Mixing, Transporting and Placing Concrete.
4. ACI 305R, Hot Weather Concreting.
5. ACI 306R, Code Weather Concreting.
6. ACI 318, Building Code Requirements for Reinforced Concrete.
7. ASTM C 33, Standard Specification for Concrete Aggregates.
8. ASTM C 39, Compressive Strength of Cylindrical Concrete Specimens.
9. ASTM C 40, Standard Test Methods for Organic Impurities in Fine Aggregate for Concrete.
10. ASTM C 88, Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate.
11. ASTM C 94, Standard Specification for Ready-Mixed Concrete.
12. ASTM C 109, Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or 50-mm Cube Specimens).

13. ASTM C 117, Standard Test Method for Materials Finer than 75mm No. 200 Sieve in Mineral Aggregates by Washing.
14. ASTM C 123, Standard Test Method for Lightweight Pieces in Aggregates.
15. ASTM C 127, Standard Method of Test for Specific Gravity and Absorption of Coarse Aggregate.
16. ASTM C 128, Standard Method of Test for Specific Gravity and Absorption of Fine Aggregate.
17. ASTM C 131, Standard Test Method for Resistance to Degradation of Small-Size Aggregate by Abrasion and Impact in the Los Angeles Machine.
18. ASTM C 136, Standard Method for Sieve Analysis of Fine and Coarse Aggregates.
19. ASTM C 138, Standard Method of Test for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete.
20. ASTM C 142, Standard Test Method for Clay Lumps and Friable Particles in Aggregates.
21. ASTM C 143, Standard Test Method for Slump of Hydraulic Cement Concrete.
22. ASTM C 150, Standard Specification for Portland Cement.
23. ASTM C 151, Test Method for Autoclave Expansion of Portland Cement.
24. ASTM C 172, Standard Practice for Sampling Freshly Mixed Concrete.
25. ASTM C 173, Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method.
26. ASTM C 191, Test Method for Time of Setting of Hydraulic Cement by Vicat Needle.
27. ASTM C 192, Standard Method of Making and Curing Concrete Test Specimens in the Laboratory.
28. ASTM C 231, Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method.
29. ASTM C 260, Standard Specification for Air-Entraining Admixtures for Concrete.
30. ASTM C 289, Standard Test Method for Potential Reactivity of Aggregates (Chemical Method).
31. ASTM C 295, Recommended Practice for Petrographic Examination of Aggregates for Concrete.
32. ASTM C 311, Standard Test Methods for Sampling and Testing Fly Ash or Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete.
33. ASTM C 494, Standard Specification for Chemical Admixtures for Concrete.
34. ASTM C 535, Standard Test Method for Resistance to Degradation of Large-Size Aggregate by Abrasion and Impact in the Los Angeles Machine.
35. ASTM C 566, Standard Method of Test for Total Moisture Content of Aggregate by Drying.
36. ASTM C 586, Test Method for Potential Alkali Reactivity of Carbonate Rocks for Concrete Aggregates (Rock Cylinder Method).

37. ASTM C 617, Standard Method of Capping Cylindrical Concrete Specimens.
38. ASTM C 618, Standard Specification for Coral Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete.
39. ASTM E 4, Standard Methods of Verification of Testing Machines.
40. ASTM E 70, Standard Test Method for pH of Aqueous Solutions With the Glass Electrode.
41. Corps of Engineers - U.S. Army (CRD): CRD C 119, Method of Test for Flat and Elongated Particles in Coarse Aggregates.
42. National Ready-Mixed Concrete Association (NRMCA): Check List for Ready Mixed Concrete Production Facilities.
43. Occupational Safety and Health Administration (OSHA): Safety and Health Regulations for Construction.
44. American Association of State Highway Transportation Officials (AASHTO): T-26, Standard Method of Test for Quality of Water to be Used in Concrete.
45. New York State Department of Transportation (NYSDOT): Standard Specifications, Construction and Materials.

### 3.8.1.3 Seismic Design

#### 3.8.1.3.1 Initial Seismic Design

The containment is a Seismic Category I structure. It was originally analyzed as a single lumped mass cantilever beam system to determine its natural frequency. For the containment structure, the damping factor as a percent of critical damping was assumed to be 2.0%. The resultant load developed from the maximum horizontal response is distributed in a triangular manner with the base of the triangle at the top of the structure. The stress criteria for the containment and all reinforced concrete members in tension were as described in Section 3.8.1.2.3 based on the response to a ground motion of 0.08g acting in the vertical and horizontal planes simultaneously. Design of the containment was checked to ensure that the combined stresses resulting from gravity, incident, and seismic loadings based on the response to a ground motion of 0.20g acting in the vertical and horizontal planes simultaneously are within the stress limits described in Section 3.8.1.2.3.

The natural period of the first harmonic was determined using an analysis consisting (for horizontal motion) of a cantilever fixed at the base with the mass lumped at the centroid of the structure. Bending stiffness was established based on a Young's modulus of $4.1 \times 10^6$ psi and shear stiffness was established based on a shear modulus of $1.8 \times 10^6$ psi. No rotation of the foundation material was considered. The natural period of the first harmonic was calculated to be 0.22 sec for horizontal motion and 0.07 sec for vertical motion.

The resultant base shear was established on the basis of the maximum response acceleration (0.46g) for the maximum hypothetical design earthquake considering 2% of critical damping. The resultant load was conservatively assumed to be distributed in the form of an inverted triangle extending the full height of the structure. The resulting maximum meridional forces are as shown in Figure 3.8-9.

#### 3.8.1.3.2 Seismic Reanalysis

As a check on the initial seismic design of the containment it was reanalyzed using normal mode theory with a number of lumped masses. A check was also made on the containment considering the rock foundation as an elastic medium with rotation and translation of the containment considered. This flexible foundation modeling of the containment changed the total shear and overturning moment on the structure by less than 5% as compared to the rigid foundation model. The base shear for the modal analysis on a rigid foundation resulted in an equivalent containment response acceleration of 0.26g as compared to the 0.46g used in design. Comparable results were obtained with respect to overturning moments. As a result of the somewhat more rigorous modal analysis, the containment design can be shown to be highly conservative. A detailed description of the modal analysis follows:

A. The containment structure is modeled as a cantilever consisting of lumped masses connected by weightless springs. This model is shown in Figure 3.8-10.

B. The normal modes are calculated using the computer program SAND. This program is a modified version of a program developed by the Jet Propulsion Laboratory for the dynamic analysis of lumped mass systems. Shear deformations and rotational inertia are included in the program SAND.

C. The input required for SAND consisted of the modal coordinates, member properties, and material properties. These are shown in Figure 3.8-10 and Table 3.8-1. The masses are calculated by the program using a density of 160 $lb/ft^3$. This is representative of heavily reinforced concrete.

D. The response in each mode is read from the response curves determined for the site, as given in Section 3.7. The deflections, accelerations, and member forces are computed in each mode and are then summed on a square root of the sum of the squares basis. This computation is executed by the computer program SPECTA.

E. The natural frequencies and response are summarized in Table 3.8-1. The mode shapes are plotted in Figure 3.8-11 and the shear forces and bending moments in Figure 3.8-12.

F. The effects of ground motion were investigated by considering the rock as an elastic medium with coefficients similar to concrete:

$E = 3.0 \times 10^6$ psi

$r = 0.2$

The fundamental frequency was reduced from 6.95 Hz to 6.28 Hz. The alterations to the deflections, accelerations, shear forces, and moments were insignificant, being less than 5%.

### 3.8.1.4 Containment Detailed Design

#### 3.8.1.4.1 Stress Analysis

##### *3.8.1.4.1.1 Analysis Methods*

The analysis of the containment structure for operating plus incident load was based upon shell theory analogy.

The containment structure was analyzed for seismic loads as a cantilevered beam with all mass assumed concentrated at the center of gravity. Both shear as well as bending stiffness were considered in determining the fundamental frequency. The total horizontal inertial load was determined from the response curves given in Figures 3.7-1 and 3.7-2 for 2% damping for the fundamental frequency of the cantilevered beam. This total horizontal load was distributed over the height of the containment structure in the form of an inverted triangle to determine the inertial overturning moment. The vertical seismic component was assumed to be unamplified due to the high axial stiffness of the containment.

Stresses induced by the horizontal and vertical components of seismic motion were combined algebraically. The seismic shear distribution assumed was that given for a hollow thin-walled cylinder with shear flow perpendicular to the containment radius and the maximum shear flow equal to twice the average value.

### *3.8.1.4.1.2 Analysis Results*

The results of this analysis for the following loading combinations are shown in Figures 3.8-13 through 3.8-15.

a. Operating plus incident load.
b. Operating plus incident plus design earthquake loads.
c. Operating plus incident plus maximum potential earthquake.

The displacement resulting from the seismic excitation will produce a base shear which is transferred via the base mat to the side walls of the structure by the radial reinforcement. During an incident these bars should be in tension. As the lateral load (i.e., earthquake shear) is imposed, these bars will react similar to a wheel with prestressed spokes with a load applied to the hub and the rim restrained from moving. In this design these members are assumed to have no shear resistance. The load transfer from the radial bars, which established longitudinal shear stresses in the wall, will occur by means of varying circumferential membrane forces in the lower portion of the wall.

The side wall, at loads resulting from the factored pressure (1.5 P), will be uncracked in a horizontal plane due to membrane prestress forces. The only cracking that occurs will be partial cracking due to secondary flexure. The depth of these cracks will be limited by the mild steel reinforcement. At the design pressure there will then be sufficient uncracked section of concrete to limit radial shear stresses to less than the maximum allowable value stipulated in ACI 318-63. Details of the radial shear analysis are provided in Section 3.8.1.4.5.1.

The amount of prestressing force provided in the meridional direction of the cylinder is determined to ensure no resultant tensile stress due to the factored load combinations described in Section 3.8.1.2. Consequently, radial cracking is predicted to be only a result of flexure which is similar to the basis for the derivation of concrete shear capacity and shear reinforcement requirements stipulated in ACI 318-63 for flexural members. The derivation of shear reinforcement requirements at the base to cylinder discontinuity is described in Section 3.8.1.4.4.

The capacity to resist membrane shears is affected by the concrete cracking. Refer to Appendix 3B for the discussion of membrane shears in the vicinity of the large openings.

For the cylindrical portion of the vessel resistance to the vertical shears resulting from the earthquake, loading will be developed in the circumferential reinforcement by dowel action (*Reference 6*). The resulting principal stress in the reinforcement will not exceed 0.95 x yield stress as provided in the design criteria. This design further ensures no failure of the adjacent concrete in bearing. Details of the longitudinal shear analysis are provided more fully in Section 3.8.1.4.5.2.

In the dome, all membrane and shear stresses resulting from the earthquake loading will be developed in the mild steel reinforcing.

The loading on the concrete shell of the containment following an accident must be transmitted to it through the liner. The liner attempts to expand under the combined influence of the temperature and pressure. Since the containment structure may be classed as a thin shell, (the diameter to thickness ratio is 30), it is considered that it would have been valid to treat the temperature rise in the liner as an equivalent pressure increase.

The analysis as performed considered an equivalent liner force occurring at the location of the liner. Such equivalent liner forces were established based on no thermal strain relief at points where concrete is uncracked. Where the liner is insulated, the liner temperature increase was assumed to be 10°F due to accident conditions. Based upon no relief of thermal strains and uncracked concrete, the effect of this temperature rise was converted to an axial force plus a moment about the centroid of this insulated section. As a design conservatism, the elastic expansion of the concrete shell under pressure and temperature loads was not used to reduce the temperature induced stresses.

#### *3.8.1.4.1.3 Analysis for Steam Generator Replacement Dome Openings*

In order to support the 1996 Steam Generator Replacement, significant structural analyses were performed to support the creation and restoration of containment dome openings. A finite element approach was used for the structural analyses to determine the containment shell structure's capabilities to support applicable loads during and after the dome opening construction and restoration. The structural evaluation of the concrete shell of the dome was based on ACI 318-63 Code Part IV-B, Ultimate Strength Design. The structural evaluation of the liner system of the dome was based on the 1965 ASME Boiler and Pressure Vessel Code, Section III. These codes are consistent with original design and are shown in Section 3.8.1.2.3.3 and the table in Section 3.8.2.1.1.2.

### 3.8.1.4.2 Rock Anchors

#### *3.8.1.4.2.1 Rock Anchor Design*

The basic criterion for the determination of anchor length was that the pull of the anchor is resisted only by the submerged weight of rock and that the rock offers no tensile strength. This criterion further assumes that the rock breaks out at an angle of 45 degrees to the bond development length of the tendon. This criterion also allowed for any additional loads on the rock imposed from the inside of the containment vessel. The hold-down capability of the rock in the rock anchor design took into consideration the circular geometry of the vessel.

The design of the rock anchors was based upon the simplified assumption that the rock breaks out at an angle of 45 degrees to the axis of the tendon with the apex of the angle at mid-height of the first stage grout. This implies that the rock failure mode is one of diagonal tension. This assumption of a half-angle of 45 degrees for rock is supported by *References 7*, *8*, and *9*.

Further verification of the conservative nature of this assumption was demonstrated by the rock anchor tests described in Section 3.8.1.7.

The sockets for the rock anchors are percussion drilled into the rock through steel pipe sleeves which are welded into the underside of the bearing plates for the rock anchors and extended through the ring girder. The sockets in the rock plus the pipe sleeves are filled with a neat cement grout in two stages after the rock anchors are installed. Protective steel covers, as shown on Figure 3.8-1, are welded to the bearing plates for the rock anchors to enclose the sidewall tendon to rock anchor couplings. The tendon conduit extending above this enclosure is 6-in. diameter schedule 40 pipe with threaded couplings. This tendon conduit is threaded into a half coupling that is welded to the top of the protective steel cover. In order to permit the required conduit movement, stainless steel bellows are provided. The tendon conduit, including the protective steel cover, is bulk filled with the corrosion protection system described in Section 3.8.1.4.3.4. This filler material is injected through a connection in the protective steel cover. The exterior surface of the containment structure was waterproofed from the edge of the ring girder to elevation 253 ft 0 in. to provide corrosion protection.

#### *3.8.1.4.2.2 Preinstallation Grouting Test*

Prior to installing any rock anchors, a test was performed by grouting a rock anchor in a water filled, clear, 6-in. diameter tube. This rock anchor contained ninety 1/4-in. diameter wires with the grout tube and bottom hardware all identical to that proposed for the permanent installation. This test demonstrated that the grout did flow so as to completely encase the tendon. However, it also indicated that the use of bleeder holes near the bottom of the grout pipe, as well as the grout pipe terminating above the bottom of the hole, tended to produce an unacceptable dispersion of the grout. This condition was remedied by deleting the bleeder holes and extending the grout pipe with the addition of a bevel to the bottom of the hole. No tests could be made on the completeness of grouting of permanent rock anchors. However, procedures used for grouting did comply with those found to be satisfactory in the test.

The side wall tendons are coupled directly to the rock anchors. Lift-off readings were made on the side wall tendons that provide a measure of the prestress force at the fixed end (i.e., upper anchor head for the rock anchors). However, in the bonded tendon, it was not possible to measure the prestress in the full rock anchor tendon.

These criteria are identical with those used for dams in the United States and Europe. Confirming information was also obtained from The Cementation Company Limited of Great Britain, a specialty firm whose activity in recent years has been devoted, in large measure, to the prestressing of both existing and new dams, especially in South Africa and Australia.

### *3.8.1.4.2.3 Previous Applications*

Large capacity, post-tensioned anchors designed on this basis have previously been used in a number of dams in Europe, Africa, Australia, and the United States to provide stability for the structures. One of the early applications was the anchoring of the Cheurfas Dam in France 1935. Similarly, prestressed rock anchors have been used for tie backs on retaining walls on a permanent as well as temporary basis and for suspension bridge anchorages. Major structures for which prestressed rock anchors were used are listed in Table 3.8-2. A list of some major applications of the BBRV ninety 1/4-in. diameter wire prestressed rock anchor assemblies is given below.

- Wanapum Dam, Washington; Mayfield Dam, Washington: Rock anchors and trunnion anchors; rock anchors for penstock slope stabilization.
- Boundary Dam, California: Rock anchors for rock stabilization.
- John Hollis Bankhead Dam, Alabama: Rock anchors for dam stabilization.
- Ice Harbor Dam, Washington: Rock anchors.
- Mangla Dam, West Pakistan: Trunnion girder anchorage, main spillway.

The design is based upon the use of the BBRV system developed originally in Switzerland and used extensively for rock anchor applications.

### *3.8.1.4.2.4 Rock Hold-Down Capacity*

Laboratory tests on core representative of rock in the approximate area and depth of the rock anchor installation indicate a bulk specific gravity of the rock of 2.54. Since the rock participating with the rock anchors is below the ground water table, the submerged weight of rock of 96 lb/ft$^3$ (2.54-1.0) x 62.45) is used in determining the hold-down capability.

The bond development length (first stage grout) for the ninety 1/4-in. diameter wire tendons is computed as follows:

For 0.60 $f_u$ = 635 kips

$$P = \frac{\left(\frac{80}{60} \times 635000\right)}{\pi \times 6 \times 170 \times 12} = 22.0\,ft.$$

(Equation 3.8-3)

Each rock anchor was initially tensioned to 80% of ultimate strength and the jacking force was then reduced at lock-off to 70% of ultimate. The bond stress assumed between rock and grout is 170 psi. This value was determined to be conservative as demonstrated during the test performed on reduced scale rock anchors and also as reported by the Swiss Federal Laboratory for the Testing of Material (*Reference 10*) and as documented in Grolversuchemit Spannankern an Talsperran der Asterreichen Bunderbahnen und die Anwendung der Vorspannbouweise auf den Talsperrenban, Von A. Ruttner, Wien, Austrian Engineering Journal, 1964. Test data obtained for the John Hollis Bankhead Dam, Warrior River, Alabama, also confirm the conservatism of a bond development length developed on the basis of the average bond stress of 170 psi between grout and rock.

The diameter of the drilled hole for each rock anchor is 6 in. The assumed breakout angle of 45 degrees to the vertical is most conservative as demonstrated during the reduced scale rock anchor test and in *Reference 7*.

Weight of rock in kips/ft circumference = $0.096d^2$

$$\text{Internal pressure in kips/ft circumference} = \frac{0.072 \cdot pd(2r - d)}{r}$$

(Equation 3.8-4)

The depth, d = 26.5 ft, was established based on preliminary design. No surcharge beyond the internal pressure of the containment vessel was considered to be effective in determining the rock anchors hold-down capability. Therefore, for varying internal pressures the rock hold-down capacity uniform around the circumference of the vessel, was as follows:

| Rock Hold-Down Capacity (Kips per ft circumference) | Internal Pressure (psig) |
|---|---|
| 0 | 67.4 |
| 60 | 240.4 |
| 69 | 266.4 |
| 75 | 283.7 |
| 90 | 327.0 |

### *3.8.1.4.2.5 Hold-Down Factor of Safety*

For the combination of operating plus incident loads (i.e., load combination a) in Section 3.8.1.2.3, the uplift per foot circumference is constant at 259.0 kips/ft which is less than the assumed rock anchor capacity of 327.0 kips/ft. Therefore, the factor of safety on pull-out against the factored load is 1.26. For the structural proof test, uplift per foot circumference was constant at 182.0 kips/ft which was less than the rock anchor capacity of 266.4 kips/ft for a factor of safety of 1.47.

For the combination of operating plus incident plus design earthquake loads (i.e., load combination b), the maximum uplift per foot circumference is 274.1 kips/ft and the minimum is 150.5 kips/ft. This considers horizontal and vertical components of ground motion occurring simultaneously and their effects added algebraically. Due to the group action of anchors, the overcapacity of the rock against lateral loads can be represented by the factor of safety against overturning. This factor, using the rock hold-down capacity based on the pressure load of 75 psig, is 2.38.

For the combination of operating plus incident plus maximum potential earthquake loads (i.e., load combination c), the maximum uplift per foot circumference is 289.2 kips/ft and the minimum is 25.4 kips/ft. The factor of safety against overturning, using the same consideration, is 1.96.

Consideration was also given for seismic loading without internal pressure. For the 0.1g ground motion (vertical and horizontal components considered to occur simultaneously and the effects added algebraically) there is no uplift.

Minimum downward component is 0.9 kips/ft. The factor of safety against overturning is 4.62. For the 0.2g ground motion (vertical and horizontal components considered to occur simultaneously and the effects added algebraically) the maximum uplift is 69.2 kips/ft. The factor of safety against overturning is 2.31.

### *3.8.1.4.2.6 Installation*

The tendons are anchored into the rock socket with an expanding grout. The grout contained an additive designed to reduce the water requirement of the cement, to have a slightly expanding action, and to retard the initial set. The expansion based upon original grout volume is 8% ± 2%. This expansion is accomplished by the reaction of aluminum powder with the alkalies of the cements. This reaction results in liberation of hydrogen gas in the form of small bubbles which have an expanding effect. Tests have verified that the molecular form of the hydrogen in the alkaline medium will not adversely affect the steel.

The top (movable) anchor head for the rock anchor is coupled to the bottom (fixed) anchor head of the side wall tendon, as shown in the fully engaged position in Figure 3.8-16. Dimensions and material are as shown. The bushing provides for coupling the smaller diameter fixed head to the larger movable (i.e., tensioning) head. The coupling has right-hand threads on each end.

During construction, after the rock anchors were tensioned, the coupling was set in place on the top head of the rock anchor. When the sidewall tendon was inserted in the conduit, the coupling was threaded onto the bottom head of the sidewall tendon to the end of thread. The coupling was then turned down onto the top head of the rock anchor resulting in all threads on both anchor heads being fully engaged as shown in Figure 3.8-16. The design of the tendon hardware ensures that the hardware remains elastic up to the ultimate capacity of the wires. Therefore, at the effective prestress force of 60% of the ultimate strength of the tendon, average strains in the coupling are designed to be no greater than 60% of the yield strain of the coupling material.

### 3.8.1.4.3 Tendons

#### *3.8.1.4.3.1 General Design*

The design for the containment provides for prestressing the concrete in the cylinder walls in the longitudinal (vertical) direction with a sufficient compressive force to ensure that upon application of the design load combinations there will be no tensile stresses in the concrete due to membrane forces. In addition to the membrane stresses there are also flexural and shear stresses which result from discontinuity effects. On the basis of the design criteria, the concrete stresses and the stresses on the mild steel reinforcing upon application of the combined loads will then be produced by combined flexure and shear and/or compression. The structural elements are then acting in a manner similar to those tested as a basis for ACI 318-63 Chapter 17, Shear and Diagonal Tension - Ultimate Strength Design, and there is a basis for designing shear reinforcing.

The design also provides for anchoring the cylindrical walls to rock with anchors which will be post-tensioned tendons anchored into grouted sockets in the rock. The anchors are designed to resist all membrane stresses in the cylindrical wall. A sufficient physical separation is provided between wall and base slab to ensure that there is no transfer of vertical reaction to the base slab.

In order to produce minimum practical base restraint and to most effectively use the rock anchors (i.e., no moment applied to the ring girder), the design provides for the development of a hinge at the cylinder to base transition using an elastomer pad. The elastomer pad permits a predictable rotation of the hinge with the only restraint to rotation being a minimal resistance due to compression on the pad. The elastomer (neoprene) pad was selected for the hinge because of its predictability of behavior, maintenance-free properties, and ability to withstand environmental conditions far more severe than that associated with the Ginna design. Detailed background data on the use of neoprene bearing pads is included in Section 3.8.1.4.4.3.

Under the dead load of the containment and the application of the prestress force, the elastomer pad will compress vertically approximately 0.08 in. Upon being subjected to the most severe loading combination, the tendon elongates and the pad reverts back to essentially its original thickness (i.e., pre-stress force equals or is slightly greater than membrane forces due to this loading combination). This elongation must extend over a sufficient length of tendon to ensure no yielding of the steel. In an effort to minimize the increase in wire stresses under load, the tendon is unbonded for the entire length from coupling between rock anchor and anchorage of tendon at the top of the side wall.

A large amount of vertical reinforcement is provided near the outer surface of the wall at the lower elevations. This steel is provided to resist bending moments which occur in the wall due to the base restraint. Mild steel reinforcement provided for flexure is shown in Figures 3.8-4 and 3.8-5. Since the wall has a steel liner on the inside, the minimum mild steel reinforcement required for crack control has been provided on the outside only, in the amount of 0.19% of the concrete cross-sectional area. The prestressing tendon is positioned at the center of the wall section, thus causing the participation of the prestress force to be minimal in resisting bending moments. The design requires all bending or shear stresses to be resisted by mild steel reinforcement, thus making the design quite conventional in the region of bending and shear.

Due to the initial tendon force (0.6 $f'_s$) the maximum average concrete membrane (meridional) stress is 640 psi compression and the liner (meridional) stress is 4500 psi compression. Considering a concrete creep and shrinkage of 320 x $10^{-6}$ in./in., the final average concrete membrane (meridional) stress is 550 psi compression and the liner (meridional) stress is 14,100 psi compression. This implies that a linear temperature gradient of 39°F through the concrete shell (i.e., a temperature on the liner side 39°F below the exterior fiber temperature) would result in a zero stress on the inner fiber. This situation is not considered credible.
During MODE 6 (Refueling), the refueling and purge system which has no cooling coils, could not reduce the interior temperature below the external ambient temperature. The containment recirculation fan coolers (CRFC) could possibly reduce the internal ambient temperature but not to the extent required to exceed the foregoing gradient. Therefore a reversal of stresses is not possible and no concern exists regarding crack control on the inner face. As noted above, a minimum mild steel reinforcement has been provided on the outside face in the amount of 0.19% of the concrete cross-sectional area. This amount exceeds the frequently used minimum amount of steel for crack control of 0.15%. The structure has liner insulation (except for a region of the dome) and will consequently not be subject to rapid temperature changes due to fluctuations in the interior ambient temperature.

The sole purpose of prestress is to balance vertical tensile membrane forces in the wall thus allowing confidence in the use of the provisions of ACI 318, Section 1701 and 1702, for shear reinforcement design. Therefore, the prestressing requirements would be those of a tension member rather than of a bending member.

All side wall tendons, can be removed or retensioned. Two tendons are permanently accessible for either operation, while the remainder can be reached by removing concrete at approximately elevation 228 ft (see Figure 3.8-2) to obtain access to the coupling enclosure. Any tendon can be uncoupled from the rock anchor for removal by opening a window in the coupling enclosure.

The two permanently accessible tendons are located on the south side of the containment vessel, and have the coupling enclosure exposed in the auxiliary building sump (Figure 3.8-2). A bolted door on the coupling enclosure permits removal and inspection of the tendons without removing concrete.

A failure of an unbonded tendon or tendons in the upper portion of the wall would result in a loss of prestress in the section of the wall subjected to bending and shear; however extensive failures of this type would cause a tensile failure in the wall, thus making a secondary shear failure at the base of little consequence.

#### *3.8.1.4.3.2 Seismic Considerations*

In evaluating the relative safety of a tendon for a prestressed concrete structure subject to seismic loads, consideration was given to the stresses in the tendon (the ninety 1/4-in. diameter wires) and to the tendon anchorage.

The design for Ginna is based upon a dynamic analysis using a basic ground acceleration of 0.2g. The design does not consider the ultimate strength and plastic deformation of the structure but considers only an elastic response with damping selected on the basis of such a response.

Other considerations that are generally recommended for seismic design and are incorporated in the design are (1) to provide a symmetrical structure thereby avoiding the torsional effect produced by structure rigidity and (2) include sufficient rattle space between the containment shell and adjacent structures, including the structures within the containment, to avoid any possible physical interaction as the structures deflect independently under the seismic load.

By using unbonded tendons, high local strains or elongations can be distributed over the length of the tendons. Another problem is the control of cracking in the concrete. In *Reference 1*, T. C. Waters and N. T. Barrett state that an adequate amount of bonded reinforcement or the bonding of a portion of the prestressing tendons will ensure that cracking of the concrete is uniformly distributed and that concentrations of large local tensile strains at particular points will be avoided. In the Ginna design where cracking might occur due to flexure produced by discontinuities, bonded mild steel reinforcement is used to control crack spacing and width. Where flexural stresses are minimal, bonded mild steel reinforcement is also provided to control the spacing and width of cracks, thereby serving to increase the ultimate capacity of the structure. The concrete containment is not susceptible to a low temperature brittle fracture. This conclusion is consistent with information provided in the First Supplement to the PSAR.

For a flexural member, there may be merit in localizing a wire failure in that the loss of pretress force might not extend over a region where maximum flexural capacity is required. This would be especially true for a failure at or near an anchorage. However, the design provides for prestressing tension, not flexural, members and there is no similar advantage in localizing the failure of a tendon in a tension member.

The behavior of the anchorage hardware is of prime importance when the element is subjected to reversal of loading produced by the dynamic loads from an earthquake. The anchorage system for this design, the BBRV (buttonhead) system, was chosen because of its positive anchorage and excellent properties when subjected to cyclic loadings. The BBRV system used parallel wires with cold formed buttonheads at the ends which bear upon a perforated steel anchor head, thus providing a positive mechanical means for transferring the prestress force. The buttonheads on the wire are formed by cold upsetting to a nominal diameter of 3/8 in. on the 1/4 in. diameter wire. Professor Fritz Leonhardt (*Reference 11*) reports that "Extensive tests show that this BBRV 'buttonhead' provides a reliable anchorage, even under dynamic loading conditions, if an anchor of softer steel (ST 52 to ST 90), provided with an appropriate bore (opening for wire) is employed." The anchor heads for the Ginna design are fabricated from C1141 steel, which is a softer steel than the wire heads approximately equivalent to ST 70 covered under the German Specification DIN 17 100.

Fatigue tests were conducted by the Swiss Federal Testing Station (EMPA) in 1960 on individual 7-mm wires with upset heads and on tendons consisting of eighteen 7-mm wires each. The anchorage heads for the tendons were for 22-wire units but the number of wires was limited by the capacity of the testing apparatus. The tests on individual wires indicate that 7-mm wire with upset heads is capable of sustaining 2,000,000 stress application cycles with an upper limit of about 1301 $kg/mm^2$ (180 ksi) when the lower limit is 95 $kg/mm^2$ (135 ksi). Several tests were conducted on the 18-wire tendons. The results of the one test with stress limits most similar to that used for design of prestressed concrete are summarized below.

With a lower limit of 95 $kg/mm^2$ (135 ksi), the tendon withstood over 2,040,000 stress application cycles to an upper limit of 111 $kg/mm^2$ (158 ksi) without any of the wires fracturing. Only after the upper limit was raised to 113 $kg/mm^2$ (160 ksi), did one of the wires break after an additional 113,000 stress application cycles. The rate of stress applications was 350 cycles per minute.

Cutting tolerance for the test tendon was plus or minus 0.5 mm. The ratio of tolerance to total wire length for the test tendon is 1/2377, which compares with 1/3210 for the rock anchors and 1/4800 for the side wall tendons. The ultimate strength of the wire being tested was 160 $kg/mm^2$ (225 ksi).

Therefore, it is concluded that dynamic loads, considering especially pulsating loads resulting from an earthquake, do not jeopardize the buttonhead anchorage.

The tendon bearing plates are 18.5 in. in diameter with a 5.5 in. center hole. Considering uniform bearing, the concrete bearing pressure due to the initial tendon force (742 kips) is 3040 psi. This compares with an allowable stress (ACI 318-63, Equation 26-1) of 3720 psi. The maximum splitting force (*Reference 11*) due to the initial tendon force considering no concrete tension is 58.0 kips, based upon tension extending from 6 in. to 30 in. below the bearing plate. The required reinforcing is 1.45 $in.^2/ft$ compared with the furnished 5/8-in. diameter spiral at 2-in. pitch with an area of 1.86 $in.^2/ft$. The calculated spalling force (*Reference 12*) is 22.2 kips/tendon for which No. 7 reinforcing bars were provided at 12.75-in. centers.

Bond development of the spiral reinforcement is not considered relevant. The reinforcement for spalling is anchored in excess of ACI 318-63 requirements. Experience indicates that long-term loadings will not degrade the integrity of the anchorage zone.

A Seismic Committee was established by the Prestressed Concrete Institute to develop guidelines for the design of prestressed concrete structures for seismic loads. In their report (*Reference 13*), the Prestressed Concrete Institute provides detailed guidelines for the design of prestressed concrete structures for seismic loads. These guidelines apply to bonded and unbonded tendons. The Ginna design has been reviewed in light of this report and has been found to comply with all guidelines.

##### *3.8.1.4.3.3 Stressing Procedure*

Stressing of tendons is accomplished by hydraulic jacks and pumping units which are equipped with dial gauges that indicate the pressure in the system within plus or minus 2%. The stressing procedure is as follows:

a. Stress by pumping until the required overstressing force is reached with backup provided by direct measurement of differential displacement of the tendon head and bearing plate made to the nearest 1/32 of an inch.

b. Insert shims, filling the space as completely as possible.

c. Reduce pressure to seat the anchor head on the shims.

d. Take lift-off reading and record.

e. Adjust shims as necessary.

The pattern and sequence of post-tensioning was established so as to provide basically for initially tensioning every 40th tendon of the total 160 tendons and then in a systematic manner to tension the tendons approximately midway between previously tensioned tendons. This approach minimizes the loss due to elastic shortening. The elongation of the side wall tendons during the stressing operation was approximately 8 in.

The philosophy behind this sequence of post-tensioning was as follows:

aa. To provide in each stage of stressing an essential symmetric loading on the containment cylindrical wall and neoprene pad at the base.

bb. The prestress load was to be applied as far as practical symmetrically with respect to the two large access openings.

cc. The curved tendons around the large access openings were to be retensioned after 1000 hours in order to counteract the time dependent losses due to shrinkage, creep and steel relaxation. The retensioning was required in order to fulfill minimum prestress requirements up to the end of plant life, which is 40 years.

The highest tendon stresses occur during the jacking operation which, in effect, pretests the tendon including all hardware prior to the application of a pressure load. The effective prestress considering all losses (i.e., 60% of ultimate stress) is 144,000 psi. Upon subjecting a tendon to the most severe loading combination (design-basis accident plus maximum earthquake), the tendon stress increases by 4.6%, i.e., 6,600 psi.

The effective prestress forces were developed in all tendons in accordance with normal industry practice. All tendons were initially tensioned to 80% of ultimate stress and then locked-off at 70% of ultimate stress. Basically all tendons are straight. A limited number have a minor curvature where they are draped around small penetrations. The tendons in all cases are located in a relatively large (6-in. diameter) rigid conduit which was sized to permit the bottom anchor head to pass through. Any wobble and friction losses will be less than 24,000 psi or 10% of the ultimate stress. The remaining losses consist of elastic shortening, concrete shrinkage and creep, creep of the elastomer pads, and steel relaxation. Anchorage losses are negligible for the length of tendon being used. The tendons are protected to ensure that there are no loss of wires due to corrosion.

The tendon temperature never sufficiently exceeds that resulting from plant operation and high ambient temperatures external to the containment. The average daily temperature of the tendon will, therefore, never exceed approximately 90°F.

The prestressing sequence for the rock anchors was generally as follows:

i. Initially, every fourth anchor was tensioned. Horizontal spacing of anchors, as shown in Figure 3.8-2 is 2 ft 1 9/16 in.
ii. Secondly, every second tendon not included in item 1 was tensioned.
iii. Finally, all remaining anchors were tensioned.

The tensioning of side wall tendons was done using a minimum of four jacks spaced generally about the circumference of the structure. Stressing positions were alternated to prevent concentrations of multiple stressed tendons adjacent to multiple unstressed tendons. This was accomplished by tensioning tendons in a sequence wherein the tensioned tendon was approximately equidistant between previously tensioned tendons. The four jacks were used so that the resultant of the prestress force remains approximately symmetrical around the circumference of the structure.

### *3.8.1.4.3.4 Corrosion Protection*

A steel conduit (6-in. diameter Schedule 40 pipe) is embedded in the side wall concrete to permit insertion of the prestressing steel tendon and in addition provide electrical shielding against stray ground currents. The conduit is specially designed where it passes through the elastomer pads so as not to jeopardize the action of the hinge by using a bellows-type expansion joint.

The 6-in. $\phi$ threaded pipe is screwed on to a 6-in. $\phi$ half coupling. This connection meets the criteria specified in the standard code for Power Piping, USAS B31.1.0 - 1967, and as such provides a leak-proof joint.

The wire was protected prior to fabrication to ensure that the surface was free from any imperfections other than a light oxide film. Prior to shipment, the tendon was protected with a coating of NO-OX-ID "490," manufactured by the Dearborn Chemical Division of W. R. Grace and Company. The NO-OX-ID "490" provides a light coating satisfactory for temporary protection. Following insertion of the tendons in the conduit, the conduit was filled with NO-OX-ID "CM" so as to provide bulk filling of the void in the conduit. An expansion reservoir is provided at the top anchorage as shown on Figure 3.8-7. Access to this reservoir is provided as shown on Figure 3.8-17. The tendon conduit is filled by pumping the NO-OX-ID "CM" in at the level of the tendon coupling and venting from the top anchorage.

The water table is approximately 16 ft above the bottom of the tendons. The tendon and its conduit are approximately 110 ft high. This leaves a hydraulic head of filler material of 94 ft which is equivalent to about 36 psi above the highest point of the water table. This ensures that there is no water seepage into the conduit.

This is an underestimation of the pressure required to displace the filler material in that it is based upon the material having the viscosity of water with no friction loss and a specific gravity of 0.9. During the actual placement of the filler material, a pressure of 42 to 45 psi was required to pump the material after it was agitated.

The radial tension bars, as shown on Figure 3.8-2, are protected against corrosion as follows:

a. In the cylinder wall the bars are coated with grease. (The grease ensures there is no bond development).
b. In the base slab, the bars are inserted in a pipe sleeve for a length of 2 ft 10 in. The annular space between bar and pipe sleeve is filled with the corrosion protection system described above for the side wall tendons.
c. The remaining length of the bars in the base slab are in intimate contact with the concrete.

The buttonheads at the rock anchor heads are encased in grout to provide continuity of environment along the full length of the wire. The movable (top) anchor heads for the side wall tendons are protected by covering the head with the NO-OX-ID "CM" made to prevent rain water from entering the conduit by the expansion reservoir. The top anchor heads can be inspected for corrosion by unbolting the cover on the expansion reservoir shown on Figure 3.8-7 and removing the wax covering the heads. The wax can also be sampled by this method.

NO-OX-ID "CM" casing filler is composed essentially of a selected paraffin-base refined mineral oil, blended with a microcrystalline petroleum-derived base (petrolatum) of definite melting point and penetration range. Additives consisting of lanolin, and sodium petroleum sulphonates are incorporated as water-displacing surface-active agents and corrosion inhibitors. The proportion of oil to microcrystalline wax in the formulation is adjusted to give a pour or gelling point within the range of 110 to 120°F. The oil and wax are highly refined long-chain saturated paraffinic petroleum derivatives, resistant to oxidation and chemical or physical degradation, within the temperature ranges to which they will be exposed in this service. The lanolin is a polar substance which enhances inhibitor performance and wetting of the metal surface by the microwax blend. The petroleum sulphonate is a surface-active, water displacing corrosion inhibitor of long tested merit. (See Table 3.8-3.)

**Quality Control Tests**

Quality control determinations on required raw materials and on the finished NO-OX-ID "CM" protective coating included those tests already being done in the standard raw material inspection procedures, plus additional controls requested on chloride, sulfide, and nitrate content. The latter tests included the following:

aa. Chlorides - The initial screening test on both raw materials and finished produce was the sensitive Beilstein Test. This is a flame determination using an oxidized copper carrier.

A green or blue-green color appears in the flame if chlorides (halides) are present. This test detects as little as 0.5 ppm halide. If a positive Beilstein indication is obtained, a confirming test is made on water extracts of the product, using standard titration or colorimetric procedures described in ASTM D-512-62T. (Note: A positive Beilstein test may be obtained when halides are not present, because of interferences from traces of pyridines, thiourea, thiocyanate, etc. This is the reason for a confirming titration on water extracts, following a positive Beilstein indication).

bb. Sulfides - The method used was a water extraction followed by a total sulfide determination. Zinc acetate was added to the extraction water to precipitate sulfides. Sulfides present were then measured in accordance with Paragraph 8 of ASTM D-1255. This method detects as little as 0.1 ppm sulfide. An alternate colorimetric procedure also was available in which sulfides are volatilized from an acidified extraction solution, to create a colored spot on zinc acetate paper. Spot intensity is measured to determine sulfide. The extraction procedure is described in ASTM D-1255.

cc. Nitrates - The method used was a water extraction followed by colorimetric measurement, based on ASTM D-992-52. Either the Brucine or phenoldisulfonic acid procedures were used. Either can detect as little as 0.01 mg/l nitrate.

dd. Cathodic Protection - All of the tendons are connected to the liner of the containment and then to the copper grounding system. Also, electrically connected to the grounding system is the mild steel reinforcement below the high ground water level. Permanent and stable potential reference cells are installed at significant locations to measure the corrosion potential.

At the time of containment construction, Durichlor anodes were installed around the perimeter of the vessel. Protective current can be applied from these anodes and regulated as needed to maintain a protective potential if cathodic protection is found necessary by measurements from the reference cells.

In addition, sacrificial steel cable has been installed next to all bare copper cables. Also, four potential bridge pipe test stations were installed on the rock anchor system to measure the magnitude of the earth potential current gradient caused by current flow into or out of the rock anchors and to provide a basis for regulating any applied current from the anodes.

A British study of the problem indicates that cold drawn perlitic wire of the type employed in the Ginna containment is not susceptible to stress corrosion cracking failure (*Reference 14*).

#### 3.8.1.4.4 Hinge Design

##### *3.8.1.4.4.1 Tension Bars*

A hinge was developed at the base of the cylinder wall by supporting the wall vertically on a series of elastomer bearing pads and anchoring the wall horizontally into the base mat with radially positioned, high-strength steel bars. The bars are approximately 20 ft long and 1-3/8 in. in diameter with two anchor plates and with a minimum ultimate tensile strength of 145,000 psi and a yield strength of 130,000 psi. The bars conform to ASTM A322-64a and ASTM A29-64 and are spaced approximately 1 ft-1 in. on centers at the centerline of the sidewall. The anchor plates conform to AISI C-1040 and can develop 100% of the bars' ultimate strength. The bars are unbonded over a predetermined length to provide for an elongation of the bar under load consistent with that required for the rotation of the wall with the elastomer pad acting as a hinge. The only rotational restraint on the base of the wall is that produced by the resistance of the elastomer pads to deformation. Actual tension bar stresses resulting from the factored loads are as follows:

| Loading Combination | Bar Force, kips | Bar Stress, ksi | % Yield Strength |
|---|---|---|---|
| A | 130 | 87.5 | 67 |
| B | 149 | 100.0 | 77 |
| C | 170 | 114.3 | 88 |

The effect of the base to cylinder discontinuity is based upon equations developed for the analogy of a semi-infinite beam on an elastic foundation (*References 15* and *16*) in which the spring constant for the circumferential bars and liner is taken as the foundation modulus. As such, the hoop stiffness is generated independently of the concrete. The elastic modulus of the uncracked concrete is assumed to be equal to

$$4.1 \times 10^6 \; psi \; (E = \omega^{1.5} \sqrt{33} f'_c \; from \; ACI \;\; 318\text{-}63)$$

(Equation 3.8-5)

The assumption on a single elastic modulus, which is considered to be a reasonable upper limit, is conservative in that it results in the highest discontinuity stresses.

Except for participation in anchoring the radial tension bars at the base of the cylinder, the base slab is not an integral part of the containment shell for this design. The loads on this slab, which is more properly described as a cap on the rock, are those from the interior structures.

A simple check was made, based on an assumed 45-degree bearing distribution, to ensure that rock bearing pressures do not exceed the limits listed in Appendix 2C.

The means for transferring the radial reaction at the base of the cylinder into the foundation rock is shown in Section 1-1 of Figure 3.8-2. The base reaction is transferred from the radial dowels into the ring girder and thickened portion of the base slab and thence, as a lateral load, on the rock outboard of the ring girder. The concrete for the ring is placed directly against the rock. The load is transferred to the rock on the interface from elevation 231 ft 8 in. to elevation 224 ft 8 in. The maximum allowed lateral pressure is 25,000 pounds per square foot as stipulated in Appendix 2C. Where no lateral rock support is available at the auxiliary building sump, a special beam and struts are required to span this area as shown in Sections 2-2 and 3-3 of Figure 3.8-2.

The details of the expansion joint in the tendon conduit at the hinge are shown in Figure 3.8-18. This is a stainless steel bellows as conventionally used on process piping for expansion joints. The bellows are 6-in. diameter, stainless steel pipe bellows complying with requirements of the ASA B31.1 Code for Pressure Piping. They provide movement capability for the rigid tendon conduit at the hinged joint to ensure a sealed tendon enclosure which retains the grease corrosion protection around the tendon and also seals against contaminants gaining access to the tendons. The bellows also provide essentially no resistance across the hinged joint to the movements. The inside diameter of the bellows is approximately 5.6 in. and the diameter of the tendon bundle is approximately 3 in. With 1.3 in. clearance and maximum predicted horizontal movement at design loads of 0.2 in., margin is available to preclude contact.

#### *3.8.1.4.4.2 Liner Knuckle*

The liner design at the hinge provides for a base to cylinder transition in the form of a knuckle with a 10-in. radius. This detail provides sufficient flexibility as the sidewall moves with respect to the base during the tensioning of the sidewall tendons and under the application of the design loads.

The stresses in the liner base to sidewall transition knuckle have been determined for the following cases. The analysis was based on the method described in *Reference 17*.

a. Under the application of the prestress force plus the dead weight of the vessel, the sidewall moves vertically downward 0.08 in. with respect to the base. The maximum bending stress in the knuckle due to this motion is 25 ksi.

b. Under loading combination a, the sidewall moves vertically upward 0.08 in. with respect to the base, and radially outward 0.08 in. The maximum bending stress in the knuckle following the movement is 10 ksi and membrane stress is 1.2 ksi. This loading combination represents the most severe loading on the knuckle.

The calculated stress for the tension bars at the base of the cylinder listed in Section 3.8.1.4.4.1 were based upon the assumption that the stiffness of the base is a function only of the tension bars. A study was made to validate this assumption. It was found that the liner knuckle offers negligible restraint to radial motions but does offer very significant restraint to lateral (horizontal earthquake) motions.

The dimensions of the liner knuckle are shown on Figure 3.8-19. The method of solution involved the use of a shell computer program based on the solution described in *Reference 17*, wherein stresses were determined on the basis of a lateral translation of Point A (Refer to Figure 3.8-19). It was conservatively assumed that the support lines for the knuckle remain circular. For the lateral motion the calculated spring constant of the knuckle is 785,000 k/in. Based upon a maximum earthquake shear force at the base of the cylinder of 12,080 k it is determined that the maximum shear stress in the knuckle is 16.4 ksi. Bending stresses are small.

#### *3.8.1.4.4.3 Elastomer Bearing Pads*

Each bearing pad is a flat pad 1.628-in. thick, made of two layers of 55 durometer hardness neoprene between three steel shims. The outer shims are 16 gauge and the middle shim is 10 gauge carbon steel.

The pads are placed between the cylinder walls and the ring beam. Because of the ability of neoprene to deform, it provides an effective medium of load transfer. By conforming to surface irregularities uniform bearing is provided. No lubrication or cleaning is necessary for the bearing. The pad dimensions are 9 in. x 42 in. and two pads were placed between each pair of pre-stressing tendons.

Each pair of pads will carry a maximum load of 371 tons resulting in a bearing pressure of 980 psi. This pressure is reduced to 840 psi after prestress losses occur. Both pressures are well within allowable values. A pad under load should not exceed a vertical deflection greater than 15% of the thickness. The steel shims being used reduce the calculated strain to 5.2, as further verified by the tests reported in Section 3.8.1.7.1. The creep of neoprene pads is dependent on the hardness of the neoprene which was the reason for using low hardness (55 durometer) pads. Creep as verified by tests is estimated to be 13% of initial deflection.

On most of the circumference of the containment, the elastomer pads are accessible or could be made accessible by removing insulation to view from one side.

Specifications for the elastomer pads are summarized in Section 3.8.1.6.6.

Neoprene pads have been in use since 1932 so that the practice at the time of the Ginna containment design was based on over 30 years of experience and research. These pads were first used in France in the late 1940s as the load transfer bearings between piers and beams. In the United States and Canada, development more or less paralleled the use of precast, pre-stressed concrete beams because of the problem of seating such beams. By 1957, concrete bridges had been built with neoprene bearings in Texas, New Hampshire, Rhode Island, and Ontario. At the time of the Ginna containment design, thousands of bridges and buildings throughout the world have been built using neoprene bearing pads.

The neoprene pads will have a local effect on seismic shears at the base. This effect however is comparable to Saint-Venant effects which are present locally at any discontinuity. The seismic design of containment for shear and moment loads as a cantilever beam is not affected by the neoprene pads since the cylindrical shell is tied to the base by means of the vertical pre-stressing.

The effect of vertical cracking of the containment shell under pressure loading will tend to reduce the stiffness of the containment which in turn, for the modal analysis discussed in Section 3.8.1.3, will increase the period and response of the structure. However this same cracking will tend also to increase structural damping and thereby reduce the structural response. Considering the large design margin contained in the actual seismic design of the containment as compared to that dictated by the more rigorous modal analysis presented in Section 3.8.1.3, the local perturbations caused by use of neoprene pads are not sufficient to affect design adequacy.

A typical properties specification for bridge bearing pads (the hardness Shore A 50 approximately applying to the pads to be used for the containment) is given by the American Association of State Highway Officials as follows:

| **Original Physical Properties** | | | |
|---|---|---|---|
| Hardness Shore A | 50 ± 5 | 60 ± 5 | 70 ± 5 |
| Tensile, minimum psi | 2500 | 2500 | 2500 |
| Elongation at break, minimum %(ASTM D-412) | 400 | 350 | 300 |
| Ozone, 1 ppm in air by volume, 20% strain, 100 ± 2°F, 100 hours | No cracks | No cracks | No cracks |
| Compression set 22 hours at 158°F, maximum % | 25 | 25 | 25 |
| Oven Aged 70 hours at 212°F | | | |
| Hardness pts. change maximum | 0 to ±15 | 0 to ±15 | 0 to ±15 |
| Tensile, % change maximum | ±15 | ±15 | ±15 |
| Elongation, % change maximum | -40 | -40 | -40 |
| Low Temperature Stiffness at -40°F | | | |
| Young Modulus, maximum psi | 10,000 | 10,000 | 10,000 |
| Tear. Die C lb/ in minimum | 225 | 225 | 225 |

### 3.8.1.4.5 Concrete

#### *3.8.1.4.5.1 Radial Shear*

The maximum value of radial shear is 253 psi and this occurs 3 ft above the highest stressed radial tension bar under the combination of operating incident and maximum credible earthquake loads (load combination c). The critical section for shear is taken 3 ft above the radial tension bar level to conform with the requirements of ACI 318, Section 1701. The ultimate shear capacity of the reinforced wall without shear reinforcement as defined in ACI 318 1701 is 126 psi. Shear reinforcement is required and is provided according to the requirements of Section 1702 as No. 7 bars at 11-in. centers. Thus, under the conditions of 60 psi internal pressure and 0.2g simultaneous earthquake (load combination c), the shear capacity of the containment wall is sufficient to resist the maximum shear stress which occurs at only one position on the circumference.

Under the combination of operating and incident loads (load combination a) the maximum shear stress which occurs uniformly around the wall is 183 psi, which is 78% of the ACI design code capacity of 253 psi. Under the combination of operating, incident, and design earthquake loads (load combination b), the maximum shear stress occurring at one point in the containment wall is 222 psi, which is 88% of the design capacity.

The detailed analysis for shear design under load combination 3 is as follows:

The ultimate shear capacity of the wall is

$v_c = \phi\ 1.9\ f'_c + 2500\ (P_w\ V_d/M) = 126$ psi

The actual maximum shear stress is

$v = 109000/(12 \times 36) = 253$ psi

whence the shear carried by stirrups is 127 psi.

Placing stirrups at 11-in. centers, the required cross-sectional area of bar using 0.85 yield stress is:

$$A_v = \frac{V'_u\ S}{\phi \cdot f_y \cdot d} = \frac{127 \times 36 \times 12 \times 11}{0.85 \times 40,000 \times 36} = 0.494 in^2$$

(Equation 3.8-6)

No. 7 bars having an area of 0.60 in.$^2$ per bar are therefore placed at 11-in. centers.

#### *3.8.1.4.5.2 Longitudinal Shears*

Under the combination of loads resulting from the simultaneous occurrence of maximum earthquake and loss-of-coolant accident, the internal pressure of 60 psi will produce vertical cracks in the cylindrical wall (maximum concrete tensile stress would be 970 psi). The capacity of the wall to resist longitudinal shears across these cracks due to the seismic loads with internal pressures is developed by the dowel action of the circumferential reinforcement.

In determining the capacity of the circumferential reinforcing bars as dowels, first the capacity of the concrete in bearing is checked and then the capacity of the bars in combined tension and shear is checked.

The concrete strength is calculated to limit the capacity to transfer shear to a dowel capacity of 38.7 kips per bar or an average shear stress of 9.7 ksi in the reinforcing bar (*Reference 6*).

In considering the strength of the reinforcing to resist shear stresses due to the dowel action and to resist tensile stresses due to the pressure load, the Mohr circle method is used to combine stresses. It is recognized that the failure mode of mild steel is one of shear. The strength envelope on the Mohr circle is a straight line parallel to the normal stresses axis at a shear stress magnitude of 19.0 psi (1/2 x 0.95 yield stress). The tabulation in Table 3.8-4, broken down as to factored load combinations, shows the allowable shear stress for a given tensile stress (due to pressure load) and the allowable tensile stress for a given longitudinal shear stress (due to lateral seismic load).

As indicated in Table 3.8-4, in every case where there is dowel action there is a margin of safety on the shear capacity of the reinforcing steel. In all cases, however, the capacity of the bar in shear is limited by the concrete in bearing and not by the steel in combined shear and tension. It should also be noted that this analysis considers only the outer ring of circumferential reinforcement for which the tensile stress is maximum.

This entire analysis is developed on the capability of the circumferential reinforcement to resist longitudinal shears with no reliance placed upon the liner capability or aggregate interlock. It is recognized that the longitudinal shear will be resisted by the interaction of dowels and liner but that the composite action will not jeopardize the integrity of the liner.

#### *3.8.1.4.5.3 Horizontal Shear*

The horizontal shear due to lateral seismic load is transferred to the cylindrical wall of the containment through the horizontal radial tension bars provided at the base. The bars act in a manner analogous to spokes of a wheel in transferring shear.

The forces in the bars have been analyzed by assuming the wall to be a stiff ring. This analysis gives an overestimate of bar force and leads to a conservative radial bar design. However, a wall section acting as a horizontal ring at the base of the vessel must also be checked as a ring for bending and shear stresses that result from differential radial tension bar forces. The worst condition for this effect will occur with 0.2g earthquake resulting in a maximum differential force between any one bar and the adjacent one of 1.55 kips. This force differential produces a moment and shear in the wall section (considering a one foot height of ring) of 1.66 kips-ft and 1.55 kips respectively. From the circumferential bar layout in the region of the wall adjacent to the radial bars this moment and shear will be resisted by a minimum of four 18S bars.

Assuming a totally cracked wall section in this region (which is not the case as circumferential hoop tensions are very small in this region) the capacity of these four 18S bars in shear is 155 kips compared to the calculated shear of 1.55 kips and in bending is 303 kips-ft compared to a computed moment of 1.66 kips-ft. Thus the wall has more than adequate capacity to resist the small moments and shears produced by any radial force differentials in tension bars.

There are two general types of bond failure. ACI 318-63 addresses the most common type of bond failure produced by a splitting type failure (i.e., concrete cracking longitudinally along the bar). The second type is that produced by shearing the concrete by the bar deformations or by shearing off the bar deformations.

It is recognized that cracks normal to the bar will reduce the bond capacity. This conduction is analogous to that occurring in a flexural member where reinforcement is subjected to tensile stresses. The code advises that splicing at points of maximum tensile stress should be avoided wherever possible but provides for using a reduced allowable bond stress where such a splice is unavoidable (refer to ACI 318-63, Section 805). Such a condition is not uncommon, as evidenced by common practice for splicing bars in negative moment regions of rigid frames.

Cracking parallel to a reinforcing, although undesirable, is controlled by the strength across the crack provided by reinforcement usually associated with an orthogonal arrangement of bars. This condition is the basis for concern for splices occurring close together for a series of bars where spirals or closely spaced stirrups are suggested for use.

It should be noted that the development of rebar bond in a prestressed structure is less severe than in conventional reinforced concrete structures such as buildings, chimneys, and tanks. On this structure the reinforcement for which bond development is required to effect the anchorage consists only of the steel required to accommodate rotational strains or to control cracking. The interrupted reinforcement where bond is relied upon does not serve as primary membrane reinforcement.

Although temperature changes may affect the crack width on the containment during MODE 5 (Cold Shutdown), it is not considered to significantly change during plant operation. Because of the time lapse between construction and plant operation, the change in strains due to concrete shrinkage is extremely small. Because of the conservative design limits established to ensure an elastic response to transient loads, the crack widths should not change due to the design earthquake loads.

#### *3.8.1.4.5.4 Anchorage Stresses*

The stresses for the anchorage of the tendons and the dome reinforcement in the vicinity of the dome to cylinder transition were analyzed and compared with *Reference 11*. The maximum bursting stress caused by the tendon anchorage is 180 psi, compared with an allowable stress of 300 psi. The maximum spalling stress is 465 psi which required the addition of reinforcement. The maximum concrete compression under maximum load at the zone between the anchorages of the tendon and the dome reinforcement is 650 psi, compared with an allowable stress of 1250 psi. The anchorages for tendon and reinforcement are separated so as to minimize overloads of anchorage stresses.

The design provides for a factor of safety of 2.2 times the factored load against shear failure at this location. Details of the anchorage zone in the dome to cylinder transition are shown in Figure 3.8-5.

#### *3.8.1.4.5.5 Shell Stress Analytical Procedures*

The analytical procedures used for the stress analysis of the shell are summarized in the following paragraphs.

**Base to Cylinder Discontinuity**

The analysis considered a stiffness circumferentially of 116.5 lb/in.$^2$

(k = ($A_s$) ($E_s$/2) = 116.5 lb/in$^2$)

Based upon the analogy of a semi-infinite beam on an elastic foundation (*References 15* and *16*) it can be shown for the model described in Figure 3.8-20 that:

$$\textit{Deflection}: y = \frac{e^{-\beta X}}{2\beta^3 EI_z}[P_0 \cos\beta X - \beta M_0 (\cos\beta X - \sin\beta X)]$$

(Equation 3.8-7)

$$\textit{Rotation}: \phi = \frac{e^{-\beta X}}{2\beta^2 EI_z}[-P_0 \sin\beta X + \cos\beta X + 2\beta M_0 \cos\beta X]$$

(Equation 3.8-8)

$$\textit{Moment}: M = \frac{e^{-\beta X}}{\beta}[P_0 \sin\beta X - \beta M_0 (\sin\beta X + \cos\beta X)]$$

(Equation 3.8-9)

$$\textit{Shear}: v = e^{-\beta X}[P_0 (\cos\beta X - \sin\beta X) + 2\beta M_0 \sin\beta X]$$

(Equation 3.8-10)

Symbols that are not defined on Figure 3.8-20 are as follows:

*E* = Young's modulus for beam material

$I_z$ = Moment of inertia of beam

$$\beta = \sqrt[4]{\frac{k}{EI_z}}$$

(Equation 3.8-11)

k = Foundation modulus

It can also be shown that:

Hoop Force: $F_\theta = r(p - ky)$

$$\textit{Base reaction: } P_0 = \frac{2p\,\beta^3 EI_z}{k}(\beta M_0)$$

(Equation 3.8-12)

Symbols not previously defined are as follows:

*r* = average radius of shell

*p* = internal pressure

All stress resultants, shears, and moments were calculated on the basis of the foregoing equations. Because of the use of the hinge, the moment at the base of the cylinder ($M_o$) consists only of the restraining moment produced by the elastometer bearing pads and pseudo-moment applied to ascertain the effect of thermal stresses.

No inclined bars (i.e., bent shear bars) are used on the containment structure. As shown on Figure 3.8-4, stirrups are used at the base of the cylinder to an elevation 10 ft 5 in. above the base. This structure is prestressed vertically and, with the hinge design at the base, is subject only to bending stresses and not to tensile membrane stresses in the longitudinal direction. Therefore, the stirrups are anchored in concrete subject to only vertical cracks due to membrane loads. As shown on Figure 3.8-4, Section 9-9, the stirrups are continuous around the structure. Consequently, anchorage is provided both by bond and by the mechanical attachment to the vertical bars on the inside face.

In general, there are two types of bond failure (*References 18* and *19*). In one type of bond failure the concrete surrounding the bar splits along the reinforcing steel. In the other, the splitting does not occur but the concrete between the deformations in the reinforcement is sheared off, thus leaving a round hole in solid concrete. For the splitting failures, the tensile strength of concrete, distance between bars, and the magnitude and distribution of lateral stress acting on the bars are important variables affecting the bond strength. The bond limits, including lapped splice requirements in ACI-318, are based upon tests in which the failures were splitting type failures. Since the bond tests were made on beams, there was an absence of lateral confining stresses. The bond strength for splitting failures would most certainly be lower than the bond strength where the failure is the shearing off of the concrete between the reinforcing steel deformations. Confinement caused by lateral pressure can change the failure from "splitting" to "shearing" and increase the bond strength considerably (*Reference 19*).

The exact increase due to lateral pressure is not known because the tests were run on small size specimens that would have little to do with any actual bond stress situation occurring in practice. It is known that in simple beam tests, the effect of the confinement at the support increases the bond strength. Where confinement is included in the design, the actual bond strength would appear to be higher than the design values permitted by ACI-318. Consequently, for the configuration of stirrups used in the cylinder to base juncture it is considered that ACI-318 design limits on anchorage provide a conservative basis for the design.

**Dome to Cylinder Discontinuity**

The analysis was based upon general shell theory (*Reference 20*) using the model shown in Figure 3.8-21. At a distance sufficiently removed from the discontinuity it can be shown based upon membrane theory that:

$$\delta_c - \delta_d = pr^2 \frac{1 - \frac{\nu_c}{2}}{E_c t_c} - \frac{1 - \nu_d}{2E_d t_d}$$

(Equation 3.8-13)

Symbols not previously defined are as follows:

$\delta_c$ = Normal displacement of cylinder

$\delta_d$ = Normal displacement of dome

$\nu_c$ = Poisson's ratio for cylinder

$\nu_d$ = Poisson's ratio for dome

$E_c$ = Young's modulus for cylinder

$E_d$ = Young's modulus for dome

$t_c$ = Shell thickness of cylinder

$t_d$ = Shell thickness of dome

In calculating the quantities $Q_o$ and $M_o$ it is assumed that the bending is of a local character and, therefore, that the bending is of importance only in the zone of the spherical shell close to the joint and that this zone can be treated as a portion of a long cylindrical shell. It can therefore be shown that

$Q_o = 1/Z\ (\delta_c - \delta_d)$

$M_o = 1/Y\ (\delta_c - \delta_d)$

where Z and Y are functions of dome and cylinder stiffnesses.

**Base, Cylinder, and Dome**

The calculated stress resultants ($N\phi$, $N\theta$), stress couples ($M\phi$, $M\theta$), meridional shears ($V\phi$), and radial displacements ($\delta_R$) for dead load, final prestress, operating temperature (winter and summer), internal pressure, accident temperature, and earthquake are as listed in Table 3.8-5. These loads were combined as shown in Table 3.8-6. The results for the load combinations are as shown in Appendix 3C.

The physical constants used in the analysis described above were as follows:

**Uncracked concrete**

$E = 4.1 \times 10^6$ psi

$G = 1.8 \times 10^6$ psi

$\nu = 0.15$

**Cracked concrete**

$E = 0$

$G = 0$

$\nu = 0$

**Rebar/liner**

$E = 29 \times 10^6$

Shrinkage and creep for the prestressed concrete were assumed to be $320 \times 10^{-6}$ in./in.

For the data tabulated, the analytical model considered was always the cracked model associated with the accident condition.

On the basis of the foregoing data the liner stresses at selected load combinations (refer to Table 3.8-6 for load combinations) are as follows:

| | Cylinder (X = 60 ft) | | |
|---|---|---|---|
| **Load Combination** | **$\sigma\phi$** | **$\sigma\phi$** | **Dome Apex** |
| 1 | -14.3 ksi | -2.6 ksi | -2.4 ksi |
| 3 | -10.7 ksi | +0.1 ksi | -0.2 ksi |
| 29 | -2.9 ksi | +27.0 ksi | |

The discontinuity stresses between the dome and cylinder were determined by considering the following:

a. That the dome concrete cracks in tension and the cylinder concrete cracks vertically in tension. The radial deformations of the cylinder and the dome are conservatively assumed to be a function of the reinforcing steel alone. The steel areas across the discontinuity are established so as to develop a compatibility of stresses and therefore also of deflections.

b. That neither the upper part of the cylinder nor the lower portion of the dome concrete cracks and that the difference in deflection of the cylinder and the dome some distance from the discontinuity is a function only of the concrete properties. The solution, as developed in Theory of Elasticity, by Timoshenko and Goodier (*Reference 21*), assumes that the lower portion of the dome behaves in a manner similar to that of a cylinder (i.e., the discontinuity moments and shears are rapidly dissipated and become minimal at a limited distance from the discontinuity). For this condition only a nominal shear and moment (4 k/ft and 18 k ft/ft) would be developed due to the most severe factored loads.

c. That the radial deformation of the cylinder some distance from the discontinuity is a function of the cracked concrete section, and the radial deformation of the dome is a function of the uncracked section. The probability of vertical cracks in the cylinder propagating into the dome is remote. The discontinuity shears and moments resulting from the condition are excessive and require the assurance by development of planes of weakness in the concrete that cracking will occur uniformly across the discontinuity.

The discontinuity stresses can be calculated with greater confidence based upon a model of cracked concrete above and below the transition. To ensure that a condition does not exist where either the pressure load produces significant cracking of concrete in the dome at the discontinuity or vertically in the cylinder, crack initiators are used to permit a uniform propagation of tension cracks in the concrete at the discontinuity.

The safety against shear (or tension) failure at the dome-cylinder intersection was investigated by the following two approaches:

aa. An ultimate strength solution based on the Mohr-Coulomb failure criteria for concrete and plane failure surfaces.

bb. An elastic solution in which the stresses were calculated at the point of maximum splitting tensile stress given by Leonhardt (*Reference 11*). The principal stresses at the point were obtained and the stability of the section verified by assuming a direct relationship between tensile and compressive strengths which was obtained from several investigators.

The first approach indicated a collapse load 2.16 times larger than the factored load of (0.95 D + 1.5 P) while the second solution led to a safety factor of 2.12 referred to the same load. On the basis of this analysis, it is concluded that the factor of safety against shear (or tension) failure at the dome-cylinder intersection is greater than the overall safety factor of the containment structure.

The section between anchorage plates for the tendons and the dome reinforcement was also checked using the analogy of a corbel and reinforcement provided as recommended by Kriz and Raths (*Reference 22*).

The dome reinforcing bars are mechanically anchored in the precompressed zone below the top anchors of the tendons. This mechanical anchorage is in the form of Cadweld connections arc welded to a continuous mild steel plate. No bond development is required to fulfill the design requirements. ACI 318-63 limits on splicing are developed upon bond requirements based on a splitting type of failure (*References 18* and *19*). These requirements are not relevant to the design of the containment anchorage.

It was not necessary to stagger the dome anchor plate from an engineering standpoint. Common practice in regular reinforced concrete structures is to stagger splices and, if possible, the anchorage of reinforcing steel. However, in this instance, anchorage is developed by mechanical means in a region of membrane compression. This conservative anchorage environment negates the need for the staggering of splice plates.

#### 3.8.1.4.6 Insulation

The liner insulation is Vinylcel as manufactured by Johns-Manville. This material is a closed-cell polyvinyl chloride foam insulation with low conductivity, low water absorption, and high strength. The insulation is 1.25-in. thick with a density of 4 pcf.

The function of the liner insulation is to limit the mean temperature rise of the liner to 10°F at the time associated with the maximum pressure as shown on the transient for the factored pressure (90 psig). For this determination the containment vessel internal ambient temperature is assumed to be 120°F and 100% relative humidity and the external ambient temperature is assumed to be minus 10°F. The insulation is covered with a metal sheeting. The insulation is capable of withstanding periodic compression of 60 psig within a temperature range of 40°F to 120°F and a single compression to 69 psig within the same temperature range, both without any detriment or change to the insulation properties.

The results of a series of tests which have been performed are included in Section 3.8.1.7.1. Also included in that section are the results of an analog study of the insulation when subjected to the pressure and temperature transients associated with an internal pressure of 90 psig.

**Hypothetical Local Insulation Failure**

If a local failure of insulation is hypothesized at a typical piping penetration, the circumferential liner stress at the point of failure is calculated as a compression stress of 6.3 ksi at design pressure and temperature. This stress compares with a tensile stress for the insulated liner of 18.5 ksi. Due to this secondary effect, the tensile stress of the mild steel reinforcement would be locally increased but that would not alter the ultimate capacity of the section.

The vertical liner stress would increase locally at the point of insulation failure until the plate yielded in compression. The consequential loss of prestress would be distributed over the full height of the wall. Considering a 2-ft dimension for the area without insulation, the loss of prestress of the affected tendon would be approximately 1%. The loss of prestress for the entire vessel would consequently be minimal.

The effect of insulation failure at a penetration would be to produce yielding of the sleeve circumferentially in compression and longitudinally in bending.

### 3.8.1.4.7 Liner

#### *3.8.1.4.7.1 Vibrations*

The main sources of liner vibrations are vibrating pipes which pass through the liner. The vibrations from these pipes are transferred to the liner from the penetration sleeves. The piping systems expected to vibrate are the following:

| Pipe | Frequency of Vibration |
|---|---|
| Main steam line | 13 Hz |
| Feedwater line | 13 Hz |
| Charging line | 1.8 to 18 Hz |
| Cooling pump seal water line | 1.8 to 18 Hz |

During a plant design life of 40 years such penetrations may be subject to full stress reversals under operating conditions which are in excess of 2,000,000 cycles. The inner end plate and sleeve of these penetrations were designed for this condition using the stress limitations of the ASME Nuclear Vessel Code.

Regarding cyclic loads due to earthquakes, the anticipated number of cycles (50 to 250) will not require reduction in the stress limits. However, as these vibrations are carried into the concrete shell through the sleeve, which is an extremely stiff member relative to the liner, the degree of participation of the liner in absorbing these vibrations is small, being a function of the sleeve movements at the sleeve liner weld connection. Due to the rigidity of the penetration and its method of fixture to the concrete sleeve, movements at this weld interface are negligible.

#### *3.8.1.4.7.2 Anchorage Fatigue Analysis*

The sidewall liner is anchored to the concrete with steel channels of 3-in. depth on approximately 4-ft 3-in centers. The channels are intermittently welded to the liner. The channels ensure elastic stability of the liner under potential compression loads and also provide the required capacity to resist instability due to vacuum loads. The steel channels had the added function of stiffening the liner during erection.

#### *3.8.1.4.7.3 Base Slab Liner*

Backup bars in the form of structural tees were embedded and anchored into the 2-ft 0-in. thick base slab as shown on Figure 3.8-6. These backup bars, all of which are continuous, were placed flush with the top concrete surface. The liner plate was placed on the concrete surface and the butt joint made as shown on the typical joint detail on Figure 3.8-6. Tolerance on height is ±3/8 in. and out-of-flatness is 1/4 in. in 10 ft.

After nondestructive testing of this weld (liquid penetrant examination), the test channels were installed and leak tested. The nominal 24-in. concrete cover was then placed and the test channels were again pneumatically tested. The liner seams and the channel to liner welds were found to be leaktight. No grout was placed between the base slab and the liner.

A nominal 24-in. concrete cover was placed over the liner. Therefore, the liner is located at mid-thickness of the concrete. The walls of the reactor cavity are assumed to act as a shear key with the required capacity to transfer earthquake loads. Consequently, the test channels should not be subject to a significant shear load.

The concrete cover placed on top of the liner does not necessarily ensure intimate contact between the liner plate and the base slab over the entire plan area, but does ensure that sufficient bearing exists to adequately distribute vertical loads from columns and walls to the base slab. All shear loads are assumed transferred by means of the walls of the reactor cavity, which acts as a shear key. Refer to Figure 3.8-3 for reactor cavity wall details.

#### *3.8.1.4.7.4 Liner Stresses*

The maximum nominal liner stress (meridional direction), considering shrinkage and creep of concrete, is 14,100 psi compression.

The liner was reinforced about all openings in accordance with the ASME Unfired Pressure Vessels Code (i.e., by replacing the cut-out area of 3/8-in. liner plate). Normally this involved the use of a common 3/4-in. plate for a group of penetrations. Minimum spacing of penetrations conforms to ASA N 6.2-1965, Safety Standard for Design, Fabrication, and Maintenance of Steel Containment Structures for Stationary Nuclear Power Reactors. The liner stress concentration at the hole is determined based upon elasticity solutions for a flat plate of constant thickness subjected to a biaxial stress field.

The combination of stresses from all effects is combined in accordance with the ASME Nuclear Vessels Code, Article 4, and evaluated on the basis of the allowable peak stress intensity, which for the liner material is 60,000 psi.

The data provided in Table 3.8-4 and the description contained in Section 3.8.1.4.5.2 do not consider the liner as resisting earthquake shears. It can be shown that the principal stress resultant is oriented nearly horizontal in that the shear component is small relative to the axial components. Nevertheless, the same model previously used where dowel action was considered was reanalyzed to determine the interaction between concrete, reinforcing bars, and liner. This analysis conservatively assumed that the liner and concrete shell acted compositely.

The maximum longitudinal shear at the base of the cylinder (i.e., on an axis normal to the direction of ground motion) due to the 0.2g ground acceleration is 67.2 k/ft. The shear modulus of the liner $G_L$ [E/2(1 + V)] equals 11,200 ksi. The effective shear modulus of the concrete wall is based on pure shear on the uncracked concrete plus the dowel action of the horizontal reinforcement across the hypothesized vertical crack. The conservative assumption was made that the shear stiffness across the crack is not increased by aggregate interlock.

The dowel stiffness is established on the basis of a load-slip relationship of 3000 kips/in. which is a linear relationship for the motions calculated in this study. The shear modulus of the cracked wall section $G_W$ equals

$$\frac{G_c}{1 + \frac{A_c G_c}{3000 \cdot L}}$$

(Equation 3.8-14)

where terms are as defined on Figure 3.8-22.

The results of this study are summarized as follows:

| Cracking Spacing L (in.) | Gw (ksi) | Linear Shear $\tau_L$ (psi) | Concrete Shear $\tau_C$ (psi) |
|---|---|---|---|
| 25 | 188 | 5200 | 87 |
| 12 | 95 | 7700 | 65 |
| 8 | 65 | 9000 | 53 |

As a check on the allowable liner shear stress, a Mohr's circle was used based upon a critical shear stress of 16 ksi (1/2 $\sigma\ \gamma$) as shown on Figure 3.8-23.

It is thus shown that the allowable shear stress exceeds the calculated shear stress based upon these conservative analytical models. It should be reiterated that these calculated stresses in no way represent expected response to the loading being considered, but instead represent an upper bound based upon a simplified model.

#### *3.8.1.4.7.5 Liner Buckling*

The liner anchors in the cylinder are 3-in. deep channels spaced horizontally at approximately 4 ft 4 in. on centers. The liner is analyzed as a flat plate, which is a conservative assumption in that the liner will have to buckle against its own curvature. For analysis it is assumed that the liner is fixed at the angles and that there will not be any differential radial moments of the boundaries. The liner anchors are designed and spaced so that the critical buckling stress will be greater than the liner stress under operating or incident conditions. In the case of a cylinder, considering conservatively a uniaxial stress field, the critical buckling stress is 99,000 psi, which compares with a maximum stress of approximately 4000 psi.

Details on the channels attached to the liner as anchors are shown in Figure 3.8-24.

The containment structure was designed to use reinforcing bars with a minimum yield stress of 40,000 psi, as this basis leads to stress levels in the liner which ensures that it does not yield when the containment is at test pressure. The calculated maximum tangential liner stress in the cylinder due to the test pressure load is 26,500 psi (tension). This compares with a calculated liner stress due to the factored accident loads (1.5 P = 90 psig) of 28,700 psi (tension). The thermal gradient is considered in developing these stresses for accident conditions, but not for test conditions. In neither case is the calculated stress equal to nor greater than the yield stress. The meridional liner stress in the cylinder under both test and accident conditions is compressive; this and the meridional or circumferential stresses in the dome are lower than those listed above.

## Cylinder Liner

In view of the large shell radius to liner thickness (630/0.375 = 1680) and shell radius to support spacing (630/52 = 26) ratios, a flat plate idealization is considered to be fully justified.

The steel liner is therefore considered to be a flat, thin, isotropic plate supported with line supports against a rigid wall as shown on Figure 3.8-25.

The buckling pattern of the panel plate is a wave surface. Therefore, the equations derived for a wave surface are used where the deformation pattern of the panel plate is as shown on Figure 3.8-25.

From the large deflection analysis of clamped plates under biaxial compression it can be shown that:

$$\frac{W_o^{\,2}}{a} = \frac{1}{\frac{1966}{400} + \frac{9}{4}\nu - \frac{1066}{400}\nu^2}\left[\frac{2}{3}\frac{t^2}{a} + \frac{3}{4\pi^2}(1+\nu)(\epsilon_1 + \epsilon_2)\right]$$

(Equation 3.8-15)

Since $W_O$ equals zero at the onset of buckling

$$\left[\frac{2}{3}\frac{t^2}{a} + \frac{3}{4\pi^2}(1+\nu)(\epsilon_1 + \epsilon_2)\right] = 0$$

(Equation 3.8-16)

Therefore, under operating conditions, when $\in_2 = -\nu\in_1$

$(\in_1)_{CR} = -9.65\ (t/a)^2$

$(\sigma_1)_{CR} = E\in_1 = -9.65 \in (t/a)^2$

For this structure wherein plate thickness is 3/8 in. and spacing between vertical anchors is 49.5 in.

$(\sigma_1)_{CR} = -9.65 \times 30 \times 10^3 (0.375/49.5)^2 = 16.6$ ksi

This applies for operating conditions only. A similar analysis is also performed for accident conditions wherein $\in_1$ is compression and $\in_2$ is tension.

Using the notation $f = N_1/N_2 = P_1/P_2$ and where $\in_1/\in_2 = (P_2 - \nu P_1)/(P_2 - \nu P_2)$, it can be shown that

$$(\sigma_1)_{CR} = -11.6 \frac{1 - f\upsilon}{(1-\upsilon)(1+f)}$$

(Equation 3.8-17)

Therefore, if f is negative, as would be the case for this structure, the critical buckling stress $(\sigma_1)_{CR}$ continues to increase as $\sigma_2$ increases in tension. In summary

| F | $(\sigma^1)_{CR}$ | $\sigma_2$ |
|---|---|---|
| | | |
| 0 | -16.6 ksi | 0 |
| -0.125 | -19.6 | +2.4 |
| -0.25 | -23.8 | +6.0 |
| -0.375 | -29.8 | +11.2 |
| -0.50 | -38.1 | +19.0 |

For this structure with the insulated liner the operating condition represents the most severe condition for the stability analysis.

From *Reference 23* it is shown that for an initial displacement $Y_o$ and the initial deflection curve, defined as:

$Y = Y_o/2[1 - \cos 2\pi (X/L)]$

that the equivalent liner strain equals

$\in_L = 1/4 (\pi_O/L)^2$

For this structure it can then be shown that for varying amounts of $Y_o$ the resulting liner strains ($\in 2$) are as follows:

| Yo | Yo/L | $\in 2$ | $\sigma 2$(psi) | N2 (lb/in.) |
|---|---|---|---|---|
| 0.1 in | $2.02 \times 10^{-3}$ | $1.01 \times 10^{-5}$ | 303 | 11.4 |
| 0.2 in | $4.04 \times 10^{-3}$ | $4.00 \times 10^{-5}$ | 1200 | 45.0 |
| 0.3 in | $6.06 \times 10^{-3}$ | $9.09 \times 10^{-5}$ | 2727 | 102 |

The welded connection between the anchor and the liner consists of a staggered 3/16 in. fillet weld on both sides of the flange; of 1.5 in. length in 4 in. This weld has a shear capacity of approximately 2.5 k/in., which obviously is sufficient capacity for possible liner dimensional imperfections.

The liner anchor connection is designed for the differential shear load, caused by a buckled liner panel, which is equal to the load in the adjacent panel under normal operating of the plant. Under internal pressure loading, the liner will be in tension in the hoop direction.

Deviation in liner anchor spacing within normal erection practice for pressure vessels will not affect liner stability or liner anchor design. Liner hoop compressive stresses are negligible during winter operation of the plant. The liner is insulated and thermal stresses are insignificant. Therefore, a local poor or inadequate weld between liner and anchor will not cause any danger with respect to liner stability.

The effect of a liner panel erected out of roundness between two adjacent anchor points can be defined as follows:

a. Under operation of the plant, the liner hoop compressive force in the neighboring panel can be transferred directly in shear to the nearest liner anchor. (See above.)
b. Under internal pressure loading, the liner hoop tensile force will be redistributed to other parts of the liner, and possibly also to the hoop reinforcing steel until the liner is being engaged to resist additional hoop stresses as the pressure load increases.

Variations in liner material yield strength are not significant in that predicted operating/accident loads are always significantly less than minimum yield. The calculated liner stresses are tabulated in Appendix 3C.

The interior of the liner below elevation 346 ft (15 ft above the dome springline) in the dome area and the cylinder can be inspected after the insulation has been removed. The liner in the dome above this elevation can be directly inspected.

**Dome Liner**

See Section 3.8.2.3 for a discussion of the dome liner stress analysis.

##### *3.8.1.4.7.6 Liner Corrosion Allowance*

No corrosion allowance has been included in the design of the liner, which has a minimum thickness of 0.25 in. The exposed surface of the liner has been given a protective coating of paint. The cylindrical portion is protected by insulation.

The outer surface of the steel is in direct contact with the concrete, which provides adequate corrosion protection due to the alkaline properties of concrete. The external underground surface of the concrete shell has a membrane waterproofing system to act as a seal for protection against underground water.

### 3.8.1.5 Penetrations

#### 3.8.1.5.1 General

All penetrations through the containment reinforced concrete pressure barrier for pipe, electrical conductors, ducts, and access hatches are of the double barrier type. Typical electrical and pipe penetrations are shown on Figure 3.8-26.

In general, a penetration consists of a sleeve embedded in the reinforced concrete wall and welded to the containment liner. The weld to the liner is shrouded by a test channel which is used to demonstrate the integrity of the joint. The pipe, duct, or access hatch passes through the embedded sleeve and the ends of the resulting annulus are closed off, generally by welded end plates. Piping penetrations have a bellows type expansion joint mounted on the exterior end of the embedded sleeve where required to compensate for differential motions. The only exceptions to providing an annulus about piping occurs for the three drain lines from sump B. Details of these penetrations are shown on Figure 3.8-27.

All welded joints for the penetrations including the reinforcement about the openings (i.e., sleeve to reinforcing plate seam) are fully radiographed in accordance with the requirements of the ASME Nuclear Vessels Code for Class B Vessels, except that nonradiographable joint details are examined by the liquid penetrant method. For fully radiographed welds, acceptance standards for porosity are as shown in Appendix IV of the Nuclear Vessels Code. The remaining liner weld seams are examined by spot radiography. (The ASME Unfired Pressure Vessels Code states that porosity is not a factor in the acceptability of welds not required to be fully radiographed.) Verification of leaktightness is by means of pressurizing test channels.

Penetrations are designed with double seals so as to permit individual testing at design pressure. In this case, an adulterant gas method is used. An air distribution system is provided for periodic testing.

All penetrations are provided with test canopies over the liner to penetration sleeve welds. Each canopy, except those noted below, is connected to, and pressurized simultaneously with, the annulus between to the penetration pipe and sleeve when under test. The exceptions are the canopy for the fuel transfer penetration, which must be pressurized independently of the annulus because of the separation posed by the transfer canal liner; and the three pipe penetrations in sump B, in which only the canopies are pressurized as there are no annuli.

For details of small penetrations analysis, refer to Section 3.8.1.5.6.

### 3.8.1.5.2 Electrical Penetrations

There are generally five types of electrical cable penetrations required, depending on the type of cable involved:

Type 1 High voltage power, 4160 V.

Type 2 Power, control and instrumentation: 600 V and lower.

Type 3 Thermocouple leads.

Type 4 Coaxial and triaxial circuits.

Type 5 Fiber Optic

All five types of penetration designs are a cartridge type, basically as shown on Figure 3.8-28. The cartridge length and the support of cables immediately outside containment are designed to eliminate any cantilever stresses on the cartridge flange.

Type 1 penetrations use a rubber insulation copper rod. This insulated rod passes through two leaktight gland fittings that are threaded into an all-welded steel pressure cartridge. High alumina insulating bushings are used as an alternative to provide the double barrier.

Type 2 penetrations use single or multi-conductor mineral insulated cable with a metallic sheath. The cable passes through two leaktight gland fittings that are threaded into an all-welded steel pressure cartridge. The ends of the mineral insulated cable are potted with an epoxy resin compound.

Type 3 penetrations are similar to Type 2 except that the conductors are thermocouple material. The sealing and terminations are identical to Type 2 penetrations.

Type 4 penetrations are used principally for coaxial and triaxial circuits. Each cable passes through two leaktight gland fittings that are threaded into an all-welded steel pressure cartridge similar to that employed in the other penetration types. Inside the cartridge, between the double barrier, a plug and receptacle connection is provided to block leakage through the cable itself.

The Type 5 penetration assembly consists of a stainless steel header plate/extension tube, feedthrough modules containing fiber optic conductors, and a stainless steel support plate system for the feedthrough modules. Additional components such as a penetration monitoring tube/plug and fill valve, are provided for leakage surveillance of the penetration. The feed-through modules containing the fiber optics pass through the header plate and are secured and sealed to the plate with specially designed Midlock stainless steel compression fittings. These fittings are installed from the outer face of the header plate and are concentric with the feedthough modules.

These penetrations are designed to permit as much shop fabrication and testing as possible and minimize on-the-job fabrication. At the same time, double barrier protection and accessibility for in-place testing is maintained.

In general, shop fabrication and quality control are used in all penetration designs where practical. For example, penetration sleeves are shop welded to certain liner plates in specified locations, and transition welds between carbon and stainless steel are shop welds.

#### 3.8.1.5.3 Piping Penetrations

Piping penetrations are provided for fluid-carrying pipes and for air purge ventilating piping. Most pipes penetrating the containment connect to equipment inside and outside of the containment, and are for either high temperature or moderate- to low-temperature service. Other pipes, such as for purge air, connect the containment volume to the outside atmosphere.

In all cases, a piping penetration consists of an embedded sleeve with the ends welded to the penetrating pipe. Provision is made for expansion with bellows type joints forming a testable compartment in the case of hot lines. Further, in the case of the high-temperature pipe lines, the penetrations are designed so that the temperature of the concrete around the penetration does not exceed ASME III, Division 2, Subsection CC-3340, Item (a) limits. For normal or any other long-term period concrete temperatures shall not exceed 150°F except for local areas around the penetration, which are allowed to have increased temperatures not to exceed 200°F. For accidents or any other short term period the temperatures shall not exceed 350°F for the inner surfaces in containment except local areas are allowed to reach 650°F from steam or water jets in the event of a pipe failure. The high-temperature pipe lines use a forced air cooling system, connected to cooling coils integrated with the penetration sleeves. The cooling coils are in the form of an embossing welded directly to the inner surface of the penetration sleeve as shown on Figure 3.8-29. The cooling air exit temperature is monitored and can be related to the concrete-to-sleeve interface temperature. A prototype test was performed under simulated operating conditions to verify assumptions made for hydraulic and thermal calculations. In addition, provisions are made to insert and monitor thermocouples at approximately mid-thickness of the concrete wall at the concrete to sleeve interface in most of the air cooled penetrations (12 of 15), and these enable exhaust air temperature and maximum concrete temperature to be correlated.

The modes of isolating these pipes during a high-pressure containment incident are covered in Section 6.2.4.

#### 3.8.1.5.4 Access Hatch and Personnel Locks

An equipment hatch, constructed of welded steel and having a double-gasketed flange and bolted dished door, is located near grade. The equipment access opening has a diameter of 14 ft.

All major components were moved into the containment prior to installation of the hatch. The hatch barrel is embedded in the containment wall. All weld seams at the joint between the barrel and the liner have test channels for periodic leak testing. For components of the hatch, including barrel and door, test channels are not provided. Details of the equipment hatch are shown in Figure 3.8-30.

An equipment hatch closure plate is available for use when in the MODE 5 (Cold Shutdown) or MODE 6 (Refueling) modes when the equipment hatch is removed. The plate is bolted to containment in place of the equipment hatch. The closure plate has a hatch door that provides an emergency means of containment egress and provision for temporary services needed during an outage to be brought into containment while still providing containment closure. The closure plate is designed to maintain containment closure during a fuel-handling accident, prohibiting excessive radiological releases. It is designed to withstand a pressure load of +0.5 psi to -0.5 psi. Plant operating procedures restrict the containment pressure differential to 0.5 psig when the closure plate is in place. The plate has a gasket system that when bolted down provides an airtight mechanical fit. No leak testing is required. The closure plate and its storage supports are Seismic Category I. As an alternative during MODE 5 or MODE 6, the equipment hatch opening can be isolated by an installed retractable overhead door. The retractable door is attached to a concrete enclosure built around the equipment hatch opening outside of containment.

Two personnel accesses are provided. One personnel hatch penetrates the dished door of the equipment hatch. The other is located diametrically opposite the equipment hatch. Each personal hatch is a hydraulically-latched double door, welded steel assembly. An equalizing valve connects each personnel hatch with the interior of the containment vessel for the purpose of equalizing pressure in the personnel hatch with that in the containment. Hatch closures are of the double-tongue, single gasket type. The access locks are properly interlocked to ensure door closure at all times, as defined in Section 12.3.2.2.7, with annunciation in the control room, except as allowed in Technical Specification 3.9.3 (Containment Penetrations). Details of the personnel hatch are shown on Figure 3.8-31.

For details of the analytical approach for large opening reinforcement design refer to Appendix 3B.

#### 3.8.1.5.5 Fuel Transfer Penetration

A fuel transfer penetration is provided for fuel movement between the refueling transfer canal in the reactor containment and the spent fuel pool (SFP). The penetration, as indicated by Figure 3.8-32, consists of a stainless steel pipe installed inside a larger pipe. The inner pipe acts as the transfer tube and connects the reactor refueling canal with the spent fuel pool (SFP). The tube is fitted with a standard stainless steel flange in the refueling canal and a stainless steel sluice gate valve in the spent fuel pool (SFP). The outer pipe is welded to the containment liner and provision is made, by use of a special seal ring, for freon gas leak testing of all welds essential to the integrity of the penetration.

The fuel transfer penetration, like all other penetrations, is anchored in the containment shell. Because this anchor point moves when the containment vessel is subjected to load, expansion joints are provided where the penetration is connected to structures inside and outside of the containment vessel. Since the penetration is located on a skewed angle, not normal to the containment shell, the expansion joints are subjected to both radial and tangential (lateral) motions. The expansion bellows inside the containment vessel provide a water seal for the refueling canal and accommodate thermal growth of the penetration from the anchor, as well as the pressure and earthquake produced motion of the anchor (the containment shell). The gasketed expansion joint accommodates motion of the sleeve within the containment shell relative to the portion of the sleeve anchored in the wall of the refueling canal in the auxiliary building. Section A-A on Figure 3.8-32 indicates a pipe to detect leakage of ground water into the penetration through the gasketed joint. The expansion bellows inside the auxiliary building performs the same function as described for that within the containment.

### 3.8.1.5.6 Typical Penetration Analysis

#### *3.8.1.5.6.1 Loss-of-Coolant Accident*

The concrete temperature adjacent to piping penetrations is limited to 200°F (see Section 3.8.1.5.3). The penetrations for high-temperature pipe lines employ air-cooled coils integrated with the penetration sleeves. The test of a prototype penetration indicated that sufficient margin existed in the design to permit an 80-min period of no coolant flow before the temperature at the interface with the concrete reached 150°F. Backup fans are provided for the air coolant with a capacity of 100% of the design requirement. The concrete shell is not designed for the two-dimensional thermal gradients in the area of the piping penetrations. The typical one-dimensional thermal gradients used in the design are shown in Figure 3.8-8.

The radial deformation of a hole in a plate subjected to a stress field is determined by performing an integration of the tangential strains around the periphery of the hole (*Reference 21*). The increase in the diameter of a hole ($\delta_D$) due to a biaxial stress field (S and S") at a location in the direction of this stress field (S) is as follows:

$$\delta_D = \frac{1}{E}\int_0^{\pi}(S - 2S\cos 2\theta - [S' - 2S'\cos(2\theta - \pi)])r\sin\theta d_\theta$$

(Equation 3.8-18)

$\delta_D = (2/3)(r/E)(5S - S')$

This corresponding elongation of the plate which would occur if the hole were not present over a length, r, is

$\delta = [2(S + \nu S')/ E]\ r$

The above derivation neglects the stiffening effect of the penetration sleeve and thus overestimates the hole distortion.

The average liner stress (horizontally) due to a loss-of-coolant accident, defined as S, is a tensile stress of 14.1 ksi. (The liner is thickened from 3/8 in. to 3/4 in. around the penetration.) The average liner stress (vertically), defined as S, is a compression stress of 10 ksi.

The maximum increase in diameter of the hole, which is in the horizontal direction for this 10-in. diameter penetration, is then:

$$\delta_D = \frac{\frac{2}{3} x5 \cdot (5x14.1 - 10)}{30x10^3} = 0.006710 in.$$

(Equation 3.8-19)

To simplify the analysis and to provide a conservative result, it is assumed that this deformation is uniform around the circumference of the penetration sleeve. Based upon this assumption:

Maximum moment sleeve = f/4 λ per inch.

Radial deformation due to constant line load, r = $fr^2\lambda/ 2E t$.

Maximum hoop stress in sleeve = frλ/ 2t.

In the above equations:

f = line load at the liner sleeve interface

r = radius of sleeve

λ = $3(1-v^2)/ R_2^2 t^2$

ν = Poisson's ratio

$R_2$ = mean radius of sleeve

t = wall thickness

The material used for the penetration sleeves is SA-106, grade B, with a minimum yield strength of 31,000 psi at 300°F and an allowable stress intensity ($S'_m$), per the ASME Nuclear Vessels Code of 20,000 psi at 300°F. The stresses produced at the liner-penetration sleeve interface are defined in the ASME Nuclear Vessels Code as secondary bending and membrane stresses and are therefore limited to a maximum value of 60,000 psi (3 $S'_m$).

For the 10-in. diameter penetration sleeve using Schedule 80 pipe

$$\Delta r = \frac{0.00671}{2} = \frac{fx5^2 x0.746}{2x30x10^6 x0.594}$$

(Equation 3.8-21)

*f* = 6400 lb/in. circumference

Maximum bending stress $f_b$ = 6400 x 6 / (4 x 0.746 x $0.594^2$) = 36,500 psi

Maximum hoop stress $f_t$ = 6400 x 5 x 0.746 /(2 x 0.594) = 20,200 psi

Therefore, both the maximum bending and hoop stresses are less than the allowable stress of 60,000 psi. Thus, the use of Schedule 80 (10-in. nominal diameter pipe of SA-106, grade B) material was satisfactory for this penetration sleeve.

The material used for the end plates is SA-201, grade B, with a minimum yield strength of 28,350 psi at 300°F and an allowable stress intensity ($S'_m$) per the Nuclear Vessels Code of 18,000 psi at 300°F.

For a typical 6-in. diameter pipe penetrating the liner through a 10-in. diameter sleeve, the resulting moment and axial force at the anchor on the pipe, which is the end plate, from a thermal flexibility analysis based on normal operating conditions are 1500 lb-ft and 200 lb. Using an end plate thickness of 3/4 in., the maximum bending stress due to the applied moment is 6840 psi and due to the axial load is 4800 psi. The sum of the stresses (11,640 psi) is less than the allowable value.

### *3.8.1.5.6.2 Loss-of-Coolant Accident Plus Earthquake*

A typical 6-in. diameter pipe line is analyzed for the combination of 0.2g ground motion and the loss-of-coolant accident (60 psig). The one pipe line generates an equivalent static force of 1500 lb due to the excitation by the 0.2g ground motion.

This force is resisted at the anchorage by a combination of shear and compression on the sleeve. For this given load, two extreme conditions were analyzed, one with the resulting load applied parallel to the axis of the sleeve and the other with the load applied normal to the axis of the sleeve.

For the case with the load applied normal to the penetration axis and the sleeve of Schedule 80 - 10-in. diameter pipe, the maximum shear stress is 1530 psi and the maximum bending stress is 2470 psi. Due to internal pressure of 60 psig, the axial load on the penetration is 4710 lb. The resulting stresses in the sleeve are a maximum compression of 2775 psi and a minimum compression of 2165 psi.

For the case with the ground motion parallel to the axis of the penetration sleeve, the resulting stresses in the sleeve are a maximum compression of 374 psi and a minimum compression of 305 psi.

From this analysis, the seismic loads on a 10-in. diameter penetration sleeve arising from approximately 100 ft of 6-in. diameter pipe produce small stresses in the penetration elements.

The deformation of the penetration as previously determined is then applied to the liner sleeve and bending and hoop stresses are calculated. This approach is most conservative in calculating tensile stresses since the hole deformations are calculated neglecting the restraining effect of the sleeve and the sleeve stresses are considered to be a function of the total hole deformation.

For a typical piping penetration the stresses calculated on this basis are as follows:

| | Leak Rate Test | Loss-of-Coolant Accident |
|---|---|---|
| Average membrane stress in liner adjacent to sleeve | +18.8 ksi | +14.1 ksi |
| Maximum circumferential stress in sleeve | +28.0 | +20.2 |
| Maximum bending stress in sleeve | +50.6 | +36.5 |

The review of penetrations indicates that the maximum tensile stresses in the penetration elements occur during the leak rate test and not during the simultaneous occurrence of the loss-of-coolant accident plus the earthquake. By defining leaktightness (i.e., the area of holes in the liner) as a function of tensile stress in the penetration elements, it can be shown that the leakage would be greatest during the test.

#### 3.8.1.5.7 Penetration Reinforcement Analyzed for Pipe Rupture

The penetrations for the main steam, feedwater, blowdown, and sample lines are designed so that the penetration is stronger than the piping system and that the containment is not breached due to a hypothesized pipe rupture combined, for the case of the steam line, with the coincident internal pressure. These penetrations were analyzed for the bending moments, torques, shears, and axial loads transmitted by the pipes. The penetration sleeves were analyzed based upon elasticity theory with the maximum principal stress not exceeding yield stress. The piping connected directly to the primary coolant system, not including the sample lines, are anchored in the shield walls around the steam generators. One isolation valve is located on either side of the anchor (shield wall). The penetrations through the shield walls are designed as anchors to ensure that one hypothesized pipe rupture will not jeopardize both valves. The major components (i.e., the reactor vessel, steam generators, reactor coolant pumps, and pressurizer) are supported so as to ensure that the severance of a primary coolant pipe does not produce coincident severance of the steam system piping (Section 3.6). Therefore, the containment mechanical penetrations designed for the pipe rupture condition do not consider coincident loads from the loss-of-coolant accident. The pipe capacity in flexure is assumed to be limited to the plastic moment capacity based upon the ultimate strength of the pipe material. For the main steam and feedwater penetrations special reinforcement is required, as shown on Figures 3.8-29 and 3.8-33. This reinforcement provides for transferring shears, torque, and moments into the concrete wall through the liner. Steel elements of the containment and penetrations are designed on the basis of stresses not exceeding yield stress based on using a load factor of 1.0. Concrete elements are designed based upon the ultimate strength design provisions of ACI 318-63.

The piping was designed based on the Code for Pressure Piping ASA B31.1-1955, which was the current standard when the piping was designed. The code was also used to design all piping systems required for safe shutdown under the loss-of-coolant accident conditions.

### 3.8.1.6 Quality Control and Material Specifications

#### 3.8.1.6.1 Concrete

##### *3.8.1.6.1.1 Ultimate Compressive Strength*

The minimum ultimate compressive strength for a standard cylinder of concrete used in the design was as follows:

| | |
|---|---|
| Containment shell | 5000 psi in 28 days. |
| Other | 3000 psi in 28 days. |

##### *3.8.1.6.1.2 Quality Control Measures*

The specifications for the original structural concrete for Ginna Station required the following quality control measures:

A discussion for the replacement concrete placed during the 1996 Steam Generator Replacement is provided in Section 3.8.1.6.1.6.

**Preliminary Tests**

The Westinghouse Atomic Power Division obtained the services of a Testing Laboratory which, prior to the contractor commencing concrete work, made preliminary determinations of controlled mixes, using the materials proposed and consistencies suitable for the work, in order to determine the mix proportions necessary to produce concrete conforming to the type and strength requirements called for herein or on the drawings. Aggregates were tested in accordance with the latest editions of the following ASTM Specifications: C29, C40, C12, C128, and C136. Compression tests conformed to ASTM Specifications C39-64 and C192-65. The contractor submitted to the Testing Laboratory, a sufficient time before concrete work commenced, all concrete ingredients required by the Testing Laboratory for the preliminary tests.

The proportions for the concrete mixes were determined by Method 2 of Section 309 of Proposed ACI 301 and as previously specified.

The engineer had the right to make adjustments in concrete proportions if necessary to meet the requirements of the specifications.

In the event the contractor furnished reliable test records of concrete made with materials from the same sources and of the same quality in connection with current work, then all or a part of the strength test specified previously could have been waived by the engineer, subject, however, to any provisions to the contrary of building codes or ordinances of the governing authority.

**Field Tests**

During concrete operations, the Testing Laboratory had an inspector at the batch plant who certified the mixed proportions of each batch delivered to the site and sampled and tested periodically all concrete ingredients. Another inspector at the construction site inspected reinforcing and form placements, took slump tests, made test cylinders, checked air content, and recorded weather conditions. Except as noted, test cylinders were molded, cured, capped, and tested in accordance with Proposed ACI 301 except that one of the three cylinders was tested at 3 days and the remaining two at 28 days. For the containment shell, a set of four cylinders was made for each 50 cubic yards or fraction thereof placed in any one day.

One cylinder was tested at 3 days, another cylinder at 7 days, and the remaining two cylinders at 28 days. Slump tests were made at random with a minimum of one test for each 10 cubic yards of concrete placed. Also, slump tests were made on the concrete batch used for test cylinders.

In the event that concrete was poured during freezing weather or when a freeze was expected during the curing period, an additional cylinder was made for each set and was cured under the same conditions as the part of the structure that it represented.

**<u>Test Evaluation</u>**

The evaluation of test results were in accordance with Chapter 17 of Proposed ACI 301. Sufficient tests were conducted to provide an evaluation of concrete strength in accordance with the specification.

**<u>Deficient Concrete</u>**

Whenever it appeared that tests of the laboratory cured cylinders failed to meet the requirements set forth in the specification, the engineer and/or Testing Laboratory had the right to:

a. Order changes to the proportions of the mix to increase the strength.
b. Require additional tests of specimens cured entirely under field conditions.
c. Order changes to improve procedures for protecting and curing the concrete.
d. Require additional tests in accordance with "Methods of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete," ASTM C42-64.

If these tests failed to prove that the questionable concrete was of the specified quality, the contractor replaced the concrete work as directed.

#### *3.8.1.6.1.3 Concrete Suppliers*

Initially, concrete for Ginna Station was supplied from the Penfield Plant of the Manitou Construction Company. This plant was a relatively new "Rex" plant made by Rex Chain Belt Inc. of Milwaukee. Its capacity was about 100 cubic yards per hour. Operation was partially automated and controlled from a central console.

Punched cards were prepared for the various mixes to be supplied. The operator inserted the proper card for the mix required, set a dial for the quantity of concrete desired, and the machine measured out the ingredients automatically. Measurements could be observed on 2-ft diameter indicating dials in the control room as follows:

Cement: 0-6000 lb in 5-lb graduations.

| | |
|---|---|
| Sand and gravel: | 0-30,000 lb in 30-lb graduations. |
| Water: | 0-3000 lb in 3-lb graduations. |

The ingredients for the mix could easily be measured and recorded to within 1% of the true values. The State of New York purchased concrete from this plant. Their inspectors made periodic checks and required aggregate measurements within 2% and cement measurements within 1%. All provisions for storage precision of measurement complied with ASTM C94-64, Standard Specifications for Ready-Mixed Concrete.

The bulk of the concrete for the containment was supplied from the Walworth Plant of the Manitou Construction Company.

Technical details of this plant were as follows:

- Rex type AD dry batch plant.
- 100 yards/hr - maximum 150 yards/hr.
- Six-compartment aggregate bin.
- Eight-compartment batcher with dial scale.
- Two-compartment 600 bbl. cement silo.
- Eight-yard cement batcher with dial scale.
- 640-gallon water weight batcher with dial scale.

The plant provided fully automatic batching using a punch card system. All weights as well as time of batch were recorded on the card. Accuracy of the scale was ±0.5%. In a 1-day run, the accumulated weights reconciled to within 5 lb as an average. All recording scales had visual dials which could be observed by the inspector. Moisture probes were embedded in the bins to determine moisture and automatic compensations were made to maintain the proper water-cement ratio. Temperature of the concrete was controlled by heating with closed steam pipes located in the bins or cooling by control of aggregate temperature. Only Type II cement was being stored and used at the Walworth Plant.

#### *3.8.1.6.1.4 Concrete Specifications*

The Ginna specification for structural concrete included the Proposed ACI Standard Specifications for Structural Concrete for Buildings, as prepared by ACI Committee 301 and presented in the Journal of the ACI, February 1966, Proceedings, Volume 63, No. 2. At the time the specification was issued, ACI 301-66 was not yet formally released. Nevertheless, ACI 301-66 contained no significant changes from the proposed standard used for the Ginna specifications. The proposed ACI standard was either equaled or exceeded in all cases. Significant requirements that supplement or differ from those in the proposed ACI standard include the following which has been extracted from the Ginna specification:

The structural concrete for the containment shell including the ring girder, cylindrical walls, and dome shall have a minimum ultimate compressive strength of 5000 psi in 28 days.

The determination of the water-cement ratio to attain the required strength shall be in accordance with Method 2, Section 308(b) of Proposed ACI 301.

All cement shall be Portland Cement conforming to "Specification for Portland Cement," ASTM C150-64, Type II ...the cement shall be confined to a single brand with an established reputation for being uniform in character and shall be acceptable to the engineer.

All structural concrete shall be considered subject to potentially destructive exposure and shall contain air in amounts conforming with Table 304(b) of Proposed ACI 301.

A water-reducing densifier shall be added to all structural concrete with a required ultimate compressive strength equal to or greater than 3000 psi at 28 days.

Admixtures containing calcium chloride shall not be used.

Maximum water-cement ratio for various strengths of concrete shall be as follows:

| Compressive Strength (psi at 28 days) | Gallons of Water/Sack of Cement |
|---|---|
| 5000 | 5 |
| 3000 | 6 |

Ready-mixed concrete shall be mixed and transported in accordance with Specifications for Ready-Mixed Concrete, ASTM C94-65. The minimum amount of mixing in truck mixers loaded to maximum capacity shall be 70 revolutions of the drum or blades after all of the ingredients, including water are in the mixer. The maximum number of revolutions at mixing speed shall be 100; any additional mixing shall be at agitating speed.

The concrete shall be delivered to the site and discharge shall be completed within 1.50 hours or before the turn has been revolved 300 revolutions, whichever comes first, after the introduction of the mixing water to the cement and aggregates or the introduction of the cement to the aggregates. In hot weather the 1.50 hour time limit shall be reduced.

The proportion of water in each strength mix shall be adjusted at least every week as required by the content of surface moisture on the aggregates. Except for this adjustment, no changes in quantity of mixing shall be made without the approval of the engineer.

Each batch of concrete shall be recorded on a ticket which provides the date, actual proportions, concrete design strength, destination as to portion of structure and identification of transit mixer.

Concrete shall be protected against adverse weather conditions in accordance with Recommended Practice for Winter Concreting, new ACI 306-66, and Recommended Practice for Hot Weather Concreting, ACI 605-59, except that accelerators such as calcium chloride and antifreeze compounds shall not be used.

Curing methods detailed in proposed ACI 301 shall be used except that a method other than a curing compound shall be used for initial and final curing of concrete in the containment shell.

For the containment shell, a set of four cylinders will be made for each 50 cubic yards of fraction thereof placed in any one day.

Slump tests will be made at random with a minimum of one test for each 10 cubic yards of concrete placed.

Construction joint surfaces shall be prepared for the placement of concrete thereon by cleaning thoroughly with wire brushes, water under pressure, or other means to remove all coatings, stains, debris, or other foreign material.

The chloride content of mixing water shall not exceed 100 ppm and turbidity shall not exceed 2000 ppm.

On construction joint surfaces in the containment vessel, including all vertical joints in the cylindrical shell and all joints in the dome, an epoxy-resin compound shall be used to bond the new concrete with the abutting pour.

The limitation in Proposed ACI 301 for a maximum slump of 2 in. was not enforced. Enforced slump limitations were as listed in Table 305(a) of Proposed ACI 301.

A listing of all codes and standards referenced in specifications for the containment construction is included in Section 3.8.1.2.5. ACI 301-66 referenced above, provides that:

> The hardened concrete of joints in the exposed work, joints in the middle of beams, girders, joints, and slabs and joints in work designed to contain liquids shall be dampened but not saturated, then thoroughly covered with a coat of neat cement. The mortar shall be as thick as possible on vertical surfaces and at least 1/2-in. thick on horizontal surfaces. The fresh concrete shall be placed before the mortar has attained its initial set.

#### *3.8.1.6.1.5 Admixtures*

The ingredients of the structural concrete for the containment include the following admixtures:

a. Air-entraining admixture - This admixture is Darex AREA as manufactured by Grace Construction Materials and is a sulfonated hydrocarbon type with a cement catalyst conforming to ASTM C260.

b. Water-reducing retarder - This admixture is Plastiment as manufactured by Seka Chemical Corporation and is a non-air-entraining, water-reducing retarder with an active ingredient which is a metallic salt of hydroxylated carboxylic acid. This admixture conforms to ASTM C-494, Type D.

No user testing of admixtures was performed.

#### *3.8.1.6.1.6 Replacement Concrete for the 1996 Steam Generator Replacement*

Repair of the dome openings following the 1996 Steam Generator Replacement was accomplished using the existing liner plate sections, new reinforcing bars and new concrete. The replacement concrete, its constituents, batching, placement, and testing activities were considered safety-related. Design specifications for “Material Testing Services”, “Purchase of Safety-Related Ready-Mixed Concrete” and “Forming, Placing, Finishing and Curing of Safety-Related Concrete” (Bechtel Documents 22225-C-101(Q), 22225-C-311(Q) and 22225-C-302(Q)) controlled the work. Concrete mix designs were developed and tested to comply with the design specification of 5000 psi minimum compressive strength @ 7 days, slump 3” to 6” and air entrainment of 6% ± 1.5%. All mix design constituents were tested to meet design specifications. Independent verification testing was performed in addition to concrete supplier testing required for mix design qualification. B.R. Dewitt Inc. supplied the ready mixed concrete. Provisions for storage of specific mix design quantities of aggregate and cement were made prior to the pour date. The final design mix is listed below:

| Constituent | Weight (per cu. yd.) |
|---|---|
| Cement | 850 lb |
| Fly Ash | 130 lb |
| Fine Aggregate[a] | 915 lb |
| Coarse Aggregate[a, b] | 1680 lb |
| Rheobuild 1000[c] | 113 oz |
| MB-VR[c] | 19 oz |
| Water | 315 lb |

a. Weight is based on saturated, surface dry condition.

b. A 1:1 blend of ASTM C 33 #5 and #7 stone may be used to provide a gradation conforming to #57 stone.

c. Admixture dosage may be adjusted within manufacturer’s limits to meet field conditions.

The amount of “superplastizer” or high range water reducing admixture which was required for workable concrete was determined through mock-up testing. A containment dome mockup structure representing the full size actual dome opening with surrounding portions of dome was constructed for opening construction and repair activities. The mix design concrete was placed, cured and tested in the mock-up by the same methodology used on the actual containment prior to the 1996 Steam Generator Replacement outage. The mock-up proved valuable in adjusting admixtures for workability, maintaining truck mixing revolutions within acceptable limits, accessing forming and consolidation techniques, and verifying the mix design parameters.

The mock-up also proved valuable in determining logistical support such as: number of inspectors, technical support from admixture and ready-mix concrete suppliers, pumping controllers, labor support and batch plant communications.

In mid-May of 1996 concrete was placed in both containment dome openings using a Putzmeister BSS 44 series concrete pumper. The dome openings were boarded with reusable forms. Block outs for concrete placement and vibration were provided at approximately 4 ft on centers. After initial set the forms were stripped and the concrete was rubbed out and curing compound was applied. The design strength of the placed concrete was verified with all compressive cylinder breaks exceeding 5000 psi at 7 days.

#### 3.8.1.6.2 Mild Steel Reinforcement

The concrete reinforcement used was deformed bar of intermediate grade billet-steel conforming to the requirements of ASTM A15-64, Specifications for Billet-Steel Bars for Concrete Reinforcement, with deformations conforming to ASTM A305-56T, Deformed Bars for Concrete Reinforcement. Special large size concrete reinforcing bars were deformed bars of intermediate grade billet-steel conforming to ASTM-A408-64, Specifications for Special Large Size Deformed Billet-Steel Bars for Concrete Reinforcement. Reinforcing steel conforming to these specifications has a tensile strength of 70,000 psi to 90,000 psi and a minimum yield point of 40,000 psi. The large diameter reinforcing bar used in the 1996 Steam Generator Replacement dome opening repair was ASTM A615 which is an equivalent of the original reinforcement. The reinforcing was produced safety-related.

All splicing and anchoring of the concrete reinforcement was in accordance with ACI 318-63. The special large size bars were spliced by the Cadweld process with splices staggered as described below. Exceptions to this splicing process were made in the repair of the 1996 dome openings in limited locations. Where physical constraints prohibited the use of cadwelds (mostly in the hexagon opening corners), #18S reinforcing bars were welded together using a prequalified weld procedure.

The intermediate grade reinforcing steel is the highest ductility steel commonly used for construction. Certified mill reports of chemical and physical tests were submitted to the engineer, Gilbert Associates, Inc., for review and approval. Each bar was branded in the deforming process to carry identification as to the manufacturer, size, type, and yield strength, as shown in the following examples:

- B - Bethlehem.
- 18 - Size 18S.
- N - New billet steel.
- Blank - A-15 and A-408 steel.
- 6 - A-432 60,000 psi yield.
- 7 - A431 75,000 psi yield.

Because of the identification system and because of the large quantity, the material was kept separated in the fabricator's yard. In addition, when loaded for mill shipment, all bars were properly separated and tagged with the manufacturer's identification number.

Visual inspection of the bars was made in the field for inclusions.

The specifications stipulated that "Arc welding concrete reinforcement for any purpose including the achievement of electrical continuity shall not be permitted unless noted otherwise on the drawings."

The concrete cover required for reinforcing steel is tabulated in Table 3.8-7. A comparison is made between values for this plant and ACI requirements.

#### 3.8.1.6.3 Cadwell Splices

Tension splices for bar sizes larger than No. 11 were made with Cadweld splice. To ensure the integrity of the Cadweld splice the quality control procedures provided for a random sampling of splices in the field. The selected splices were removed and tested to destruction. For details of the destructive testing of Cadweld splices, refer to Section 3.8.1.7.1.

Where the special large size bars (i.e., 14S and 18S) were spliced, the Cadweld process was used so that the connection could develop the required minimum ultimate bar strength. Where the Cadweld splice was used, including the cylinder and dome, the splices were staggered a minimum of 3 ft. An exception to this practice was in the vicinity of the large openings. Where reinforcing bars were anchored to plates or shapes, such as is the case for the dome bars anchored into the cylinder and the interrupted hoop bars at penetrations, the Cadweld splices all occur in one plane. In addition to this, the cadweld splices made in 1996 for the Steam Generator Replacement dome opening repair were not staggered. This is typical around the perimeter sides of both dome openings. The dome openings were laid out such that at each side or face of the opening, two out of three layers of the #18S reinforcing bars project into the hole. Lapped splices are detailed in accordance with ACI 318-63.

Where Cadweld splices were used to anchor reinforcing bars to a structural steel member, as shown typically on Figure 3.8-4, a procedure of testing coupons was used to demonstrate that the welding process was under control. This procedure required each welder to initially make coupons, as shown on Figure 3.8-34, as a qualification procedure. The procedure was repeated at a frequency of one coupon for each 100 production units. Each coupon required testing of two Cadweld connections.

In addition, the welding procedure complied with the specifications of the American Welding Society and provided for 100% visual inspection of welds.

A sampling of 20 splices was initially tested to destruction to develop an average $(\bar{X})$ and standard deviation ($\sigma$). Thereafter sufficient samples were tested to provide 99% confidence that 95% of the splices met the specification requirements. As additional data became available, the average $(\bar{X})$ and standard deviation ($\sigma$) were updated and the quantity of samples revised accordingly.

The distribution established on this basis permitted the development of the lower limit below which no test data should fall. If the result of any test fell below this limit, the subsequent or previous splice was sampled. If this result was above the lower limit, the process was considered to be in control. If this result was again below the lower limit, the process average had changed and an engineering investigation was required to determine the cause of the excess variation and reestablish control.

### 3.8.1.6.4 Radial Tension Bars

Bars were received by Stressteel Corporation from Bethlehem or U.S. Steel along with certified mill reports of chemical and physical tests. The high-strength alloy steel bars were proof stressed to the minimum specified yield stress of 130,000 psi and then stress relieved in an oven at 700°F for 5 to 6 hours. Chemical test reports on each mill heat of steel used for bars and load-strain curves certifying physical properties of the stress relieved bars were provided. Other bar steel fabricated in the Stressteel plant was of equal or higher strength. Furthermore, the physical appearance of the bar steel, including smooth surfaces and threaded end, completely eliminated possible substitution with other construction materials in the field.

### 3.8.1.6.5 Containment Liner

#### *3.8.1.6.5.1 Fabrication and Workmanship*

The details of the fabrication and workmanship, with certain exceptions, conformed to the requirements of the ASME Nuclear Vessels Code for Class B Vessels. These exceptions included the following:

a. Materials - The steel plate for the main shell including the hemispherical dome, cylindrical walls, and base conformed to ASTM A442, Grade 60, and met the impact test requirements of ASTM A-300, except that the Charpy V-specimens were tested at a temperature of at least 30°F lower than the lowest service metal temperature. For the main liner shell, the lowest service metal temperature was calculated to be 48°F. Rolled sections including test channels and stiffeners conformed to ASTM A36.

b. Weld Inspection - Longitudinal and circumferential welded joints within the main shell, the welded joint connecting the hemispherical dome to the cylinder, and any welded joints within the hemispherical dome were inspected by the liquid penetrant method and spot radiography all in accordance with the ASME Unfired Pressure Vessels Code.

c. Opening Reinforcement - The liner is reinforced about all openings in accordance with ASME Unfired Pressure Vessels Code.

The ASTM A442, Grade 60, material has a specified minimum elongation in 8 in. of 20% and in 2 in. of 23%.

Quality control measures required by these standard specifications included the following:

**ASTM A442**

One tension test and one bend test shall be made from each plate as rolled. In addition, mill test reports will be obtained for heat.

**ASTM A300**

Each impact test value shall constitute the average value of three specimens taken from each plate as rolled (Note 3) with not more than one value below the specified minimum value of 15 ft-lb, but in no case below 10 ft-lb. Because of the material thickness, subsize specimens are used thereby altering the above-mentioned impact values to 12.5 and 8.5 ft-lb, respectively.

**ASTM A131**

Two tension and, except as specified in Paragraph (b), two bend tests shall be made from each heat of structural steel and steel for cold flanging, unless the finished material from a heat is less than 25 short tons when one tension and one bend test will be sufficient. If, however, material from one heat differs 0.15 in. or more in thickness, one tension test and one bend test shall be made from both the thickest, and the thinnest material rolled, regardless of the weight presented. When so specified in the order, a bend test may be taken from each plate of structural steel as rolled. Two tension and two bend tests shall be made from each heat of rivet steel.

When material is ordered for cold flanging and is subject to test and inspection by a ship classification society, one bend test shall be required from each plate as rolled.

#### *3.8.1.6.5.2 Penetrations*

The specifications for the containment liner further required that "The materials for penetrations including the personnel and equipment access hatches, as well as the mechanical and electrical penetrations, shall conform with the requirements of the ASME Nuclear Vessels Code for Class B vessels. All materials for penetrations shall exhibit impact properties as required for Class B Vessels."

The material for the penetrations conformed to ASTM A201-61T, Grade B Firebox, Tentative Specification for Carbon-Silicon Steel Plates of Intermediate Tensile Ranges for Fusion-Welded Boilers and Other Pressure Vessels, which was modified to ASTM A300-58, Standard Specification for Steel Plates for Pressure Vessels for Service at Low Temperature.

Quality control measures required for ASTM A201 included the following:

Two tension tests, one bend test, and one homogeneity test shall be made from each firebox steel plate as rolled. One tension test and one bend test shall be made from each flange steel plate as rolled.

#### *3.8.1.6.5.3 Welding*

The specifications for the containment liner further required the following quality control measures for welding:

The qualification of welding procedures and welders shall be in accordance with Section IX "Welding Qualifications" of the ASME Boiler and Pressure Vessel Code. Contractor shall submit welding procedures to the engineer for review.

The qualification tests described in Section IX, Part A, include guided bend tests to demonstrate weld ductility. All penetrations shall be examined in accordance with the requirements of the ASME Nuclear Vessels Code for Class B Vessels. Other shop-fabricated components, including the reinforcement about openings, shall be fully radiographed. All nonradiographable joint details shall be examined by the liquid penetrant method.

Full radiography shall be in accordance with the procedures and governed by the acceptability standards of Paragraph N-624 of the ASME Nuclear Vessels Code.

Methods for liquid penetrant examination shall be in accordance with Appendix VIII of the ASME Unfired Pressure Vessels Code.

In order to ensure that the joints in the liner plate and penetrations as well as all weld connections of test channels were leaktight, the specifications for the containment liner required that all welds "shall be examined by detecting leaks at 69 psig test pressure using a soap bubble test or a mixture of air and freon and 100% of detectable leaks arrested." These tests were preliminary to the performance of the initial integrated leak rate test which ensured that the containment leak rate was no greater than 0.1% of the contained volume in 24 hours at 60 psig.

### *3.8.1.6.5.4 Erection Tolerances*

Erection tolerances of the containment liner were:

| | |
|---|---|
| Overall out-of-roundness | ±3 in. |
| Deviation from round in 10 ft | 1-1/2 in. except at seams. |
| Overall deviation from the plumb line | ±3 in. |
| Deviation from line between tangent points at cylinder to dome transition and base to cylinder transition | ±3/4 in. |

Shell plate edges to butt for a minimum of 75% of wall thickness

The locations of penetrations with regard to azimuth location to be within ±1/2 in. measured on the circulate section. The horizontal and vertical dimensions associated with the radial dimension shall be ±1/2 in.

During erection, internal wind stiffness temporary braces were added to the liner to maintain roundness tolerances. This bracing was removed after pouring of the wall concrete. The liner erector's adherence to the tolerances specified for the liner were checked by means of a control survey.

### *3.8.1.6.5.5 Painting*

The containment liner was painted as follows:

a. All interior surfaces of the cylinder and dome (i.e., all exposed surfaces including the wall behind the insulation panels) had a minimum of a 2.5-mil coat of Carbozinc #11 Gray, as manufactured by the Carboline Company.

b. All other surfaces except the underside of the base liner had a minimum of a 1.5-mil coat of paint conforming with Federal Specification TT-P-645A, Primer, Zinc Chromate Alkyd.

#### 3.8.1.6.6 Elastomer Pads

The elastomer pads used for the containment number 320 and were manufactured to the following dimensions:

A. Plan area: 42 in. by 9 in.

B. Neoprene: two layers of neoprene each 11/16-in. thick.

C. Steel shims: an outer shim on each face with a minimum thickness of 16 gauge and one shim between the two neoprene layers of 10 gauge.

The neoprene has a nominal durameter hardness of 55. Physical requirements of the neoprene are shown in Table 3.8-8.

#### 3.8.1.6.7 Tendons

##### *3.8.1.6.7.1 Materials*

The prestressing system used for the containment is the BBRV system utilizing ninety 0.25-in. diameter wires. The wires are high tensile steel, that is, bright, cold-drawn, and stress-relieved conforming to ASTM A421-59T, Type BA, Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete, with a minimum guaranteed ultimate strength of 240,000 psi. The BBRV system uses parallel wires with cold formed buttonheads at the ends which bear upon a perforated steel anchor head, thus providing a mechanical means for transferring the prestress force. The buttonheads are formed by cold upsetting to a nominal diameter of 3/8 in. on the 1/4-in. diameter wire. The materials used for anchorage components were as follows:

| Item | Size | Material |
|---|---|---|
| Movable anchor head | 7-7/8 in. O.D. x 3-1/2 in. | C1141 heat treated |
| Fixed anchor head | 5-1/8 in. O.D. x 3-3/4 in. | C1141 heat treated |
| Bushing (adaptor for couplers) | 7-7/8 in. O.D. x 5-1/8 in. I.D. | C1045 |
| Couplers | 10-1/2 in. O.D. x 7-1/8 in. I.D. | C1018 |
| Bearing plate | 18-1/2 in. O.D. x 2-1/2 in. | A36 |
| Split shims | 8-1/2 in. O.D. x 1-1/2 in. wall | HFSM Tube C1026 |

The C1141 material is heat treated to Rockwell C30 to C33.

The material used for the exposed bearing plates at the upper end of the vertical tendons conformed to ASTM A36, Specification for Structural Steel, including the optional requirement of this specification of silicon killed fine grain practice for steel used at temperatures where improved notch toughness is important.

#### *3.8.1.6.7.2 Tests and Inspection*

All anchorage hardware was 100% visually inspected to ensure that no surface flaws, notches, and similar stress raisers existed. Hardness tests were performed on each anchor head to verify adequate heat treatment and strength. The tendon fabricator cut coupons from each end of each reel of wire, formed buttonheads, and tested the specimens. These tests were to ensure that the wire would rupture before failure of the buttonhead and that the wire would meet the physical requirements of ASTM A421. Coupons and the coils they represented not meeting the requirements were rejected. Records were maintained for each coupon test and for the tendons in which each coil of wire was used. Anchorage components were fabricated from materials specified on the manufacturer's parts drawings. Requirements for machining, tolerances, and heat treating were as specified on the parts drawings.

All buttonheads were visually inspected and a minimum of 10% of the buttonheads were randomly checked for size verification. Dimensions of the buttonheads were as follows:

a. Diameter equal to or greater than 0.372 in. and equal to or less than 0.388 in.

b. Length equal to or greater than 0.252 in. and equal to or less than 2.272 in.

c. A bearing surface on all sides.

Limitations on splits (cracks) in buttonheads were as follows:

aa. Splits are not to be inclined more than 45 degrees to the axis of the wire.

bb. Sum of the widths of all splits are less than 0.06 in. with inclinations less than 20 degrees to the axis of the wire.

cc. No more than two splits occur in buttonheads which have splits inclined more than 20 degrees but less than 45 degrees to the axis of the wire. In no event do the two cracks occur in the same place.

### 3.8.1.6.8 Liner Insulation

The inside surface of the liner may be inspected in the wall and dome area. However, the walls are covered by panels of thermal insulation to protect the liner in the event of an accident. Corrosion of the liner is not expected because the outside surface is in contact with concrete; the lower portion of the inside surface is protected from sweating by the insulation; and the entire liner is tied into the overall cathodic protection system. It is possible, however, to remove a section of insulation periodically to examine the liner if required.

The liner insulation is 1.25-in. thick Vinylcel, which is a rigid cross-linked polyvinyl chloride (PVC) foam plastic manufactured by Johns-Manville. Dimensions for full size sheets are 44 in. x 84 in. Sheet faces are finished with 0.019-in. thick sheets of type 304 stainless steel.

The sheets are attached to the steel liner with stainless steel studs (KSM #304 stainless #10-24). The full size sheets have six studs each. A 1.125-in. diameter neoprene backed stainless steel combination washer is placed outside the sheet over the stud and held in place by a self-locking stainless steel hexagonal head nut. Backs of the sheets are routed to fit over the test channels on the liner. Sheets are erected with the 44-in. dimension vertical and vertical joints are staggered. The joints at the base of the routed edges are taped with 3/8-in. wide tape and the routed area is filled with Dow Corning Sealant #780 silicone rubber base sealant or equivalent to make a flush finished joint.

At penetrations or other irregular surfaces, the sheets are cut to fit and the edges are beveled and caulked with the sealant. A similar caulked joint is provided at the extremities of the insulated area.

If for any reason a panel or section must be removed, it is possible to do so by cutting along the joints and removing the fastening nuts. Replacement would only involve reapplication of nuts and new sealant.

The PVC material is chemically compatible with steel and no degradation of either material because of contact and/or environment results. The sealant is an acid-free inorganic type; again, no chemical reaction results. The sealant is waterproof and remains pliant down to 80°F and does not soften up to 350°F.

The reports of tests performed to ensure meeting the functional requirements are included in Section 3.8.1.7 and Appendix 3E.

### 3.8.1.7 Testing and Inservice Inspection Requirements

#### <u>3.8.1.7.1 Construction Phase Testing</u>

Preoperational inspections and tests were performed in several stages which finally led to the structural proof and integrated leak rate tests. Inspections and tests of the structural elements of the containment vessel included the liner, tendons, concrete and concrete reinforcement, elastomer pads, and rock anchors.

##### *3.8.1.7.1.1 Liner*

Longitudinal and circumferential welded joints within the main shell, the welded joint connecting the dome to the cylinder, and all joints within the dome were inspected by the liquid penetrant method and spot radiography. All penetrations including the equipment access door and the personnel locks were examined in accordance with the requirements of the ASME Nuclear Vessels Code for Class B Vessels. All other shop-fabricated components including the reinforcement about openings were fully radiographed. All other joint details were examined by the liquid penetrant method. Full radiography was performed in accordance with the procedures and governed by the acceptability standards of Paragraph N-624 of the ASME Nuclear Vessels Code. Spot radiography was performed in accordance with the procedures and governed by the standards of Paragraph UW-52 of the ASME Unfired Pressure Vessels Code. Methods of liquid penetrant examination were in accordance with Appendix VIII of the ASME Unfired Pressure Vessels Code. All piping penetrations and personnel locks were pressure tested in the fabricator’s shop to demonstrate leaktightness and structural integrity.

A prototype of the air-cooled penetrations was tested to verify thermal and hydraulic design calculations.

All accessible weld seams on the liner were spot radiographed, except for penetrations which were fully radiographed. Spot radiography was performed in accordance with Section UW-52 of the ASME Unfired Pressure Vessels Code, which required that:

> One spot shall be examined in the first 50 ft of welding in each vessel and one spot shall be examined for each additional 50 ft of welding or fraction thereof. Such additional spots as may be required shall be selected so that any examination is made of the welding of each welding operator or welder. The minimum length of spot radiograph shall be 6 in.

The liner weld seams were also examined by pressurizing the test channels to design pressure (60 psig) with a mixture of air and freon, and checking all seams with a halogen leak detector. All detectable leaks were corrected by repairing the weld and retesting.

#### *3.8.1.7.1.2 Prestressing Tendons*

The rock anchors and wall tendons for the containment were inspected by both the supplier, Joseph T. Ryerson and Son, Inc., and the prime contractor, Westinghouse Atomic Power Division.

Ryerson performed all tests enumerated in Section 3.8.1.6, and reports are retained in the Quality Control file.

Westinghouse did the following:

a. Submitted certified mill test reports to the designer, Gilbert Associates, Inc., for their review and comment.
b. Monitored the shop procedures and inspection by Ryerson.
c. Inspected each tendon at the Ryerson shop before shipment to ensure conformance to specifications and proper preparation for shipment.

In addition to the foregoing, a test was performed on each item of anchorage hardware to confirm that it was capable of developing the ultimate capacity of the tendon. Reports of these tests are included in Appendix 3D.

#### *3.8.1.7.1.3 Concrete Reinforcement*

Tension splices for bar sizes larger than No. 11 were made with the Cadweld splice designed to develop the ultimate strength of the bar, or with the use of deformed bars conforming to ASTM A408-64, Intermediate Grade (minimum tensile stress of 70,000 psi). A sampling of 20 splices was initially tested to destruction to develop an average ($\bar{X}$) and standard deviation ($\sigma$). Sufficient samples were tested to provide a 99% confidence level that 95% of the splices would meet the specification requirements. The average of all tests also was required to remain above the minimum tensile strength. As additional data became available, the average and standard deviations were updated. The actual frequency of testing carried out was one specimen for each 25 splices made for each crew for the first 250 splices made by that crew and one test for each 100 splices thereafter. In addition, where deformed bars were attached to structural steel members, specimens were made and tested to ensure that the weld of the splice to the member did not fail before the rebar or the splice. The frequency of testing these specimens was the same as that for the normal splices. A plot of the results of all tests over a period of time is shown in Figure 3.8-35.

No arc welding was permitted on the Class I structures for splicing reinforcing bars during the original construction. All rebar splices of the major reinforcement in the containment structure (i.e., special large size bars) were made with the Cadweld process. There were no special requirements for chemical composition of reinforcing bars beyond the requirements of ASTM A15 and A408. Generally, no tack welding of reinforcing bars was permitted. The only exception involved those locations specifically shown on the drawings (refer to Figure 3.8-4) which were located where rebar strength was not required and bars were provided solely to provide electrical continuity below ground water level.

In sampling the Cadweld splices a test was concurrently performed on the rebar. Where the rebar failed prior to the splice, a check was provided on the ultimate strength of the rebar, thus providing a check on conformance with the manufacturer's certifications and the ASTM standards. In addition, certified mill test reports were received from the rebar supplier and checked for conformance with specification requirements. The splice and mill test reports are retained in the Quality Control file.

Replacement reinforcement for the dome openings constructed in the 1996 Steam Generator Replacement was #18S ASTM A615 Grade 60. The reinforcing bars were connected primarily with T-series Grade 60 Cadweld splices as manufactured by Erico Products. Prior to starting production splicing, a member of each splicing crew was qualified for performing cadwelds in each of three positions; horizontal, vertical and diagonal. During production, a specified number of sister splices were made in-place next to production splices, under the same conditions, and by the same crew. For each crew the following tensile tests on the sister splices were made:

A. Test one sister splice for the first 10 production splices.

B. Test four sister splices for the next 90 production splices.

C. Test three sister splices for the next and subsequent units of 100 splices.

The cadweld sister splices were tested to failure. All splices were determined to be capable of developing cadweld design criteria of 1.25 times the minimum yield strength of the replacement reinforcement which was 60,000 psi. The limited number of welded splices were performed using a prequalified arc welding procedure and visually inspected in accordance with AWS D.1.4.

##### *3.8.1.7.1.4 Concrete*

The prime contractor obtained the services of a testing agency which made preliminary determinations of controlled mixes, using the materials proposed and consistencies suitable for the work, in order to determine the mix proportions necessary to produce conformance to the type and strength requirements. During concrete operations, the testing agency maintained an inspector at the batch plant who certified the mixed proportions of each batch delivered to the site and sampled and tested periodically all concrete ingredients and monitored aggregate surface moisture. One or more inspectors were retained at the construction site to take slump tests, make test cylinders, check air content, and record weather conditions. For the reactor containment, a set of no less than four cylinders was made for each 50 cubic yards or fraction thereof placed in any day. Two cylinders each were tested in 7 days and in 28 days. Slump tests were made at random, with a minimum of one test for each 10 cubic yards of concrete placed. Also, slump tests were made on the concrete batch used for test cylinders. A running average of test results through September 26, 1967, for 5000 psi concrete is shown in Figure 3.8-36.

Acceptance standards for compressive strength were based upon ACI 301, Section 1703 which stated that: "Strengths of ultimate strength type concrete and prestressed concrete shall be considered satisfactory if the average of any three consecutive strength tests of the laboratory cured specimens representing each specified strength of concrete is equal to or greater than the specified strength, and if not more than 10% of the strength tests have values less than the specified strength."

Acceptance standards for slump were based upon those limits stated in ACI 301, Table 304(a) which established a maximum slump of 3 in. for reinforced and plain footings, caissons, and substructure walls; 4 in. for slabs, beams, reinforced walls, and building columns; and also established a minimum slump of 1 in.

Figure 3.8-36 provides a moving average of compressive strength for 5000 psi concrete on five previous test groups. There were two periods of time when these averages fell below the specified 5000 psi, 28-day compressive strength. The occasions when this occurred involved the use of the first mix in areas requiring by design only 3000 psi concrete, namely the containment base slab and the turbine pedestal. The mix was then modified to produce the more satisfactory results thereafter reflected on the chart of the running average. At no time did in-place concrete fail to meet the specification requirements.

Type II cement, modified for low heat of hydration, was used to minimize shrinkage.

Grab samples were taken periodically at the batch plant, upon delivery of cement. Each sample was tested by the testing laboratory for conformance to ASTM C150, and the results were also compared with the certificate supplied with each delivery of cement.

### *3.8.1.7.1.5 Elastomer Bearing Pads*

Tests were performed on elastomer specimens to ensure compliance with requirements for: (1) original physical properties including tear resistance, hardness, tensile strength, and ultimate elongation; (2) change in physical properties due to overaging; (3) extreme temperature characteristics; (4) ozone cracking resistance; (5) oil swell, and (6) shear modulus. In addition, two full size pads were tested, one for creep and one for ultimate load. Specimen No. 1 was initially placed under essentially a constant compressive load of 1000 psi (the design pressure) for 4 days to measure creep. This pad was then loaded up to 2000 kips (5.3 times design load) when the test was terminated without failure. Specimen No. 2 was similarly loaded up to 2000 kips without failure. The rebound of the pads after the 2000-kips load was removed was essentially complete. A summary of the test results is shown in Figures 3.8-37 and 3.8-38.

### *3.8.1.7.1.6 Rock Anchor Tests*

Three scaled-down test rock anchors were installed to demonstrate the holddown capacity of the rock and the capacity of the bond between rock and grout.

Two tests were made on rock anchor A, which was installed at the center of the proposed containment. The first test, called test A-1, was to determine rock hold-down capacity. The set-up for test A-1 is illustrated in Figure 3.8-39. The beam support piers were located beyond the assumed influence circle of rock having a diameter of 23 ft 6 in. An independent frame was erected to obtain deflection measurements on the concrete pier at the anchor. This placed all supports for lifting as well as measuring devices outside the influence circle of rock. Dial gauges were used to measure the movement of the concrete pier and the anchor head. The test load was applied with a 150-ton jack mounted on the beams spanning the test anchor.
Measurements of the jacking force were made with a dynamometer, calibrated immediately before the test. The second test on rock anchor A (test A-2) and the tests on rock anchors B and C, also installed near the center of the proposed containment, were made to demonstrate bond capacity. The set-up for test A-2 and for rock anchors B and C was an arrangement whereby the jack was supported directly by the concrete pier adjacent to the test anchor.

Rock anchor A consisted of twenty-eight 0.25-in. diameter wires grouted for a length of 4 ft 5.5 in. in a 3.5-in. diameter hole. All test rock anchors were oversized so that the test load of 100 kips would develop only about 30% of the ultimate capacity of tendon wires while developing a bond stress of 170 psi, which is the design stress for the containment rock anchors. This permitted testing bond stresses well in excess of design (170 psi) without exceeding ultimate wire stresses. The test procedure for test A-1 is described in the following paragraph.

The anchor was loaded in 20,000-lb increments to 100,000 lb. The load was maintained at each increment for 15 min prior to taking measurements for elongation of the tendon and elevations of the concrete pedestal and adjacent rock surface. Because the anchor head appeared from visual observation not to have lifted off at the 100,000-lb load, the load was increased to 110,000 lb, at which point lift-off was apparent. Subsequent review of measurements on the movement of the anchor head indicate that actual lift-off occurred between 80,000 lb and 100,000 lb, as would be expected.

In test A-2 and the tests on rock anchors B and C, the tendon was jacked from the concrete pier immediately adjacent to the tendon.

Table 3.8-9 lists measurements taken during test A-1. Figures 3.8-40 through 3.8-42 show plots of load versus elongation deflection for all tests.

The application of a test load of 110 kips to rock anchor A (as indicated by the results of test A-1 shown on Figure 3.8-40) is equivalent to 137.5% of the calculated hold-down capacity assumption used in the design. The plot of load versus elongation deflection for rock anchor A tests A-2 (see Figure 3.8-40) and B and C (see Figures 3.8-41 and 3.8-42) indicate a factor of safety against slippage by the grout and rock of at least 2.0 (200-kips load versus 100-kips design load) for rock anchor B. If slippage occurred within the grout, the factor of safety against failure is even greater. The plot of load versus elongation for rock anchor A shows an apparent discontinuity which is indicated by a dashed line on Figure 3.8-40. This represents settlement of the concrete pier adjacent to the rock anchor when the load was transferred from the lifting frame used in test A-1 to the lock nut that bore on the concrete pier.

##### *3.8.1.7.1.7 Large Opening Reinforcements*

Testing of large opening reinforcements is discussed in Appendix 3B.

##### *3.8.1.7.1.8 Liner Insulation*

Tests were conducted on the Vinylcel for confirmation of the following material properties:

- Conductivity factor (Btu/hr $ft^2$/°F/in.), per ASTM C177-63, at 75°F, 100°F, 150°F.
- Compressive yield strength (psi), per ASTM C165.
- Moisture vapor permeability (per inch) by dry cup, per ASTM C355-64.
- Shear strength (psi).
- Shear modulus (psi), per ASTM C273-61.
- Compressive modulus (psi), per ASTM C165-54.
- Density (lb/$ft^3$), per ASTM D16 22-63.
- Average coefficient of linear expansion (in./in./°F) for temperature range.

Results of these tests are included in Appendix 3E. Also included are the results of a test to determine resistance to flame exposure, plus the results of an analog simulation of the insulation system due to the pressure and temperature transients associated with the 50% overpressure condition.

#### <u>3.8.1.7.2 General Description of the Structural Integrity Test</u>

##### *3.8.1.7.2.1 Pressurization*

After completion of the entire containment, a structural integrity air pressure test at 115% of design pressure was maintained for 1 hour.

The pressurization of the containment was done at 5 psi increments. Readings and measurements were taken at 35 psig, 50 psig, 60 psig, and the final test pressure of 69 psig. Except for the final pressure level, the vessel pressure was always increased 1 psi above the level at which measurements were made. The pressure was then reduced to the specified value and observations made after a delay of at least 10 min to permit an adjustment of strains within the structure.

Because the structure is so large, displacement measurements (absolute or relative) could be made with precision and could be used as confirmation of the previously calculated response. The test program further included a visual examination of the containment during pressurization to observe deformations and to demonstrate that no distortions occurred of a significantly greater magnitude than those calculated in advance based upon the same analytical models used for the design of all structural elements for the loading combinations described in Section 3.8.1.2.

Prior to the test, a table of predicted strain, deflection, and rotation values was developed for an internal pressure of 69 psig, which was the pressure of the structural proof test, as well as those lower pressure levels used to take measurements. Strain, displacement, and rotation predicted from the analytical model for an internal pressure of 69 psig were used as a basis for verifying satisfactory structural response. Although strain gauges were installed on designated areas of the liner, concrete reinforcement, and tendon shims, the analytically derived strains were not used as acceptance figures for the actual values. The obtained values were analyzed and evaluated to determine magnitude and direction of principal strains. If the test data included any displacements which were in excess of the predicted extremes, such discrepancies required resolution including review of the design, evaluation of measurement errors and material variability and, conceivably, exploration of the structure. Prior to the test, maximum anticipated crack widths were predicted. If any crack widths occurring during the test were in excess of predicted values, such discrepancies were required to be satisfactorily resolved in a similar manner as for displacements. The anticipated values for crack widths and a complete report on other anticipated measurements were provided before the test.

#### *3.8.1.7.2.2 Measurements*

During the test at each specified pressure level, a series of measurements and observations were made as follows:

a. Radial displacements of the cylinder at three elevations and at three azimuths in order to ascertain if the response was symmetrical and to verify the estimated response due to average circumferential membrane stresses. On the same three azimuths, horizontal displacements were measured immediately above and below the dome to cylinder transition.

b. Vertical displacement of the cylinder at the top relative to the base ring girder at three azimuths to determine the vertical elongation of the side wall and average tendon strains.

c. Cylinder base rotation and displacement at three azimuths to verify hinge action and symmetrical response.

d. Horizontal and vertical displacements of the reinforcing ring around the equipment access hatch opening.

e. Strain of reinforcing bars near the concrete surface around the equipment access opening. Small access ports to selected reinforcing bars were left in the concrete to mount strain gauges just prior to the structural test. These gauges were provided only in those places where this limited exposure of the steel reinforcement would not be injurious to the behavior of the structure under test. Following completion of the structural test the access ports were sealed.

f. The liner was instrumented with electrical resistance strain gauges in the region of several typical penetrations as well as a region unaffected by geometric discontinuities. Redundancy in strain readings were accomplished by placing strain gauge rosettes at several points about the penetration openings and by instrumenting four penetrations which were subjected to similar loadings and restraints.

g. To determine principal stresses, in magnitude and direction, the gauges employed were in the form of 120-degree rosettes. Associated with the gauges was the application of a strain-indicating brittle lacquer to qualitatively augment the local values indicated by the gauges and to show the existence of a symmetrical, or otherwise, overall stress pattern.

h. Horizontal displacements were measured immediately above and below the dome to the cylinder discontinuity. Strain gauges were installed on reinforcing bars near the exposed concrete surface above and below the discontinuity. Detailed concrete crack observations were made in the immediate vicinity of the discontinuity.

i. Load cells were used on four tendons at the top anchorages to verify the stress variation over the range of test pressures. Also, strain measurements were made on a limited number of bearing plates at the top anchorages.

In addition to displacement and strain data, observation for cracks in the concrete was made in the following manner:

aa. The containment was visually inspected for cracks and crack patterns.

bb. At selected locations, the surface was white-washed for detailed measurements of spacing and width of cracks to verify that local strains were not excessive. These selected locations included:

1. Quadrant of reinforcing ring for large opening.
2. Cylinder to dome transition.
3. The cylinder, where circumferential membrane stresses are maximum and where flexural stresses are maximum.

The movable (top) anchor heads of the sidewall tendons were inspected for wires which had failed. A ruptured wire would be readily evident because the energy release upon rupture causes the wire to noticeably rise and remain loose.

The maximum calculated radial displacement due to the test pressure of the cylinder was 0.62 in., and a minimum radial displacement calculated at the hinge (base of cylinder) was 0.06 in. Local variation in geometry of the structure made it extremely doubtful that uniform and predictable strain measurements would be achieved from the strain gauges installed on designated areas of the liner, concrete reinforcement, and tendon shims. Therefore, specific strain measurements could not be reasonably established as acceptance standards.

The program for instrumentation of the containment structure was established to permit installing the instruments immediately before the test, thereby precluding the necessity of providing unusual protection against construction abuse and weather. Shielding enclosures were provided on those external surfaces of the containment vessel where strain gauges were to be located.

Instrumentation for making displacement measurements included dial gauges, scales, and theodolites used to read prepositioned targets. All gauges and targets were installed immediately prior to the test.

All measuring devices, including theodolites and dial gauges, produced measurements of sufficient precision to ascertain satisfactory structural response. For a theodolite located approximately 150 ft from the targets, it was possible to measure within 0.01 in. For a maximum expected measurement of radial deflection of 0.62 in., a precision of 0.02 in. (twice the expected measuring accuracy) should be satisfactory. Dial gauges used at the hinge detail could measure to the nearest 0.001 in. which was sufficient to define the displacement and rotation of the hinge. Where it was practical to use dial gauges for greater accuracy, they were used to make displacement measurements.

### *3.8.1.7.2.3 Test Pressure Justification*

The 115% design pressure used in the structural proof test was justified for the following reasons:

a. The principal tensile stress in the liner during a simultaneous loss-of-coolant accident (60 psig pressure) and 0.08g earthquake amounts to 19.9 ksi assuming the liner participates fully in taking earthquake shears.

   The tensile stress in the liner under the 69 psig test for structural integrity is 26.5 ksi. This means that before the leak rate test at 60 psig the liner has been subjected to tensile stresses in excess of those which would occur during a simultaneous loss-of-coolant accident and 0.08g earthquake. During the leak rate test the tensile stress in the liner is 23 ksi. During a loss-of-coolant accident, without earthquake, the tensile stress is 19.2 ksi.

b. The principal tensile stress in the outer circumferential reinforcement band during a loss-of-coolant accident and simultaneous 0.08g earthquake is 26.4 ksi. The principal tensile stress in this reinforcement during test for structural integrity is 26.5 ksi.

c. The average stress in a tendon during a loss-of-coolant accident is 145.2 ksi, the average stress in a tendon during tests for structural integrity is 145.5 ksi.

d. The test pressure conforms with the recommendations of Oak Ridge National Laboratory regarding testing of concrete vessels (Reference: ORNL - NSIC - 5, Volume II U.S. Reactor Containment Technology, page 10.8).

### *3.8.1.7.2.4 Test Results*

See Section 14.6.1.6.10 for the results of the preoperational structural integrity test of the containment.

### *3.8.1.7.2.5 Containment Return to Service Testing Post 1996 Steam Generator Replacement*

After placement, curing and acceptance of the 1996 Steam Generator Replacement dome opening repair concrete, the structure underwent a full pressure Integrated Leak Rate Test (ILRT) and a partial Structural Integrity Test (SIT). These tests were combined to satisfy the requirements of 10CFR50 Appendix J, “Primary Reactor Containment Leakage Testing for

Water-Cooled Power Reactors," and to demonstrate that the containment design and dome opening repairs are adequate to withstand postulated pressure loads. The containment interior and exterior were structurally inspected for cracks and anomalies prior to pressurization and after depressurization. Embedded strain gages were installed on the replacement rebar and monitored throughout the testing. The ILRT test pressure was 60 psig. This test was performed and accepted prior to increasing the pressure for the SIT. The original SIT pressure was 69 psig which represented 115% of the design pressure. A test pressure of 72 psig was used in 1996 which supports a potential increase in the design pressure to 62 psig.

The repaired dome openings and adjacent areas were monitored during the SIT. Crack mapping was performed in these areas prior to, at pressurization, and after depressurization. Vertical growth of the structure was monitored at the spring line and the dome apex. Radial growth measurements were taken at defined elevations at three azimuth locations. Predicted rebar strains, design vertical and radial displacements, and crack size and length criteria were used as the test acceptance criteria.

### <u>3.8.1.7.3 Postoperational Surveillance</u>

#### *3.8.1.7.3.1 Leakage Monitoring*

Postoperational leakage rate testing is discussed in Section 6.2.6 and in Section 14.6.1.6.9.

#### *3.8.1.7.3.2 Initial Tendon Surveillance Program*

Means are provided to allow surveillance of all upper tendon terminations. The initial tendon surveillance program incorporated the following:

a. Visual inspection of all tendon terminations was made after the structural integrity test. A record was kept of all broken wires.

b. A number of tendons equally spaced around the containment were to be inspected 6 months, 1 year, 3 years, and 10 years after the structural test. If more than 1% of additional wires were found broken, additional equally spaced tendons were to be inspected until it was established that less than 1% of all wires inspected were broken.

c. A prestress confirmation lift-off test is made on the tendons referred to in item 2 above, to compare relaxation of tendons with a predicted curve. Tests were to be conducted 6 months, 1 year, 3 years, and 10 years after tensioning. This phase of the program provides for obtaining a lift-off reading by using a hydraulic jack to just lift the upper anchor head off the shim. This procedure provides a determination of the stress level in the tendons and also is used to confirm previously predicated stress losses including steel relaxation and concrete creep. Before reseating the tendon, the hydraulic jack is used to lift the termination sufficiently to apply an additional stress in the wires equal to that applied during pressurization of the shell (6%) to verify its ability to withstand additional stresses applied during accident conditions.

d. Each of 40 tendons includes an extra unstressed 0.25-in. diameter wire specimen, obtained from a reel represented in the tendon. The specimen extends from the top anchor head down to approximately elevation 240 ft. One wire is removed on an annual basis for examination. This provides a periodic check on tendon corrosion.

The initial structural integrity test of the containment was conducted at 69 psi. Displacement measurements were recorded during this test for pressures of 35, 50, 60, and 69 psig. The continuing structural integrity of the containment is verified by the tendon surveillance program and displacement measurements taken during subsequent leak rate tests. General agreement with initial measurements indicates a structural response similar to the initial tests. This, plus the tendon surveillance program, establishes a high degree of assurance that the integrity of containment has been maintained.

The initial 10-year tendon surveillance program has been completed as follows:

| | |
|---|---|
| Prestressing of rock anchors | Fall 1966 |
| Prestressing of tendons | March-April 1969 |
| Structural integrity test | April 1969 |
| 6-month inservice inspection | October 1969 |
| 1-year inservice inspection | May 1970 |
| 3-year inservice inspection | May 1972 |
| 8-year inservice inspection | June 1977 |
| 10-year inservice inspection | October 1979 |
| Retensioning of tendons - new time zero | June 1980 |

In June 1980, retensioning of 137 out of the total of 160 tendons was done. The 23 tendons that were not included in the retensioning program had been retensioned in May 1969, approximately 1000 hours after their original stressing.

#### *3.8.1.7.3.3 Current Tendon Surveillance Program*

The current tendon surveillance program includes the following:

a. Commencing with the new time zero, June 1980, an inspection for the presence of broken wires and prestress lift-off tests are to be conducted after 1 year, 3 years, and 5 years and every 5 years thereafter. The 1-year inspection was conducted in July 1981, the 3-year inspection was conducted during July and November 1983, and the 5-year inspection was conducted in August 1985. Inspections continue every 5 years (with a 25% extension allowed per Technical Specifications).

b. Fourteen tendons, equally spaced around the containment are to be inspected for the presence of broken wires. The acceptance criteria for the inspection are that no more than a total of 38 wires in 14 tendons are broken and that not more than five broken wires exist in any one tendon. If more than 38 broken wires are found, all tendons are to be inspected. However, if more than 20 wires in 14 tendons have been broken since the last inspection, all tendons are to be inspected. If inspection reveals more than 5% of the total wires broken, the containment must be declared inoperable.

If more than five broken wires are found in any one tendon, four immediately adjacent tendons (two on each side of the tendon containing more than five broken wires) are to be inspected. The acceptance criterion then will be no more than four broken wires in any of the additional four tendons. If this criterion is not satisfied, all of the tendons are to be inspected and if more than 5% of the total wires are broken, the containment must be declared inoperable.

c. Prestress confirmation lift-off tests are to be performed on the 14 tendons identified in item b. above.

   The lift-off readings are obtained in the same manner as described above for the initial tendon surveillance program. Before reseating a tendon, additional stress (6%) will be imposed to verify the ability of the tendon to sustain the added stress applied during accident conditions. If the average stress in the 14 tendons is less than 144,000 psi (60% of ultimate stress) equivalent to 636 kips, all tendons are to be tested for prestress and retensioned, if necessary, to a stress of 144,000 psi (636 kips). If a tendon fails its lift-off test lower limit of 636 kips, the two adjacent tendons are tested. If either adjacent tendon fails its lift-off test, a NRC report is required due to possible abnormal degradation of containment. If both adjacent tendons pass their test and no more tendons fail their test, the single tendon failure is considered unique and acceptable.

d. One unstressed wire specimen is removed during each surveillance for examination for corrosion as in the initial tendon surveillance program. The wire is also tensile tested. Failure of the wire below its ultimate strength identifies an unacceptable wire and requires NRC notification.

e. A visual inspection of the top anchorage assembly hardware for the 14 tendons identified in item b. above is also performed. The surrounding concrete is also inspected during integrated leak rate tests when the containment is at its maximum test pressure. Finally, the filler grease for the 14 tendons is inspected and tested. If significant deterioration of any tendon anchorage assembly, local concrete, or filler grease is observed, NRC notification is required.

f. If NRC notification is required, the report should include a description of the tendon condition, the condition of the concrete (especially at tendon anchorages), the inspection procedure, the tolerances on concrete cracking, and the measures being implemented if the tolerances are exceeded.

#### *3.8.1.7.3.4 Current Tendon Surveillance Program Results*

The 3-year surveillance of the containment vessel tendons performed after retensioning was during July and November 1983. A representative sample of 18 tendons was selected. The results following the surveillance are documented in the containment vessel tendon surveillance report submitted to the NRC by *Reference 24* and the conclusions are summarized as follows:

a. The results of the completed tendon surveillance, in which 18 sample tendons were lift-off tested, indicated that the forces in the tendons are maintained at the levels expected, and that no abnormal force losses have occurred. The agreement between the actual and predicted tendon forces is better than that which is generally experienced on other containments.

b. Based on the forces measured in the sample tendons, the average force level of the tendons in the containment is 711 kips, which exceeds the minimum required value of 636 kips appearing in the Tendon Surveillance Program by 11.8%.

c. Based on the results of the 1983 surveillance, a recommendation was made for future surveillances that the predicted tendon force calculations be based on a 40-year wire relaxation of 16%, applicable to all tendons, and multiplied by factors to account for the retensioning effect.

d. From the results of the surveillance and a comparison of actual stress relaxation with that predicted, no future retensioning of tendons should be required for the remainder of the expected plant life.

In the safety evaluation report based on the results of the 1981 and 1983 lift-off tests, the NRC concluded that it appears that the tendon forces are stable and that there are no abnormal tendon force losses; and that the adequacy and integrity of the containment is ensured.(*Reference 52*).

The 5-year surveillances of the containment vessel tendons were performed in August 1985, August 1990, October 1995, and December 2000. The results of the August 1985 surveillance are documented in a report submitted to the NRC by *Reference 53*. It has been concluded that the surveillance program methodology provides an effective means of monitoring tendon forces and that the results of the surveillances confirm the structural adequacy of the containment vessel. Future surveillances will be conducted at 5-year intervals in accordance with the Tendon Surveillance Program. The 1990, 1995, and 2000 surveillance tests showed that the required tendon prestress continues to meet all design requirements. As part of the test program, a sacrificial tendon wire is extracted, examined, and tested during each surveillance. The wires extracted show no evidence of corrosion and test out to its specified yield and ultimate strengths. The grease that surrounds the tendon was analyzed using methods consistent with Regulatory Guide 1.35, Revision 2, and showed no evidence of water or unacceptable levels of chlorides, nitrates, or sulfides.

#### *3.8.1.7.3.5 Test on Rock Anchors*

In the June 1980 retensioning, 137 of the 160 tendons were stressed to at least 0.735 ultimate stress. This force had to be resisted by the rock anchors. Consequently, the tendon retensioning also constitutes a test of the rock anchor. The elongations of the wall tendon, measured at its upper anchor head, are a combination of (1) the wall tendon strains times the tendon length, plus (2) the movement, if any, of the upper anchor head of the rock anchor. The measured elongations agreed closely with those predicted based solely on the wall tendon strains. These results indicate that the rock anchors developed a force of 0.735 ultimate stress with no perceptible slippage or movement of their upper anchor head.

#### *3.8.1.7.3.6 Inservice Inspection*

The Nuclear Regulatory Commission issued an amendment to 10 CFR 50.55a, Codes and Standards, on August 8, 1996, that required the implementation of the 1992 Edition with the

1992 Addenda of ASME Section XI Code, Subsections IWE, IWL and applicable IWA requirements with limitations, modifications and supplemental requirements as described within the rulemaking. These requirements became effective on September 9, 1996 and are identified within the Containment Program and the Containment Repair and Replacement Program in the Inservice Inspection (ISI) Program document. Later Editions and Addenda of ASME Section XI Code may be used as specified within 10 CFR 50.55a that are identified within the Inservice Inspection (ISI) Program document. The second and later 10-year interval requirements are identified within the Containment Inservice Inspection (CISI) Plan.

## *3.8.2 STRUCTURAL REANALYSIS PROGRAM*

### 3.8.2.1 Design Codes, Criteria, and Load Combinations - SEP Topic III-7.B

#### <u>3.8.2.1.1 Introduction</u>

The Franklin Research Center, under contract to the NRC, compared the structural design codes and loading criteria used in the R. E. Ginna Nuclear Power Plant design against the corresponding codes and criteria currently used for licensing of new plants at the time of the Systematic Evaluation Program (*Reference 25*). The objective of the code comparison review was to identify deviations in design criteria from current criteria and to assess the effect of these deviations on margins of safety.

##### *3.8.2.1.1.1 Seismic Category I Structures*

Franklin Research Center, for purposes of the review, considered the following to be Seismic Category I structures.

**<u>Containment.</u>**

- Cylindrical wall, dome, and slab.
- Liner (no credit for structural strength under mechanical loads).
- Equipment hatch.
- Personnel locks.

**<u>Internal structures.</u>**

- Steam generator/reactor coolant pump compartments (reviewed in Generic Task A-2).
- Biological shield (reviewed in Generic Task A-2).
- Fuel transfer canal.

**<u>External structures.</u>**

a. Auxiliary building.
    - Spent fuel storage pool.
    - New fuel storage area.
    - Portions of the fuel transfer tube.
    - Seismic Category I equipment.

i. Safety injection pumps and residual heat removal pumps (in pit beneath basement floor).
ii. Refueling water storage tank (RWST).
iii. Boric acid storage tanks.
iv. Containment spray pumps.
v. Waste holdup tanks.
vi. 480-V switchgear.

b. Control building.
   - Control room.
   - Battery room.
   - Relay room.

c. Portions of the intermediate building (which house auxiliary feedwater pumps).
d. Cable tunnel.
e. Intake/discharge structure and screen house (service water (SW)) portion only.
f. Diesel-generator annex.

Major structures not classified as Seismic Category I are the turbine building and the service building.

###### *3.8.2.1.1.2 Structural Codes*

The structural codes governing design of the major Seismic Category I structures for the Ginna Nuclear Power Plant were as follows:

| Structure | Design Criteria | Current Criteria |
|---|---|---|
| **CONTAINMENT** | | |
| Concrete (including shell, dome, and slab) | ACI 318-63 | ASME B&PV Code, Section III, Division 2, 1980 (subtitled ACI 359-80) |
| | ACI 301-63 (specifications for concrete) | ACI 301-72 (Revision 1975) |
| Liner | ASME B&PV Section III, 1965 (Provisions of Article 4[a]) | ASME B&PV Code, Section III, Division 2, 1980 (Subtitled ACI 359-80) |

| Structure | Design Criteria | Current Criteria |
|---|---|---|
| | ASME B&PV Section VIII (undated), (Fabrication Practices for Welded Vessels Only) ASME B&PV Section IX (undated), (welding procedure and welders qualifications only) | |
| Personnel locks and equipment hatches | ACI 318-63 for concrete ASME B&PV Section III, 1965, for steel | ASME B&PV Code Section III, Division 2, 1980 (subtitled ACI 359-80) |
| **AUXILIARY BUILDING** | AISC-1963<br>ACI 318-63 | AISC-1980<br>ACI 349-80 |
| **CONTROL ROOM BUILDING** | AISC-1963<br>ACI 318-63 | AISC-1980<br>ACI 349-80 |
| **PORTIONS OF THE INTERMEDIATE BUILD-ING** | AISC-1963<br>ACI 318-63 | AISC-1980<br>ACI 349-80 |
| **CABLE TUNNEL** | ACI 318-63 | ACI 349-80 |
| **INTAKE/DISCHARGE STRUCTURE AND SCREEN HOUSE** | AISC-1963<br>ACI 318-63 | AISC-1980<br>ACI 349-80 |
| **DIESEL-GENERATOR ANNEX** | AISC-1963<br>ACI 318-63 | AISC-1980<br>ACI 349-80 |

a. The two significant applications for this article are (1) determination of thermal stresses in the liner and (2) analysis of pipe penetration attached to liner.

#### *3.8.2.1.1.3 Code Comparison*

The current and older (Ginna design) codes were 6/501 compared paragraph by paragraph to determine what effects the code changes could have on the load carrying capacity of individual structural members. Appendix 3F is a summary of the code comparison findings. Those code changes judged by Franklin Research Center to have the potential to significantly degrade margins of safety are listed in Tables 3.8-10 through 3.8-14. Table 3.8-15 lists the structural elements for which a potential existed for margins of safety to be less than that originally computed because of load criteria changes since plant design and construction. Rochester Gas and Electric was requested by the NRC to review all Seismic Category I structures at Ginna Station to determine if the structural elements listed in Table 3.8-15 occur in the designs, and for those that occur, to assess the actual impact of the associated code changes on margins of safety.(*Reference 26*) The results of this assessment were reported in *References 27* and *28* and are summarized in Section 3.8.2.1.2.

### 3.8.2.1.2 Assessment of Design Codes and Load Changes for Concrete Structures

The concrete structural elements identified by Franklin Research Center as being potentially affected by concrete design code changes and by any associated load or load combination changes were evaluated and the results were as follows (*References 26* and *28*).

#### *3.8.2.1.2.1 Columns With Spliced Reinforcing*

ACI 349-76, Section 7.10.3, specifies requirements for columns with spliced reinforcing which did not exist in the ACI 318-63 Code. The ACI 349-76 Code requires that splices in each face of a column, where the design load stress in the longitudinal bars varies from $f_y$ in compression to 1/2 $f_y$ in tension, be developed to provide at least twice the calculated tension in that face of the column (splices in combination with unspliced bars can provide this if applicable). This code change requires that a minimum of 1/4 of the yield capacity of the bars in each face of the column be developed by both spliced and unspliced bars in that face of the column.

To assess the impact of this change on Ginna Station, concrete outline drawings, reinforcing fabrication drawings, and available original calculations were reviewed to determine to what extent columns with spliced reinforcing exist. As a result of these reviews, a total of 57 columns with spliced reinforcing was found. They occur in the auxiliary building (14), control building (1), diesel-generator building (6), intermediate building (20), and screen house (16). All of the columns found use lap splices which occur at the bottom of the columns.

To evaluate the columns in the auxiliary building, control building, diesel-generator building, and intermediate building, they were divided into groups according to their reinforcing details and size. This grouping resulted in the formation of nine groups of similar columns. The column within each group judged to have the most severe load from the applicable loads and load combinations was chosen for evaluation. Additionally, one column from the screen house was chosen for evaluation. These columns were evaluated for compliance with ACI 349-76 provisions. The capacity of the spliced reinforcing was calculated in accordance with the code and this capacity was used with the worst-case load combination to determine if the code-required factor of safety was met. If the splices did not have the minimum required splice length to fully develop the bar in accordance with ACI 349-76, the splice capacities were reduced by a factor of $L_p/L_d$ (where $L_p$ is the splice length provided and $L_d$ is the ACI 349-76 required splice length).

The results of the evaluation found that all concrete columns evaluated meet and/or exceed the code-required factor of safety.

#### *3.8.2.1.2.2 Brackets and Corbels (Not on the Containment Shell)*

ACI 318-63 did not have any specific requirements for brackets and corbels. Provisions for these components are included in ACI 349-76, Section 11.13. These provisions apply to brackets and corbels having a shear-span-to-depth ratio of unity or less. The provisions specify minimum and maximum limits for tension and shear reinforcing, limits on shear stresses, and constraints on the member geometry and placement of reinforcing within the member.

Concrete outline drawings and available original calculations were reviewed to determine if brackets and corbels were used at Ginna. A total of 12 corbels was found during these reviews. They occur in the auxiliary building (4), intermediate building (3), and containment interior structures (5). Seven of these corbels support primary structural elements (e.g., beams, slabs). The remaining five corbels support secondary elements (e.g., a corbel on the auxiliary building exterior walls which supports a 4-in. architectural brick facing) which generally cause no significant load on the corbel.

Corbels having similar geometry and reinforcing details were grouped together, and the corbel from each group judged to have the worst load was evaluated. If this corbel was acceptable, then the others in the group were judged acceptable. The selected corbels were first evaluated for compliance with ACI 349-76 requirements for minimum and maximum reinforcing, geometry constraints, and placement of reinforcing. If all of these requirements were met, the capacity of the corbel was calculated in accordance with ACI 349-76. This capacity was used, along with the load from the worst-case load combination, to determine if the code-required factor of safety was met. If a corbel did not conform to the above requirements, then the shear stresses in the concrete imparted by the loads on the corbel were compared to the code permissible shear stress for unreinforced concrete (even though there actually was some reinforcing in the corbel). If the actual stress was less than that permitted, the corbel was judged acceptable.

The results of the evaluation of the twelve corbels were:

a. Six of the seven corbels supporting primary structural elements meet the code requirements for reinforcing, geometry, and factor of safety. The remaining corbel does not conform to the code requirements for minimum reinforcing, but the stresses in this corbel are small and the corbel was judged to have an acceptable margin of safety.

b. The five corbels which support secondary elements do not comply with the code requirements for reinforcing. However, all of these corbels have loads which produce insignificant stresses in the corbels and are therefore judged to have an acceptable margin of safety.

#### *3.8.2.1.2.3 Elements Loaded in Shear With No Diagonal Tension (Shear Friction)*

The provisions for shear friction given in ACI 349-76 did not exist in ACI 318-63. These provisions specify reinforcing and stress requirements for situations where it is inappropriate to consider shear as a measure of diagonal tension.

Concrete outline drawings and available original calculations were reviewed to determine if conditions requiring evaluation for shear friction exist at Ginna. As a result of this review, a total of 203 shear-friction conditions was found. They occur in the auxiliary building (12), containment interior structures (133), and screen house (58). These conditions exist for embedded plates supporting steel beams, concrete ledges, removable concrete slabs, beam pockets, and several miscellaneous situations.

To evaluate these conditions found in the auxiliary building and containment interior structures, they were divided into a number of groups by similarity, considering their geometry and reinforcing details. This approach resulted in the formation of 15 groups. The condition in each group judged to have the most severe load from the applicable loads and load combinations was evaluated for compliance with the code provisions. Two conditions in the screen house were also evaluated for compliance with the code provisions.

The controlling conditions were first evaluated by determining their shear friction capacity utilizing only those details strictly conforming to the code. No credit was taken for other reinforcing installed which did not meet ACI 349-76 provisions. This capacity was then compared to the controlling factored load combination to see if the code-required factor of safety was met. If the factor of safety was not satisfied, several alternative evaluation approaches were used to assess safety, and these are described below along with a summary of all results.

The results of the evaluations for this code change indicate the following for the 15 groups in the auxiliary building and containment interior structures evaluated:

a. Six groups representing 26 conditions have safety factors that are equal to or greater than the code-required factor of safety, considering only code-satisfying reinforcing.

b. Five groups representing 108 conditions have safety factors that are equal to or greater than the code-required factor of safety, considering code-satisfying reinforcing plus taking credit for any additional well-anchored reinforcing installed.

c. Two groups representing three conditions have factors of safety that are equal to or greater than the code-required factor of safety for shear stresses in unreinforced concrete. These elements had small loads and the capacities were checked ignoring any reinforcing present in the design.

d. One group representing six conditions (beam pockets for beams supporting the intermediate building floor at column line N) have an actual factor of safety less than the code-required factor of safety (considering appropriate load factors) but greater than unity against ultimate failure (with all load factors reduced to 1.0).

e. One group representing two conditions (thrust blocks at the base of each reactor coolant pump) meets the code-required factor of safety assuming an in-situ concrete strength (f'c) of 3300 psi, as opposed to the 28-day strength of 3000 psi. This in-situ strength is judged to be reasonable based upon typical concrete compressive strength increases over long time periods.

The results of the evaluation for this code change in the screen house show the safety factors are greater than those required by the code considering only code-satisfying reinforcing.

### *3.8.2.1.2.4 Structural Walls - Primary Load Carrying*

**Shear walls.**

ACI 349-76, Sections 11.15.1 through 11.15.6, specifies requirements for reinforcing and permissible shear stresses for in-plane shear loads on walls.

The ACI 318-63 Code had no specific requirements for in-plane shear on shear walls.

Concrete outline drawings and available original calculations were reviewed to determine if shear walls exist at Ginna. All walls which connect a roof or floor to a lower floor were considered to act as shear walls. As a result of the drawing and calculation review, a total of 187 shear walls was identified. They were found in the auxiliary building (87), intermediate building (1), control building (3), diesel-generator building (16), containment interior structures (59), and screen house (21).

To evaluate the shear walls in the auxiliary building, control building, intermediate building, diesel-generator building, and containment interior structures, the walls in each building were considered as a separate group. Each group of walls was further broken down by classifying each wall as either an interior or exterior wall. One wall judged to be representative of each classification within the group was then evaluated. If these representative walls were found to be acceptable, then the other walls within their classification were judged acceptable. A wall was evaluated by first determining the controlling load combination for the wall, and then determining the in-plane vertical, in-plane horizontal, and lateral loads on the wall. Using these loads, the walls were evaluated using the code provisions. Vertical and lateral loads on the walls were evaluated in addition to in-plane horizontal loads because they directly influence the requirements for reinforcing in the walls. The shear walls in the screen house were qualitatively evaluated by comparison to the auxiliary building.

The results of this evaluation are as follows:

a. The shear walls in the auxiliary building, intermediate building, control building, containment interior structures, and screen house meet the code requirements.
b. The shear walls in the diesel-generator building do not meet the current code requirements for in-plane loads or flexural bending from lateral loads. (This was reevaluated and is being upgraded as part of the Ginna Station Structural Upgrade Program.)

**Punching shear.**

ACI 349-76, Section 11.15.7, specifies permissible punching shear stresses for walls. ACI 318-63 had no specific provisions for walls for these stresses. Punching loads are caused by relatively concentrated lateral loads on the walls. These loads may be from pipe supports, equipment supports, duct supports, conduit supports, or any other component producing a lateral load on a wall.

Concrete outline drawings, available original calculations, and pipe support drawings and load sheets from the Ginna Piping Seismic Upgrade Program were reviewed to determine where punching loads occur and what the magnitude of these loads are. As a result of this review, both pipe and equipment support loads were judged to cause the most severe punching loads.

To evaluate the walls for equipment punching loads, the loads found from the above review were applied to the walls considering the specific details of each design. To evaluate the loads from pipe supports, since there are so many supports, the most severe loads found were applied to the thinnest wall found, conservatively using a 6-in.$^2$ area of application. These loads were used, along with the capacity of the wall calculated in accordance with the ACI 349-76 provisions, to determine if the code-required factor of safety was met.

As a result of the above evaluations, it was found that the walls, in all cases, meet the code-required factor of safety for punching shear.

###### *3.8.2.1.2.5 Elements Subject to Temperature Variations*

ACI 349-76, Appendix A, specifies requirements for consideration of temperature variations in concrete which were not contained in ACI 318-63. These new provisions require that the effects of the gradient temperature distribution and the difference between mean temperature distribution and base temperature during MODES 1 and 2 or accident conditions be considered. The new provisions also require that thermal stresses be evaluated considering the stiffness and rigidity of members and the degree of restraint of the structure.

Concrete outline drawings and pertinent calculations (in buildings where a possible thermal differential condition of any consequence could occur) were reviewed to determine the extent of possible thermal differential conditions in restrained concrete elements.

A total of six possible conditions/elements was found during this review. These conditions occurred in the containment interior structures (5) and in the cable tunnel (1). Based on restraint and degree of thermal differential, the cable tunnel condition was judged to be the worst case and was therefore evaluated to determine the effect on the factor of safety. The conditions for the containment interior structures are less severe because the temperature differential is less and the temperature would tend to dissipate and equalize.

The evaluation determined the moments in the cable tunnel, using the worst loading combination. The actual factor of safety was determined by dividing the theoretical moment capacity of the concrete section by the applied moments due to the loads imposed. This actual factor of safety was then compared to the ACI 349-76 required factor of safety.

The actual factor of safety for the cable tunnel was greater than the code-required factor of safety. Because the cable tunnel was considered the "worst-case" condition for the thermal differential requirement, the remaining five elements were judged to meet the current code requirements of ACI 349-76, Appendix A, for thermal loads.

#### *3.8.2.1.2.6 Areas of Containment Shell Subject to Peripheral Shear*

Concrete containment design is currently governed by the ASME Boiler and Pressure Vessel Code (B&PV Code), Section III, Division 2, 1980. The provisions for peripheral (punching) shear appear in code Section CC-3421.6. These provisions are similar to the ACI 318-63 Code provisions for slabs and footings, except that the allowable punching shear stress in CC-3421.6 includes the effect of shell membrane stresses. For membrane tension, the allowable concrete punching shear stress in the ASME code is less than that allowed by ACI 318-63.

Significant shell punching shear loads can occur at shell penetrations. To evaluate the impact of the code change, all penetrations found from a review of the containment shell concrete drawings were documented. As a result of this review, 126 penetrations, including two large access openings, were identified. Since the punching shear capacity of the shell at penetrations was expected to be closely related to penetration size, the penetrations were grouped by penetration sleeve diameter. The nominal penetration sleeve diameters range from 6 in. to 54 in. and the two large access openings are 9 ft 6 in. and 14 ft 0 in. A total of 10 groups of penetrations was defined in this manner.

All penetrations were found to be provided with a circumferential ring arrangement to allow transfer of the punching shear load directly to the concrete. The effect of the peripheral shear code change was evaluated by examining the shell capacity of the penetrations for current code adequacy. Where simple calculations or judgment showed that a penetration group is clearly adequate, the need for assessment was eliminated. For those groups that were assessed, a "worst-case" penetration from each group was chosen and the shell capacity for those penetrations was evaluated. Actual factors of safety were calculated and compared to the factor of safety required by the code. When the shell capacity for the "worst-case" penetration in a group was found adequate, the capacity of the other penetrations in the group was judged adequate.

The results of the evaluations are as follows:

a. For penetration groups with 6-in., 12.50-in., and 14.25-in. diameter sleeves, shell capacity was found adequate by calculations. For these penetrations, the code-specified punching shear capacity of the concrete exceeds the ultimate axial load of the pipe penetration. This axial load is the maximum that the process pipe is capable of developing based on its tensile strength.

b. For penetration groups with 24-in. and 54-in. diameter sleeves, the shell capacity was judged to be adequate. No significant punching shear loads were identified, and an evaluation was not considered necessary.

c. At the large access (equipment and personnel) openings (one group), significant punching shear loads occur due to containment internal pressure only. Adequacy against punching failure local to the penetration under the abnormal loading condition (90 psig internal pressure, which is 1.5 $P_a$) was demonstrated by calculations.

d. For the groups with 10-in. and 24.25-in. diameter sleeves, the shell capacity was shown adequate. The calculated punching shear loads for the "worst case" penetrations are well below the code-specified punching shear capacity of the concrete. Pipe break loads were used for the evaluation and were obtained by conservatively using a factor of 2.0 times the pipe operating pressure times the pipe area. This method is consistent with current industry practice.

e. For the 29-in. and 45.25-in. diameter sleeve groups (feedwater and main steam penetrations), the shell was found not to meet the current code-required factor of safety when using pipe rupture loads from the original plant design calculations. However, the actual factor of safety is greater than 1.0, thereby providing a margin of safety against ultimate failure.

###### *3.8.2.1.2.7 Areas of Containment Shell Subject to Torsion*

Concrete containment design is currently governed by the ASME B&PV Code, Section III, Division 2, 1980. Section CC-3421.7 of the code contains provisions for the allowable torsional shear stress in the concrete. Such provisions were not contained in the ACI 318-63 Code. The present allowable torsional shear stress includes the effects of the membrane stresses in the containment shell and is based on a criterion that limits the principal membrane tension stress in the concrete.

Only two types of penetrations, the main steam and feedwater, are provided with torsion resisting elements which rely upon the concrete capacity. In both cases, redundant elements are provided. The penetration sleeves have lugs welded to them, which could resist torsional loads and impart torsional shear stresses to the concrete. However, the final design noted in the original calculations shows that the tie rods incorporated into the penetration details were adequately designed to resist torsion. These tie rods do not rely upon the torsional shear capacity of the concrete, and, therefore, a torsional shear stress check was not required.

###### *3.8.2.1.2.8 Brackets and Corbels (On the Containment Shell)*

The ACI 318-63 Code did not specify requirements for brackets and corbels. Provisions for these components are included in the ASME B&PV Code, Section III, Division 2, Section CC-3421.8. These provisions apply to brackets and corbels having a shear-span-to-depth ratio of unity or less. The provisions specify minimum and maximum limits for tension and shear reinforcing, limits on shear stresses, and constraints on the member geometry and placement of reinforcing within the member.

Concrete outline drawings and original calculations for the containment shell were reviewed to determine if brackets and corbels were used in its design. As a result of the review, no brackets or corbels were found on the containment shell. Therefore, no further evaluation was required.

###### *3.8.2.1.2.9 Areas of Containment Shell Subject to Biaxial Tension*

Increased tensile development lengths are required for reinforcing steel bars terminated in biaxial tensile areas of reinforced-concrete containment structures in accordance with Section CC-3532.1.2 of the ASME B&PV Code, Section III, Division 2, 1980. For biaxial tension loading, bar development lengths, including both straight embedment lengths and equivalent straight lengths for standard hooks, are required to be increased by 25% over the standard development lengths required for uniaxial loading. Nominal temperature reinforcement is excluded from these special provisions. ACI 318-63 had no requirements related to this increase in development length.

Containment shell concrete outline drawings were examined to identify the areas where the main reinforcing bars are terminated with either straight development lengths or standard hooks. Special attention was paid to such areas as penetrations, where bars are likely to be terminated. The drawing review revealed nine areas where the main reinforcing bars in the wall and dome are terminated.

These cases involve vertical reinforcement in the wall and meridional bars in the dome above the ring girder. Main horizontal wall bars were found to be terminated using positive mechanical anchorage devices (such as Cadwelds and structural steel shapes) that are capable of transferring forces to other reinforcement. Typically, main horizontal and vertical bars terminated at penetrations are anchored using these positive mechanical anchorages. However, the drawing review revealed seven additional areas where supplementary bars are terminated at penetrations.

Thirteen of the 16 areas were evaluated individually by first determining the location of the critical section to be evaluated and then comparing the tensile development lengths required for the controlling load combination to the development lengths provided. The remaining three areas are similar to three of the areas evaluated, and individual evaluation was not considered warranted. In all of the 13 areas evaluated, the provided tensile development lengths exceeded ASME Code requirements. In several of the areas investigated, bars were actually terminated outside of the biaxial tensile stress area (i.e., in compressive areas which are excluded from these special requirements). As a result of this evaluation, it is concluded that the code change did not reduce the containment shell margin of safety.

#### *3.8.2.1.2.10 Steel Embedments Transmitting Loads to Concrete*

Appendix B to ACI 349-76 is a new appendix which specifies new requirements for stress analysis of steel embedments used to transmit loads from attachments into the reinforced-concrete structure. The only area of concern of this change was the integrity of the containment dome liner and studs under pressure and temperature loads that are caused by the loss-of-coolant accident and steam line break loading conditions. An evaluation of the integrity of the liner and studs was conducted by Gilbert Commonwealth for RG&E and submitted to the NRC by *Reference 29*. The conclusions were that although some failures of studs could possibly occur, these would be at the shank of the studs and thus, no tearing of the liner would occur. Details of this analysis are provided in Section 3.8.2.3.

### 3.8.2.1.3 Assessment of Design Codes and Load Changes for Steel Structures

Rochester Gas and Electric reported on the results of the evaluation of steel code and load changes by *Reference 28*. Seismic loadings for steel structures were not specifically analyzed because RG&E considered that the main structural steel elements were determined suitable by the Lawrence Livermore Laboratory Analysis documented in NUREG/CR-1821, Seismic Review of the Robert E. Ginna Nuclear Power Plant, which was approved by the NRC by *Reference 30*. The steel code changes concerning coped beams, moment connections, and steel embedments were evaluated relative to the seismic loads and load combinations in conjunction with the Structural Upgrade Program.

The evaluation of code changes and new load changes was performed for all eight major findings of the AISC 1963 versus AISC 1980 Code comparison and the one major finding of ACI 318-63 versus ACI 349-80 Code comparison. The evaluations were for loads and load combinations involving normal and operating-basis earthquake loads. Safe shutdown earthquake loads were generally addressed in NUREG/CR-1821 with exceptions noted above. Tornado loads were addressed in the Ginna Structural Upgrade Program. The results were as follows:

#### *3.8.2.1.3.1 Shear Connectors in Composite Beams*

The code change that required this evaluation involved new requirements added in the AISC 1980 Code, Subsection 1.11.4, as compared with AISC 1963 Code, Subsection 1.11.4. The code change affects the distribution, diameter, and spacing of shear connectors in composite beams.

The approach used for this evaluation was to review the calculations and the construction drawings for the use of shear connectors for composite beams.

The results of the above review showed no use of shear connectors for composite design on the plant structures reviewed, and therefore, no change to the margin of safety.

#### *3.8.2.1.3.2 Composite Beams With Steel Deck*

This evaluation is required due to the addition of a new Subsection 1.11.5 to the AISC 1980 Code. The code addition defines requirements for composite beams where a formed steel deck is used for support of the concrete slab.

The approach used for this evaluation was to review the calculations and the construction drawings for composite beams with steel decking.

The results of the review determined that the main beams and girders on the turbine building operating floor elevation 289 ft 6 in. and located between all columns, had shear connectors attached to the top flange. The concrete slab was supported by steel decking.

Selected beams were analyzed for the loads shown on the drawings. The results of the analysis showed that composite design was not required for these beams and it is surmised that the shear connectors were added to provide lateral support for the top flange. Therefore, the code change has no effect on the margin of safety.

#### *3.8.2.1.3.3 Hybrid Girders*

This evaluation was required due to the addition of a new requirement by the AISC 1980 Code to Subsection 1.10.6 which did not appear in the AISC 1963 Code. This new requirement limits the maximum stress in the flange of a hybrid girder.

The approach used for this evaluation was to review the construction drawings and specifications for the existence of hybrid girders.

The results of the review showed no use of hybrid girders on the plant structures. Therefore, this code change does not affect the margin of safety.

#### *3.8.2.1.3.4 Compression Elements*

This evaluation is based on a revision to Subsection 1.9.1 of the AISC 1963 Code by new provisions in Subsection 1.9.1.2 and Appendix C of the AISC 1980 Code.

These new provisions revise the approach for designing certain unstiffened compression elements which exceed the width to thickness ratios prescribed in the codes.

From the results of case study 10 in the Franklin Research Center report (*Reference 25*), it was concluded that only T-sections in compression need to be reviewed as the AISC 1963 Code is more conservative for other members in compression.

The approach used for this evaluation was to review the members in the structural model of the plant to determine where T-sections were used and if they were subject to compression, under the normal operating load combinations evaluated in this report.

The results of the computer output review showed none of the T-sections failing the code check for normal load combinations with the member in compression.

It was therefore concluded that for normal load combinations the margin of safety for members affected by this code change is still acceptable.

#### *3.8.2.1.3.5 Tension Members*

This evaluation was necessary because of a new requirement in the AISC 1980 Code added in Subsection 1.14.2.2.

This code addition defines the requirements for the design of axially loaded tension members where the load is transmitted by bolts or rivets through some but not all of the cross section of the member.

A generic review of the two codes was performed to compare a design example using the formulas and allowables for each. The results showed that the AISC 1963 Code provided a more conservative design.

It was therefore concluded that this code change does not decrease the margin of safety.

#### *3.8.2.1.3.6 Coped Beams*

A new requirement was added in the AISC 1980 Code requiring that beam end connections, where the top flange is coped, be checked for a tearing failure, "block shear capacity", along a plane through the fasteners.

The method used to evaluate this code change was to completely review all steel fabrication drawings for major members with bolted connections and coped top flanges. Girts, platform steel, stair stringers, and miscellaneous steel were not included as these members are lightly loaded and shear is not a concern.

The drawing review turned up 452 coped beams with 335 different erection marks. From this total a random selection of 55 beams was statistically chosen for evaluation of the code change effects.

The evaluation consisted of calculating the block shear capacity of each of the beams selected and comparing this capacity against either the loads shown on the construction drawings, the shear capacity of the connection bolts, or the reaction based on the maximum allowable load for the beam span.

In all cases the block shear capacity was higher than these other controlling reactions.

It was therefore concluded that, using a statistical approach at a 95% confidence level, no more than 5% of the population of coped beams may have capacities controlled by this code change. (Safe shutdown earthquake checks were conducted as part of the Ginna Station Structural Upgrade Program.)

##### *3.8.2.1.3.7 Moment Connections*

A new requirement was added in the AISC 1980 Code in Subsections 1.15.5.2, 1.15.5.3, and 1.15.5.4. These subsections define the requirements for column web stiffeners where moment connected members frame into columns.

The construction and fabrication drawings were thoroughly reviewed for the use of moment type connections. This survey found that only some roof beams in the screen house were designed and detailed as moment connections.

These connections were then checked against the AISC 1980 Code and it was determined that, based on the member sizes, details, and original applied loads, no column web stiffeners are required.

It was therefore concluded that for the load combination reviewed the code change does not affect the margin of safety for the structures reviewed. (Safe shutdown earthquake checks were conducted as part of the Ginna Station Structural Upgrade Program.)

##### *3.8.2.1.3.8 Lateral Bracing*

The AISC 1963 Code, Section 2.8, has been revised by AISC 1980 Code, Section 2.9. This code change revises the formulas for determining the maximum spacing for lateral supports of members designed using plastic design methods.

This code change was evaluated by a review of the existing available calculations and the original FSAR. No evidence was found of plastic design methods being used.

It was therefore concluded that this code change does not affect the margin of safety for the structures reviewed.

##### *3.8.2.1.3.9 Steel Embedments*

This code change involves the use of the ACI 349-80 Code, Appendix B, for the design of steel embedments in concrete structures. The ACI 318-63 Code used in the original design did not specifically address the design of steel embedments. It was up to the individual designer to provide an embedment which satisfied the allowable stresses in the code. Working stress design was the method used for determining loads and stresses.

The latest ACI Code requires the use of ultimate strength design which includes the use of factored loads and larger allowable stresses. This difference alone would make direct comparison of the margins of safety difficult.

There are many other differences in the methods and details that the designer would use for a given embedment and a given code, but the main difference is the requirement of ACI 349-80 Code, Appendix B, that the anchorage design be controlled by the ultimate strength of the embedment steel. Concrete strength of the anchorage must not control no matter what actual loads are applied to the anchorage. Unless the designers were fully cognizant of the requirements of ACI 349 during the actual design it is unlikely that all anchorages would satisfy this code requirement, since it allows only a ductile (steel) failure of the anchorage irrespective of the calculated or actual applied loads.

Due to these difficulties in direct comparison of the two codes it was decided to statistically select a random number of anchorages for evaluation against the ACI 349-80 Code.

From a total population of 194 columns, 51 columns were selected for evaluation. Of the 51 columns selected (*Reference 46*) had anchorage into concrete.

The approach taken for this evaluation was to analyze the column anchorage to determine if it met the ductile failure and other requirements, including minimum edge distances, embedment depth, anchor size, etc. of the ACI 349-80 Code. If the code requirements were met, it was concluded that the margin of safety for the anchorage is acceptable.

If the requirements were not met then the ultimate concrete capacity of the anchorage or the allowable steel capacity whichever was less, using the ACI 349-80 Code as the basis, was compared to the applied factored loads. Only normal design loads using current load combinations were used in the comparison. If the concrete or steel capacity, whichever controls, was still greater than the applied loads the anchorage was deemed to have an acceptable margin of safety.

The results of the evaluation for this code change are as follows:

a. Of the 46 column anchorages evaluated, a total of 22 did not meet the ACI 349-80 Code.
b. Of the 22 that did not meet the code, a total of five anchorages was unacceptable for the applied loads.

The result of this design code evaluation, using a statistical projection, is that at a 95% confidence level, no more than 21% of the population of 194 column anchorages would have unacceptable margins of safety for normal load combinations. (The issue of anchorages for normal, safe shutdown earthquake, and tornado loads was reviewed under the Ginna Station Structural Upgrade Program.)

#### 3.8.2.1.4 Summary

RG&E defined all applicable loads and load combinations considered limiting for the concrete and steel safety-related structures at Ginna Station (*Reference 27*). The NRC staff concluded that these loads and load combinations were acceptable in *Reference 31*. The evaluation of Ginna structures for design code and load changes showed that for tornado-related loadings, all required safety-related structures were either able to meet currently required factors of safety, were shown to meet margin-to-failure criteria through detailed calculations or were provided with additional reinforcement as part of the Structural Upgrade Program. For seismic loadings, it was determined that all concrete code changes were acceptable, except for the shear walls in the diesel-generator buildings, coped beams, moment connections, and steel embedments. These were further evaluated and resolved as necessary in conjunction with the Structural Upgrade Program.

### 3.8.2.2 Structural Reevaluation of Containment

#### 3.8.2.2.1 Introduction

The containment structure was reviewed as part of the SEP. The Lawrence Livermore National Laboratory performed a seismic review of Ginna Station for the NRC. This review included the containment and other structures and the results were reported in *Reference 30*. The Lawrence Livermore National Laboratory performed a further evaluation (structural review) of the capacity of the containment to withstand combined loss-of-coolant and safe-shutdown earthquake loads. The results of this evaluation were reported in *Reference 32*. For this latter evaluation, seismic loads were developed by scaling the loads developed previously in the SEP program for the 0.2g peak ground acceleration safe shutdown earthquake to 0.17g, which is consistent with the site specific ground response spectra developed by Lawrence Livermore National Laboratory (Section 2.5.2.2). Thermal and pressure loads were developed from pressure and temperature transients developed by Lawrence Livermore National Laboratory for the loss-of-coolant accident conditions.

An axisymmetric, multilayer shell of revolution analytical model was developed for the containment. The model included the concrete vertical wall and dome and the steel liner. Appropriate boundary conditions representing the shell-to-base-slab interface through neoprene pads were included. Since the base slab is founded on rock and the presence of the neoprene pads essentially isolates the base slab from the containment vessel, the base slab was not included in the model. No details, such as hatches or other penetrations, were evaluated.

New seismic, thermal, and pressure loads were developed for the Ginna containment structure as part of the SEP. New seismic loads and the adequacy of the structure to withstand the seismic loads alone were reported in *Reference 30*. New temperature and pressure time-histories were developed by Lawrence Livermore National Laboratory.(*Reference 33*) The normal operating loads, peak pressure loads, and the thermal loads corresponding to the peak pressure conditions, peak thermal loads and the pressure loads corresponding to peak thermal conditions, and seismic loads were combined. This implies that the safe shutdown earthquake occurs approximately 2 minutes after a loss-of-coolant accident. This is considered extremely unlikely and, therefore, the assumed load combination is considered very conservative.

#### 3.8.2.2.2 Containment Temperature

The normal operating temperatures assumed for the Ginna evaluation correspond to a typical "cold day." Ambient temperature inside the containment is 110°F and the outside temperature is 2°F. This condition was selected as the operating condition in that thermal gradients and thermal stresses were expected to be most severe for a cold day. The assumed operating conditions were also the initial conditions for calculating the thermal gradients through the shell. Figure 3.8-43 shows the transient time-history of the containment temperature used in the analysis. This temperature transient has been shown to be much more severe than the predicted actual postaccident temperatures that may occur (see Section 6.2.1). A maximum temperature of approximately 421° F is indicated approximately 34 sec after the start of the transient. However, the internal temperature decreases to less than 300°F at approximately 91 sec which is the time the peak pressure occurs. The rate of change of temperature compared to the resonant frequencies of the containment was such that the temperature loads could be considered as equivalent static loads.

#### 3.8.2.2.3 Containment Pressure

Containment pressure corresponding to the accident condition was developed by Lawrence Livermore National Laboratory (*Reference 33*). The time-history pressure variation within the containment is shown in Figure 3.8-44. This pressure transient is much more severe than the predicted worst-case conditions inside the containment following a loss-of-coolant accident or a steam line break (see Section 6.2.1). A maximum pressure of approximately 86 psia occurred at approximately 91 sec after the start of the transient. A 14.7 psia ambient pressure was assumed and this resulted in a pressure difference of 71.5 psig, compared with the 60 psig design pressure. The time of maximum pressure did not correspond with the time of maximum temperature. Therefore, a separate load case corresponding to the time of maximum thermal effects on the liner together with the internal pressure at that time was included.

Also, the evaluation was conducted for the conditions at 94 sec rather than 91 sec. The same peak pressure was used but a computer printout for the liner temperature which controlled the thermal stress results was available at 94 sec.

#### 3.8.2.2.4 Seismic Loads

Dynamic seismic loads acting on the Ginna containment structure were replaced by a set of equivalent static loads. The equivalent static seismic loads were computed from a previous analysis of the containment structure conducted by Lawrence Livermore National Laboratory.(*Reference 30*) In the previous Lawrence Livermore National Laboratory analysis, the containment shell was modeled as a fixed base system of lumped masses connected by weightless springs (Figure 3.8-10). Table 3.8-16 lists the values of masses and characteristics of the connecting beams for the model. A response spectrum approach was used to determine the dynamic response of the model, i.e., the first 10 modal responses of the model were combined using the square root of the sum of the squares approach. The Regulatory Guide 1.60 spectrum at 0.2g and 7% critical damping was used for the analysis reported in *Reference 32*. For the structural review, the responses were scaled to a peak ground acceleration of 0.17g in the horizontal direction and 0.11g in the vertical direction. The 0.17g acceleration level is consistent with the site specific safe shutdown earthquake for Ginna. Table 3.8-17 lists modal frequencies of the model, and Table 3.8-18 shows moment, shear, and axial loads induced in each connecting beam element scaled to 0.17g.

For the combined pressure, thermal, and seismic analysis, the containment shell was modeled as an axisymmetric shell of revolution and seismic loads acting on the shell were input in accordance with the first harmonic mode shape. The circumferential stiffness of the Ginna containment shell was much higher than its radial stiffness and, therefore, only a tangential load was applied to model the lateral seismic loads. Harmonic load amplitudes for the Ginna containment are listed in Table 3.8-19.

#### 3.8.2.2.5 Design and Analysis Procedures

##### *3.8.2.2.5.1 Containment Model*

For the SEP reevaluation of Ginna Station, several new analyses were performed to evaluate the structural acceptability of the plant for the current loading conditions that were not considered in the original Ginna design (*Reference 30*).

Even though the containment building is surrounded by the auxiliary, intermediate, and turbine buildings (Figure 3.8-45) there are no structural connections between the containment building and the other buildings. The containment building was therefore modeled and analyzed independently.

The model for the containment shell was similar to the fixed-base cantilever beam model with 12 lumped masses shown in Figure 3.8-10. Mass and section properties are uniform up to elevation 232.66 ft. The remaining shell wall and the dome are modeled by four equivalent beam elements, each with a different uniform section. The following assumptions were made in modeling the containment building and its interior structures:

a. The containment has a rigid foundation at the basement floor (elevation 235.66 ft) and has no lateral support from the surrounding soil above that elevation.
b. Since the concrete containment shell is much stiffer than the steel crane structure, the constraints from the crane structure can be neglected in modeling the containment shell.

This model, shown in Figure 3.8-10, was analyzed by the response spectrum method in the horizontal and vertical directions. The spectral curves of Regulatory Guide 1.60 were scaled to 0.2g peak acceleration for the horizontal component and 0.13g for the vertical component and input as the base excitations. Modal responses and responses to horizontal and vertical excitations were both combined by the square root of the sum of the squares method.

##### *3.8.2.2.5.2 Seismic and Loss-of-Coolant Accident Loads*

The analysis for combined seismic and loss-of-coolant accident load combination was performed by Lawrence Livermore National Laboratory (*Reference 32*). For this analysis an axisymmetric, multilayer shell of revolution analytical model was developed for the Ginna containment. The model included the concrete vertical wall and dome and included the 3/8-in. steel liner. Appropriate boundary conditions representing the shell to base slab interface through neoprene pads were included. Since the base slab is founded on rock and the presence of the neoprene pads essentially isolates the base slab from the containment, the base slab was not included in the model.

Since the scope of this evaluation was to concentrate only on the overall ability of the containment building to withstand the combined seismic and loss-of-coolant accident pressure and thermal loads, numerous details such as personnel and equipment hatches as well as piping and electrical penetrations were not included. The containment shell was assumed to be adequately reinforced around the equipment hatch and other openings so that the effects of these openings on the overall shell response were assumed to be small. Neither were any jet impingement or pipe whip forces considered during this phase of the SEP. The loss-of-coolant accident included both the primary loop loss-of-coolant accident as well as the secondary loop steam line break.

Two different computer codes were used to carry out the analysis. The computer program ANSYS (*Reference 34*) was used to determine the temperature gradient through the shell for steady-state (normal operating) temperature and the transient temperature conditions. Once the temperatures in the shell were determined, the computer program FASOR, Field Analysis of Shells of Revolution (*References 35* and *36*) was used to calculate displacements, stresses, and stress resultants under various loading conditions. FASOR employs a numerical integration method called the "field method" to solve the differential equations of a shell. A shell in FASOR may be modeled as a multilayer shell of revolution, where the thickness material properties, and temperatures for each layer are specified separately. The shape of a shell may be described as a general arc so that there is no need to divide the shell into small elements. The program defines integration points along the shell from an error tolerance specified by the user.

### *3.8.2.2.5.3 Pressure, Seismic, and Operating Temperature Loads*

For pressure, seismic loads, and operating temperature loads, the shell was modeled as two-layers, i.e., a 0.375-in.-thick layer of steel connected to a layer of concrete. The concrete thickness changes from 3 ft. 6 in. in the cylinder to 2 ft. 6 in. in the dome. These thicknesses are nominal values. The true relevant engineering values are dependent on the specific location in the structure and the loading condition that is present. Concrete and steel material properties used in the analysis are listed in Table 3.8-20. For accident temperature loads, the shell was modeled as three layers, i.e., the steel liner and two layers of concrete. The temperature gradient through each layer was assumed to be linear. The boundary condition at the base was assumed to be fixed in the tangential direction. Radial stiffness at the base was computed to be 46.9 kips/in./in. as discussed above.

It was determined from a preliminary analysis that the insulation was effective in limiting the heat flow through the cylindrical portion of the structure and maintaining the insulated liner at a significantly lower temperature than that in the uninsulated liner in the dome. This was verified by a Lawrence Livermore National Laboratory analysis where the temperature of the inside surface of liner and effective film coefficients were computed throughout the containment for the transient thermal loads. This temperature included the temperature drop through the film coefficient at the liner inside surface. In order to develop the thermal gradients through the shell, a transient thermal analysis was performed using ANSYS (*Reference 34*) with the inside liner surface temperature developed by Lawrence Livermore National Laboratory specified as a boundary condition.

It was found that the insulated part of the containment shell remained close to its steady-state condition throughout the transient time period. On the other hand, temperatures of the uninsulated liner as well as a very thin layer of the concrete containment next to the liner increased significantly as a result of internal transient air temperature. Figures 3.8-46 and 3.8-47 show the temperature gradient through the liner and adjacent concrete 94 sec and 380 sec after the start of the accident. Figure 3.8-46 corresponds to the time of peak pressure and Figure 3.8-47 corresponds to the peak liner temperature during the accident. Although this part of the concrete has only a small effect on the overall shell response, it was included as a separate layer in the analysis. The containment shell was therefore modeled as a three-layer shell consisting of the steel liner and two layers of concrete.

The temperature gradient was assumed to be linear in each of the layers. For the insulated liner, the liner temperature remained approximately at 69°F throughout the accident. The outer concrete surface temperature for both insulated and uninsulated parts of the containment was calculated to be approximately 10°F.

#### 3.8.2.2.6 Structural Acceptance Criteria

For the SEP reevaluation, the seismic capability of critical structures was evaluated using loads developed in the reanalysis. A structure was generally judged to be adequate without the need for additional evaluation for the following two cases:

A. Where loads resulting from the reanalysis were less than those used in the original design.
B. Where loads resulting from the reanalysis exceeded the original loads (or where there was insufficient information about the original seismic analysis for a comparison) but the resulting stresses were low compared to the yield stress of steel or the compressive strength of concrete.

For cases in which the seismic loads from the reanalysis were not low and exceeded the steel yield stress or the concrete compressive strength, conclusions were reached on the basis of the estimated reserve capacity (or ductility) of the structures; that is, the capability of structures to deform inelastically without failure.

#### 3.8.2.2.7 Structural Evaluation of Containment

The structural acceptability of the containment based on the SEP reevaluation is described in the following.

##### *3.8.2.2.7.1 Seismic Analysis*

There was sufficient information available for the containment building original seismic design and analysis to make a comparison to current criteria.

The original analysis was an equivalent static analysis, which was checked by a response spectrum analysis using Housner spectra. The seismic design loads were based on the equivalent static analysis. The reanalysis gave seismic loads higher than those of the original

Housner response spectrum analysis but lower than the seismic design loads from the equivalent static analysis (Figure 3.8-10). The containment building is therefore considered to be acceptable in light of current criteria if the structure meets the original design criteria.

#### *3.8.2.2.7.2 Load Combinations*

It was found that the effect of accident temperature was mainly in the uninsulated part of the dome. The meridional moment increased from 290 kips-ft/ft for the operating temperature to a peak value of 551 kips-ft/ft after 380 sec based on the very conservative accident curves used (see Sections 3.8.2.2.2 and 3.8.2.2.3). The moment in the cylinder remained at approximately 400 kips-ft/ft throughout the transient. Containment axial response to dead-weight and prestress loads were computed to be 74 kips/ft and 299 kips/ft, respectively. Since it is unlikely that peak horizontal and peak vertical seismic loads happen at the same time, they were combined using the square root of the sum of the squares method. Since the pressure load and seismic loads were acting upwards, there was very little additional margin of safety available to resist containment uplift in the case of a combined seismic event and loss-of-coolant accident. However, even if the prestress and deadweight loads were overcome over a small segment of the shell, the vertical tendons would remain intact and the liner knuckle flexibility would provide for some uplift before liner failure could be expected. The seismic response of the structure for this case was based on the assumed 7% damping as discussed in *Reference 30*. To determine the required limiting capacity of the shell, two load combinations were considered. For the load combination D + P + E loads, radial shear, moment, and hoop tension were dominated by the peak pressure load (86 psia), while tangential shear was mainly due to the seismic lateral loads. For the $D + P + E + T_a$ load combination the displacement and meridional moment in the shell were very much affected by the transient accident temperature. The peak response parameters, especially hoop tension and meridional moment in the dome, were higher than their original design values.

It should be noted that the high meridional moment in the dome was mainly due to the thermal gradient through the shell which has a self-limiting effect due to shell cracking.

In order to check the stresses in concrete and reinforcing steel in the dome, a cracked section analysis based on simple elastic bending theory was carried out. The analysis was for the temperature load which corresponded to a pressure load of 69 psia. The results showed that the maximum stress in the main reinforcing steel in the dome was 12.8 ksi which was much lower than the ASME code allowable of $0.9\ \sigma_y = 36$ ksi. Also, the peak stress in the welded wire fabric which was placed towards the outer surface of the containment shell was below the steel yield stress. Maximum concrete compressive stresses were computed to be 3700 psi which was less than the code allowable of $0.85\ f'_c = 4250$ psi.

Radial restraint to withstand the temperature and pressure loads at the base slab-containment vessel interface was provided by radial bars. The maximum tensile stress in these bars under the combined loads was approximately 54 ksi. The 130 ksi minimum yield strength of these bars provided a substantial margin of safety.

The seismic overturning moment in combination with internal pressure was resisted by the dead weight of the vessel and the rock anchors. A factor of safety of approximately 1.0

existed for separation of the cylinder and base slab assuming 7% of critical damping in the seismic response of the structure. However, the liner knuckle was found to have adequate flexibility to resist some uplift without failure.

#### 3.8.2.2.8 Structural Evaluation of Large Openings

Principal stress-resultants and stress-couples were computed and found to be co-linear or essentially so for all panels which were significant in the design check. Likewise the orientation of stress-resultants and stress-couples was found to essentially coincide with the mild steel reinforcement for all significant panels. Interaction diagrams were prepared based upon procedures for ultimate strength design of ACI 318-63.

The interaction diagrams showed that sufficient reinforcement was provided to carry all loads, including the full thermal stress-resultants and stress-couples.

#### 3.8.2.2.9 Structural Evaluation of Tension Rods

The radial loads are resisted by the radial tension rods in the outward direction, while the radial loads in the inward direction are resisted by the concrete base slab in bearing. The thermal and pressure loss-of-coolant accident loads result in radial expansion and tension in the rods. The stiffness of the liner knuckle in the radial direction is very low compared to the rods and virtually no radial loads are transmitted through the liner. The maximum tensile stress computed in the rod for the combined load case was approximately 54,000 psi. No shear stress was developed in the rods due to the clearance between the rod and sleeve in the base slab. The minimum tensile yield strength in the rods is 130,000 psi so that a factor of safety of approximately 2.6 exists for this detail.

### 3.8.2.3 Dome Liner Reevaluation

Gilbert Associates performed an analysis to evaluate the behavior of the containment dome liner and studs under the pressure and temperature loads that are caused by the loss-of-coolant accident and steam line break loading conditions (*Reference 29*).

#### 3.8.2.3.1 Dome Liner Studs

The stud scheme were used in supporting the dome liner is shown in Figure 3.8-48.

The scheme starts at the springline between the dome and cylinder, and extends to the apex. In this region the studs are 5/8-in. diameter Nelson S6L studs, and they are spaced at 2 ft-0 in. as shown in Figure 3.8-48. The S6L studs have internal threads to accept 1/2-in. diameter threaded fasteners. One-half in. diameter rods were threaded into the studs and the other end of the rod was bent around the three layers of #18 reinforcement in the dome. This was done to support the liner during concrete placement.

#### 3.8.2.3.2 Loads

##### *3.8.2.3.2.1 Loss-of-Coolant Accident*

The dome liner and studs were evaluated based on the loss-of-coolant accident pressure and temperature transients in Figures 6.2-1 and 6.2-2.

##### *3.8.2.3.2.2 Steam Line Break*

The peak air temperatures for the steam line break exceed the loss-of-coolant accident peak temperature. However, for the liner evaluation it is the peak liner temperature rather than the peak air temperature which is important. The peak liner temperatures are not very different for the loss-of-coolant accident and steam line break because even though the peak loss-of-coolant accident air temperature is less than the steam line break air temperatures, the loss-of-coolant accident temperature remains near its maximum considerably longer than the temperatures for the steam line break, thus allowing more time for the liner temperature to increase. Based on this, the temperature of the liner is not expected to be significantly different from the loss-of-coolant value of 250°F.

A liner temperature of 250°F coincident with a pressure of 57.8 psig was used for the steam line break condition in the evaluation of the dome liner and studs.

#### 3.8.2.3.3 Model Definition

##### *3.8.2.3.3.1 General Dome Model*

In the general dome area, Figure 3.8-49, the liner panels between studs are stressed equally under the pressure and temperature loads corresponding to the loss-of-coolant accident or steam line break conditions. For a liner without imperfections, all of the liner panels between the studs would reach their limiting stress capacities simultaneously. Under this condition, there would be no resultant shear force on the studs. However, if one panel is assumed to buckle prior to others, shear forces would be experienced by the adjacent studs. With the one panel buckled, the adjacent panels and studs displace towards the buckled panel. As a result of this displacement, the buckled panel displaces laterally further away from the concrete and exhibits a fall-off in its membrane stress as described in *Reference 37*. The extent of stress fall-off depends on the final displacement, Δ, of the studs on either side of the buckled panel. The difference between the fall-off stress in the buckled panel and the final stress in the adjacent panel produces a shear on the stud. The largest shear force and displacement occur for stud #1.

The liner plate material for the Ginna liner is ASTM A 442 grade 60 carbon steel, which has a minimum specified yield strength of 32 ksi. It is expected that the liner would have an actual mean yield strength of 48 ksi based on the information in *Reference 38*. In the general dome for the liner panels between the 3/4-in. diameter headed studs spaced at 4 ft-3 in., the calculated buckling stress is 5.8 ksi. For the liner panels between the 5/8-in. diameter S6L studs spaced at 2 ft-0 in., the calculated buckling stress is much less than the 32 ksi or 48 ksi yield strength, the calculated buckling stress is used as the value of limiting stress for all panels in the model adjacent to panel 1-1. The limiting compressive stresses in these panels of 26 ksi (or 5.8 ksi) combine and displace the critical stud (#1) in the direction of the buckled panel in the model for the general dome.

##### *3.8.2.3.3.2 Insulation Termination Region Model*

In the insulation termination region, the stresses in the liner behind the insulation are small relative to the large compressive stresses produced in the uninsulated portion of the liner. In the liner panel immediately outside the insulation, the largest compressive stress that is capable of being developed will produce the largest displacement of the studs. This is the limiting stress corresponding to the calculated buckling stress of 26 ksi. With the panel stressed to this value, all of the studs behind the insulation will displace as indicated in Figure 3.8-49. The stud which experiences the greatest displacement is stud #1.

### 3.8.2.3.4 Analysis

#### *3.8.2.3.4.1 Controlling Loads*

The controlling loss-of-coolant accident loads on the dome liner are a liner temperature of 250°F coincident with an internal pressure of 42 psig. For the controlling steam line break condition the liner temperature is 250°F with an internal pressure of 57.8 psig. The 250° F temperature applies in the uninsulated portion of the dome liner. Behind the insulation the liner temperature decreases as indicated in Figure 3.8-50. The liner stresses were obtained using the elastic, shell analysis computer program KSHEL1 (*Reference 39*) for the controlling loss-of-coolant accident and steam line break loads. The results of these analyses indicated that the stresses in the uninsulated portion of the liner were generally in the neighborhood of 45 ksi compression. This value exceeds the limiting stresses of 26 ksi and 5.8 ksi discussed previously. Therefore, these limiting stresses control and were used in the liner-stud interaction analyses.

As additional cases, the liner-stud interaction analyses also reviewed somewhat higher values of limiting stresses in order to determine the sensitivity of the stud displacements to variations in the stress limits. This accounts for the real possibility that some liner panels may buckle at a stress greater than their theoretical value. For this purpose, the limiting stress of 26 ksi for the 2 ft 0 in. panels was increased only 10%, resulting in 29 ksi as an additional case for the analysis of the 5/8-in. diameter S6L studs in the general dome and in the insulation termination region. Considering the usual scatter in buckling test results, it is not unreasonable to expect that there would be liner panels which could develop membrane compressive stresses 10% above the theoretical buckling value of 26 ksi. For the liner panels in the general dome where the 3/4-in. headed studs at 4 ft 3 in. spacing exist (since the 5.8 ksi stress limit was relatively low) it was practically doubled to 12 ksi. This value was used as a conservatively high stress limit.

#### *3.8.2.3.4.2 Liner-Stud Interaction*

For the general dome, the analysis was based on the method developed in *Reference 37* using the model in Figure 3.8-49. The appropriate equations in this reference were modified to include the effect of the internal pressure on the stress fall-off curve for the buckled panel 1-1. For the insulation termination region, a somewhat different liner-stud interaction analysis was performed using the model in Figure 3.8-49. The main difference is that the stress in the buckled panel (26 ksi or 29 ksi) is given and the stress fall-off concept does not apply.

In these analyses, the force-displacement curves of the embedded studs are required for the 3/4-in. diameter headed studs and for the 5/8-in. diameter S6L studs. The determination of these curves is discussed below.

**3/4-Inch Diameter Studs**

The curve used for the 3/4-in. headed studs is shown in Figure 3.8-51. This curve is based both on the test results and recommendations from *Reference 40* and from test data reported in *Reference 41*. From *Reference 40* the shape of the force-displacement relationship is provided by Equation (4) in *Reference 40* as $Q = Qu\ (1\text{-}e^{-18\,\Delta})^{2/5}$. In the equation, $\Delta$ is the stud displacement, Q is the corresponding stud force, and Qu is the ultimate stud capacity. The ultimate stud capacity was obtained from Equation (3) of *Reference 40* as 31.1 kips. Test data from *Reference 41* for 3/4-in. diameter studs in shear (Table IX) support this value for Qu.
The ultimate shear force values reported here from four stud tests all exceed 31.1 kips. Also from *Reference 41*, a displacement of 0.341 in. at failure (Table X) is reported for the 3/4-in. studs, and this value is used as the ultimate displacement in Figure 3.8-51.

**5/8-Inch Diameter Studs**

Unlike the 3/4-in. headed studs, force-displacement property data for the 5/8-in. S6L studs was not found in the Nelson literature. Therefore, the curve for these studs was constructed indirectly from tests on other types of anchors. In *Reference 42*, direct shear tests on 3/8-in. diameter and 1/2-in. diameter Nelson D2L deformed reinforcing bar anchors are reported. The embedment lengths of these bars varied over 3 in., 6 in., 12 in., and 18 in. The test results for the 18-in. long bars indicate that these bars failed in shear at or slightly above the minimum specified tensile strength of the bar material, which was 80 ksi. The results from the tests on the 18-in. long bars are believed to be applicable to the 5/8-in. S6L studs installed on the dome liner since these studs were actually extended in length by the 1/2-in. diameter threaded rods that bend around the 3 layers of #18 dome reinforcement. The studs with the rods had straight embedment distances of 9-1/2 in., 14 in., and 18-1/2 in. This configuration will adequately develop these studs to allow them to achieve their minimum specified tensile capacity in shear based on the test results for the 18-in. long straight deformed bars. The capacity for the 5/8-in. diameter S6L stud then becomes

$$Fu = As\ fs = (\pi/4)(0.437)^2(60\text{ ksi}) = 90\text{ kips}$$

where the minimum diameter of the stud (0.437 in. at the base) was used. For use in the evaluation as a lower bound study capacity, 8.3 kips was used. This represents approximately a 10% reduction of the 9.0 kips value.

Actually the 9.0 kips value itself would appear to be a conservatively low value for the S6L studs due to their lower specified tensile strength of 60 ksi compared with the corresponding value of 80 ksi for the deformed bars tested in *Reference 42*. This would be the case because the actual tensile strengths of the S6L stud material are expected to consistently exceed their 60 ksi minimum specified value by greater margins than would occur for the 80 ksi strength material for the deformed bars. An example of the increase for 60 ksi grade studs is seen in the tests on the 3/4-in. diameter headed studs discussed previously. The steel for these studs (A108) has a minimum specified tensile strength of 60 ksi, which when multiplied by the stud area (0.442 in.$^2$) gives a capacity of 26.5 kips. However, these studs consistently failed above 30 kips in the tests reported in *References 40* and *41*. Therefore, use of the value Qu = 8.3 kips in the liner-stud interaction analyses is regarded as a conservative lower bound on the expected actual capacity of the 5/8-in. diameter S6L studs. The determination of a more realistic value is discussed below.

The results and recommendations of *Reference 40* were used to establish what is regarded as an expected value for Qu for the 5/8-in. diameter S6L studs. *Reference 40* is applicable because the headed studs tested in this reference have a minimum specified tensile strength of 60 ksi which is the same as the specified tensile strength of 5/8-in. S6L stud material. Also, the embedment afforded the S6L studs on the dome by the bent 1/2-in. threaded rods is believed to be at least as effective as the head on the studs tested in *Reference 40*. Using Equation (3) from *Reference 40* gives:

$$Qu = \frac{1}{2} A_s \sqrt{f_c' E_c} = 0.5 \frac{\pi}{4} \cdot 0.437^2 \sqrt{(5)(4000)} = 10.6 kips$$

(Equation 3.8-22)

Curves corresponding to Qu = 8.3 kips (lower bound) and Qu = 10.6 kips (best estimate) are shown in Figure 3.8-52. The ultimate displacement of 0.167 in. is the limit chosen for the 5/8-in. diameter S6L studs. In the absence of any specific data on these studs, the 0.167-in. value was obtained from the tests on 1/2-in. diameter headed studs reported in *Reference 41* (Table X). The value is in reasonable agreement with the deformed bar tests from *Reference 42*. In these tests on the 1/2-in. diameter by 18-in. long deformed bars, an ultimate displacement of approximately 0.160 in. was reported.

In summary, the liner-stud interaction analyses were based on the force displacement curve for the 3/4-in. diameter headed studs shown in Figure 3.8-51. This curve is based on actual test results as reported in *References 40* and *41*. The curves for the 5/8-in. diameter S6L studs are shown in Figure 3.8-52. In the absence of specific test data on the S6L studs, lower bound and best estimate curves were constructed based on tests reported in *References 40* and *42*.

### *3.8.2.3.4.3 Effect of Internal Pressure on Liner Buckling*

The internal pressure potentially affects the liner buckling stress and stud evaluation in all three regions of the dome liner shown in Figure 3.8-48. Therefore, an evaluation of all studs was performed considering the internal pressure effect as a separate case in addition to the liner-stud interaction analyses described previously.

In order to specifically address the effect of internal pressure, it was necessary to solve the fundamental buckling problem of a straight strut, clamped at its ends, under the combined loads of a uniform temperature increase over the length of the strut plus a uniform lateral pressure. In addition the strut is continuously supported on the side opposite the pressure, which permits buckling to occur only in the direction opposed by the pressure. The resulting model is shown in Figure 3.8-53. The length of the strut, L, corresponds to the stud spacing of either 2 ft 0 in. (24 in.) or 4 ft 3 in. (51 in.). The temperature increase of the strut, $\Delta T$, corresponds to the temperature increase (above a stress free state at 70°F) which the liner experiences under a loss-of-coolant accident or steam line break condition. Likewise, the pressure on the strut, P, corresponds to the internal pressure in the containment (above atmospheric) occurring simultaneously with the liner temperature.

The buckling problem was solved using an energy method. In this approach, expressions were derived for the strain energy in the strut both before (straight) and after (deflected) buckling. In the unbuckled position the strain energy is that due only to the membrane compressive stress in the strut produced by the full restraint to ΔT. In the deflected position, both bending and membrane strain energy are present. Also in the deflected position, only lateral displacements which satisfy the equilibrium conditions on the strut are admissible. The buckling problem is solved by determining the value of temperature increase, ΔT, in the presence of the pressure, P, required to make the strain energy of the straight strut equal to the sum of (1) the strain energy of the deflected strut and (2) the work done as P displaces from the straight to the deflected position of the strut. This value of temperature is the temperature increase required to buckle the strut (the liner panel) as it is concurrently acted upon by the specific pressure.

The resulting buckling curves for the liner panels corresponding to stud spacings of 24 in. and 51 in. are shown in Figure 3.8-54. The values at P = 0 are ΔT = 27.4°F for L = 51 in. and ΔT = 123.6°F for L = 24 in., both of which produce corresponding liner stresses equal to the Euler buckling values. From the curves in Figure 3.8-54, the increase in liner temperature required to cause buckling as the pressure increases is evident. For example, an internal pressure of 10 psig (24.7 psia) increases the buckling temperature (and stress) by factors of 6.0 (L= 51 in.) and 1.7 (L = 24 in.).

Superimposed on the buckling curves are values of liner temperature and internal pressure which are based on the loss-of-coolant accident curves of Figures 6.2-2 and 6.2-1. These are discussed in Section 3.8.2.3.2.

### 3.8.2.3.5 Results and Conclusions

The results of the liner-stud interaction analyses are presented first for limiting stresses of 26 ksi and 29 ksi for the 5/8-in. diameter S6L studs spaced at 24 in. and for limiting stresses of 5.8 ksi and 12 ksi for the 3/4-in. diameter headed studs spaced at 51 in. Following this, the effect that the internal pressure has on the results are discussed.

#### *3.8.2.3.5.1 Insulation Termination Region*

The results from four separate liner-stud interaction analyses are presented in Table 3.8-21 for the studs in the insulation termination region of the dome liner. These studs are the 5/8-in. diameter S6L studs. Column (1) identifies the stud capacity, Qu, which is based on the force-displacement curve from Figure 3.8-52 used in the particular analysis. Column (2) identifies the stress in the liner just outside the insulation. The acceptance criteria for the studs is based on stud displacement, and the maximum displacement occurs for the #1 stud in Figure 3.8-49. These values are shown in column (3) and they are to be compared with the ultimate stud displacement of 0.167 in. in column (4). The percentage of the maximum displacement relative to the ultimate value is indicated in column (5). These values range from 84% to 99%. The results associated with the 10.6 kips stud capacity are more applicable than the values associated with the 8.3 kips lower bound stud capacity for the reasons discussed in Section 3.8.2.3.4.2. Therefore, the maximum stud displacement is estimated to be either 84% or 95% of its ultimate value, depending on the maximum stress which will be developed in the liner. These results are less than the 100% value indicating stud failure. However, considering the magnitude of the displacements and their sensitivity to the 10% increase in the theoretical limiting liner stress of 26 ksi, some of the studs located just outside the insulation could possibly fail.

Any stud failures which might occur would not be expected to tear the liner, based on test results reported in *Reference 43*. This reference describes tests conducted on 1/2 in., 5/8 in., and 3/4 in. diameter headed studs attached to steel flanges of various thicknesses, ranging from 0.128-in. thick to 0.389-in. thick. A total of 41 specimens were tested in all. The primary objective of the tests was to determine the mode of failure of the studs and under what conditions failure would occur by tearing of the flanges rather than in the stud itself. The main conclusion reached from the tests is that if the ratio of stud diameter to flange thickness is less than 2.7 then the studs will fail in their shank and flange tearing or pull-out will not occur. For the 5/8-in. diameter S6L studs, the diameter-to-thickness ratio is 0.437/0.375 or
1.17. This value is much less than the 2.7 limiting value; therefore, any failure of the S6L dome liner studs would not result in a tearing of the liner.

### *3.8.2.3.5.2 General Dome*

The results of the liner-stud interaction analysis for both regions of the general dome are presented in Table 3.8-22. The results in columns (1) through (5) were identified earlier. For the buckled panel in the general dome model (Figure 3.8-49), the displacements and strains are also of interest and these values are indicated in columns (6) through (9). The results for the 5/8-in. diameter S6L studs and 3/4-in. diameter headed studs are discussed separately below.

**5/8-Inch Diameter S6L Studs**

As indicated in the previous discussion of the insulation termination region, the results which are based on the stud capacity of 10.6 kips, rather than 8.3 kips, are considered to represent the best estimate for the S6L studs. The results in column (5) of Table 3.8-22 indicate a maximum stud displacement of either 68% or 102% of the ultimate value, depending on whether the limiting stress in all the unbuckled panels is 26 ksi or 29 ksi. Thus, the stud displacements are very sensitive to the stress limit developed in the adjacent panels. These results can be interpreted as follows referring to the model in Figure 3.8-49. For the studs adjacent to the buckled panel (1-1) to actually displace 102% of their ultimate value, the stress in all 19 adjacent panels would have to reach 29 ksi. This condition would occur only if there were no initial imperfections in these panels to cause them to buckle at a stress less than 29 ksi. If only one panel within the 19-panel group were to buckle at less than 29 ksi, the displacement of the
#1 stud in the model would probably be reduced to below 100%. Considering the results, it is possible that some of the S6L studs in the general dome region could fail. However, based on the test results in *Reference 43* discussed previously, any stud failures would not tear the liner in the process.

The relatively large lateral displacements in column (6) of Table 3.8-22 for the 24-in. buckled panel (1-1) deserve some attention because of the large associated strains. Due to these lateral displacements of the buckled panels, plastic hinging is calculated to occur. The strains which are produced across the liner section in the hinge region are given in columns (7), (8), and (9).

The largest membrane strain from column (7) of Table 3.8-22 for a Qu of 10.6 kips is 0.0096 in./in. compression. This value is six times the yield strain based on a 48 ksi liner yield stress. However, this strain, being compression, is not significant as far as liner integrity is concerned.

The extreme fiber strains (bending plus membrane) indicated in columns (8) and (9) of Table 3.8-22 are large by conventional measures as the results in column (10) indicate. Here, for the worst case, the extreme fiber strain is 39 times the yield strain of the liner material. To put this magnitude of strain in perspective, an extreme fiber strain equal to 39 times yield would be produced in a bend test if the liner were bent around a circular pin having a diameter of 5.6 in. The liner, being a low carbon steel, is ductile enough to be bent to this diameter without tearing. The version of the ASTM specification, A442, used for the Ginna containment liner material required that liner specimens be cold bent through 180 degrees around a pin diameter equal to the liner thickness of 0.375 in. without cracking the specimen. It is indicated in Section 3.8.1.6.5 that these tests were performed for each as-rolled liner plate supplied. This test produces an extreme fiber strain in the liner which is calculated to be 313 times the yield strain. These tests demonstrated that the liner is capable of undergoing bending strains which are much larger than those calculated for the buckled panels. Therefore, the structural integrity of the liner will not be impaired under the strain conditions calculated to exist.

**<u>3/4-Inch Diameter Headed Studs</u>**

The maximum stud displacements corresponding to limiting stresses in the unbuckled panels of 5.8 ksi and 12 ksi are shown in column (3) of Table 3.8-22. In both cases the maximum stud displacements are small, being only 11% of the ultimate value at worst. The corresponding strains in the buckled panel (1-1) due to the lateral displacement of the panel are also small; the largest value is only 1.5 times the yield strain. Thus, even though the liner is supported by a relatively large stud spacing of 51 in., which results in a low bucking capacity, the displacement of the liner does not produce strains which would impair its structural integrity.

Based on these results, it can be concluded that failure of the 3/4-in. diameter headed studs is extremely unlikely. Any stud failures that might unexpectedly occur would not tear the liner, even for studs as large as these. Recalling the conclusions from *Reference 43*, the stud diameter-to-liner thickness ratio is 0.75/0.375 or 2; and this is well within the 2.7 limit below which stud failure does not tear the liner in the process.

#### *3.8.2.3.5.3 Effect of Internal Pressure on Liner Buckling and Stud Integrity*

The buckling capacity of the liner under the combined effects of a temperature increase and coincident pressure is presented in Figure 3.8-54. The curves in the figure define the buckling capacity in the two regions of the liner where the stud spacings of 24 in. and 51 in. exist. For comparison, values of the liner temperature and internal pressure are indicated. These result from the loss-of-coolant accident conditions in Figures 6.2-1 and 6.2-2. The liner temperatures were obtained from a heat transfer analysis of the loss-of-coolant accident temperature transient in Figure 6.2-2. The time into the loss-of-coolant accident transient is indicated for several of the pressure and temperature values. For example, at 100 sec into the transient the liner temperature has increased 173°F (above 70°F) and the simultaneous pressure on the liner is 53 psig (67.7 psia).

The comparison in Figure 3.8-54 indicates that for the first 2.15 hours (7740 sec) into the transient, the internal pressure prevents the liner from buckling in all regions of the dome. During this time, the liner reaches a maximum temperature of approximately 260°F (190°F increase above 70°F), which is considerably above the temperature required to buckle it even in the region where the studs are spaced at 24 in. However, buckling does not occur because at this temperature the coincident containment pressure is 42.7 psia (28 psig). After 2.15 hours into the transient, when the internal pressure has decreased to 24.7 psia (10 psig), the results indicate that the region of the liner where the studs are spaced at 51 in. (3/4-in. headed studs) is susceptible to buckling. By that time, the liner temperature has reduced to approximately 250°F. The region of the liner where the studs are spaced at 24 in. (5/8-in. S6L studs) remains unbuckled. The effect of these results on the liner and stud evaluation is discussed below.

For the insulation termination region and the general dome region, the conclusions regarding the potential for stud failure were that failure of some of the 5/8-in. diameter S6L studs located in the insulation termination region and in the general dome region might occur, depending on whether or not the limiting stress of 26 ksi is actually exceeded. For the 5/8-in. diameter S6L studs in the general dome, this conclusion was based on an initial assumption that one panel has buckled. However, the comparison in Figure 3.8-54 indicates that the liner panels associated with these studs are not likely to buckle because of the effect of the internal pressure. The assumption that a buckled panel exists with the result that shear forces are produced in the studs is not considered to be realistic in light of these results. Therefore, stud failure is not expected to actually occur. For the remaining 5/8-in. diameter S6L studs in the region of the liner where the insulation terminates, the fact that the liner panel remains unbuckled increases the stress that is capable of developing well above the 26 ksi and 29 ksi limits used in the previous interaction analyses. The stress increases to a maximum value of approximately 47 ksi, which corresponds to the maximum liner temperature of 260 °F. The 47 ksi compressive stress exceeds the specified minimum yield strength of 32 ksi, but it is considered to be achievable since the actual average yield strength of the liner plates is expected to be in the neighborhood of 48 ksi. The effect of a 47 ksi stress occurring in the liner region outside the insulation would be to cause failure of the studs in the insulation termination region of the dome. However, based on the test results in *Reference 43* discussed previously, failure of these studs would not affect the integrity of the liner.

The remaining studs are the 3/4-in. diameter headed anchors in the region of the general dome which extends from the 55-degree meridian to the apex. The conclusions regarding the general dome area were that because of the relatively low buckling capacity of the liner in this region, the limiting stresses were small. The corresponding calculated stud displacements were considerably less than their ultimate values and stud failure was considered to be very unlikely. When the pressure effect is taken into account, it is also concluded that these studs will not fail during at least the first 2.15 hours of the loss-of-coolant accident transient because the liner panels would not buckle and, consequently, no unbalanced panel forces would exist to produce shear on the studs. Beyond this time, from Figure 3.8-54, the loss-of-coolant accident pressures and temperatures fall somewhat below the buckling curve for the 51-in. stud spacing and buckling of some liner panels could occur. If one panel buckles but adjacent panels do not, the 250°F liner temperature would produce a 45 ksi compressive stress in the unbuckled panels. This would result in an unbalanced shear force in the studs that is large enough to cause their failure. However, this condition would not affect liner integrity because the ratio of stud diameter-to-liner thickness being 2.0 is significantly less than the limiting value of 2.7 required to tear the liner. After 2.15 hours into the loss-of-coolant accident transient, the internal pressure is down to approximately 10 psig which is far below the maximum value of 60 psig that the containment structure has been designed to resist and the stresses in the reinforced-concrete structure are relatively low.

#### **3.8.2.3.6 Overall Conclusions**

Of the results and conclusions presented above, those based on a consideration of the internal pressure are considered to be more realistic since pressure would actually be present in a loss-of-coolant accident transient loading condition on the liner.

In the region of the dome where the insulation terminates, the liner is expected to remain in an unbuckled condition. As a result, unbalanced compression stresses in the liner are produced which are large enough to result in failure of the 5/8-in. diameter S6L studs located in this region based on the results of the liner-stud interaction analyses described herein. However, failure of these studs would be limited to the shank of the studs and not in the liner. Therefore, the leaktight integrity of the liner will be maintained.

Above the insulation and extending to the 55-degree meridional coordinate axis on the dome, a distance of approximately 35 ft, the liner is expected to remain in an unbuckled condition, and no unbalanced compressive stresses exist in the liner. Because of this, no shear forces are produced in the 5/8-in. diameter S6L studs in this region and, consequently, stud failure would not be expected to occur.

Above the 55-degree meridional coordinate axis and extending to the apex of the dome, the liner panels are susceptible to buckling late in the loss-of-coolant accident transient after the containment pressure has reduced to approximately 17% of the design pressure of the containment structure. In the event that a panel buckles but adjacent panels remain unbuckled, unbalanced compressive stresses are produced which are large enough to fail some of the 3/4 in. diameter studs in this region. However, failure of these studs is predicted to occur in the shank of the studs and not in the liner. In addition, the liner plate material has demonstrated the capacity to accommodate strains which are much greater than the strains which the buckled liner panels are expected to undergo. Therefore, the leaktightness of the liner will be maintained.

The NRC Staff reviewed the analyses and concluded that it is unlikely that any stud failure will result in tearing of the containment liner and, therefore, the liner will retain its leaktight integrity during the postulated loading conditions (*Reference 44*).

## ***3.8.3 CONTAINMENT INTERNAL STRUCTURES***

### **3.8.3.1 Description of the Internal Structures**

The containment interior structures include the concrete reactor vessel support, concrete floors (at elevations 245 ft, 253.25 ft, and 278.33 ft), concrete shield walls, the steel overhead crane support structures, the nuclear steam supply system, and other auxiliary equipment (see Figure 3.8-55).

The concrete internal structure is supported entirely on the base slab. No structural connections exist between the concrete internal structure and the containment shell and radial gaps permit unrestrained relative motion between the two structures. The only connection between the containment shell and its interior structures is at the top of the crane rail, where the rail top may bear on the concrete shell at four locations of neoprene pads. Figure 3.8-55 shows the overall configuration of the reactor building including the internals and major nuclear steam supply system equipment items.

### 3.8.3.2 Applicable Codes, Standards, and Specifications

The SEP reevaluation of the containment internal structures was performed using ACI 349-80.

### 3.8.3.3 Loads and Load Combinations

#### 3.8.3.3.1 Load Combinations Considered

The loads (defined in Table 3.8-23) and load combinations to be considered on a generic basis according to current requirements (ACI 349-80) are as follows:

1. $1.4\ D + 1.4\ H + 1.7\ L + 1.7\ R_o$
2. $1.4\ D + 1.4\ H + 1.7\ L + 1.7\ E_o + 1.7\ R_o$
3. $1.4\ D + 1.4\ H + 1.7\ L + 1.7\ W + 1.7\ R_o$
4. $D + H + L + T_o + R_o + E_{ss}$
5. $D + H + L + T_o + R_o + W_t$
6. $D + H + L + T_a + Ra + 1.25\ P_a$
7. $D + H + L + T_a + R_a + 1.15\ Pa + 1.0\ (Y_r + Y_j + Y_m) + 1.15\ E_o$
8. $D + H + L + T_a + R_a + 1.0\ P_a + 1.0\ (Y_r + Y_j + Ym) + 1.0 E_{ss}$
9. $1.05\ D + 1.05\ H + 1.3\ L + 1.05\ T_o + 1.3\ R_o$
10. $1.05\ D + 1.05\ H + 1.3\ L + 1.3\ E_o + 1.05\ T_o + 1.3\ R_o$
11. $1.05\ D + 1.05\ H + 1.3\ L + 1.3\ W + 1.05\ T_o + 1.3\ R_o$

Any earth pressure loads are included in live load (L).

#### 3.8.3.3.2 Applicable Load Combinations

Additional review of each of the code change elements was conducted to determine if the remaining loads, generically applicable to the structure, had any potential impact. As a result of this additional review, loads H, $T_o$, W, and $W_t$ were considered not to have any significant effect. The H loads were not considered because there is no significant hydrostatic head on the containment interior structures. The $T_o$ loads were not considered because they tend to equalize throughout the containment interior, thus resulting in no significant temperature differentials. The W and $W_t$ loads were not considered because containment interior concrete is enclosed by the containment shell, which withstands wind and tornado loads. Considering the results of both reviews, the generic load combinations are reduced to the following applicable combinations:

1. $1.4\ D + 1.7\ L + 1.7\ R_o$
2. $1.4\ D + 1.7\ L + 1.7\ E_o + 1.7\ R_o$
3. $1.4\ D + 1.7\ L + 1.7\ R_o$
4. $D + L + R_o + E_{ss}$
5. $D + L + R_o$
6. $D + L + R_a$
7. $D + L + R_a + 1.15\ E_o$
8. $D + L + R_a + E_{ss}$
9. $1.05\ D + 1.3\ L + 1.3\ R_o$
10. $1.05\ D + 1.3\ L + 1.3\ E_o + 1.3\ R_o$
11. $1.05\ D + 1.3\ L + 1.3\ R_o$

### 3.8.3.4 Design and Analysis Procedures

#### 3.8.3.4.1 Original Design

In the original design of Ginna Station reinforced-concrete structures inside the containment were modeled as simple cantilever beams with all mass lumped at the center of gravity. Analysis was by the equivalent static method as follows:

A. The fundamental period was calculated based on the assumption that the structure is a simple harmonic oscillator.
B. The response acceleration was taken from the appropriate response spectrum (Figures 3.7-1 and 3.7-2).
C. This acceleration times the total mass acting at the center of gravity gave the shear force and overturning moment at the base.
D. The shears and moments were distributed throughout the model in proportion to structural stiffness, which was based on the flexural properties of the wall systems.
E. Structural element design capacity was evaluated.

Walls and floor slabs were designed for the concentrated seismic reactions of the attached major components.

Overhead crane support structures within the containment building were reportedly evaluated for natural periods of simple harmonic motion in the two horizontal directions. Equivalent horizontal seismic forces were then obtained by applying the corresponding acceleration from the seismic response spectra to the mass of the crane. Vertical response of the crane and crane support structure was taken as the peak of the response spectra. Vertical forces were obtained by applying the peak acceleration to the mass of the crane, crane support structure, and lifted load.

#### 3.8.3.4.2 Systematic Evaluation Program Reevaluation

During the Systematic Evaluation Program seismic reevaluation (*Reference 30*) Lawrence Livermore National Laboratory developed a mathematical model that included the interior structures, the nuclear steam supply system, and the crane structure and was based on a model developed for RG&E by Gilbert Associates, Inc., in 1979 (*Reference 45*). The following assumptions were made in modeling the interior structures:

A. The model for the interior structures and crane supports included the constraint effect from the containment shell at the crane top.

B. The interior structures were assumed to have rigid diaphragms at elevations 245, 253.25, 267.25, and 278.33 ft. Masses of all concrete floors and walls were lumped to the centers of gravity of the diaphragms. Major nuclear steam supply system equipment items, including steam generators, coolant pumps, and the reactor vessel, were modeled as lumped-mass systems.

C. The crane structure was assumed to have two lumped masses located at the center of the crane structure at elevations 329.66 ft and 311 ft.

D. Based on the recommendation in NUREG/CR-0098, damping was assumed to be 7% of critical damping for the steel-and-prestressed-concrete part of the structures and 10% for the concrete part.

The interior structures model, which was prepared for the computer program STARDYNE, included plate elements for the concrete shield walls and rigid beams for the rigid floors (Figure 3.8-56). The concrete-and-steel columns were represented by elastic beam elements. The nuclear steam supply system and the neoprene pads at the crane top were included as equivalent stiffness matrices. A cantilever beam model that had seven lumped masses represented the containment shell. The total mass of each floor was lumped to the center of gravity of the floor, and rotational inertia was accounted for. Equipment masses were represented by lumped masses at the corresponding nodes. There were 99 nonzero-mass degrees of freedom in the model. Use of the Guyan reduction technique reduced the 99 to the 45 associated with the interior structure floor centers of gravity and containment shell nodes.

### 3.8.3.5 Method of Analysis

The model was analyzed by the response spectrum method in the horizontal and vertical directions. The spectral curves of Regulatory Guide 1.60 were scaled to 0.2g peak acceleration for the horizontal component and 0.13g for the vertical component and input as the base excitations. Modal responses and responses to horizontal and vertical excitations were both combined by the square root of the sum of the squares method.

A time-history method was used to generate in-structure response spectra for the interior structures. Only horizontal excitations were included in the analysis. The input base excitation was a synthetic time-history acceleration record for which the corresponding response spectra were compatible with the 0.2g Regulatory Guide 1.60 spectra. Response spectra associated with two orthogonal horizontal base excitations were generated independently at equipment locations and then combined by the square root of the sum of the squares method. Peaks of the spectra were broadened ±15% in accordance with current practice.

#### 3.8.3.6 Structural Acceptance Criteria

All Seismic Category I components, systems, and structures in the original design of Ginna Station were designed to meet the following criteria:

A. Primary steady-state stresses, when combined with the seismic stress from simultaneous 0.08g peak horizontal and vertical ground accelerations, are maintained within the allowable working stress limits accepted as good practice and, where applicable, set forth in the appropriate design standards (ASME Boiler and Pressure Vessel Code, USAS B31.1 Code for Pressure Piping, ACI 318 Building Code Requirements for Reinforced Concrete, and AISC Specifications for the Design and Erection of Structural Steel for Buildings).

B. Primary steady-state stresses, when combined with the seismic stress from simultaneous 0.2g peak horizontal and vertical ground accelerations, are limited in such a way that the safe-shutdown function of the component, system, or structure is unimpaired.

For the SEP reevaluation the structural acceptance criteria was as stated in Section 3.8.2.2.6.

#### 3.8.3.7 Structural Evaluation

Results from the reevaluation showed that the estimated seismic stresses of interior structures, including concrete shield walls, steel and concrete columns, and crane support structures, are low. No further evaluation was necessary.

### *3.8.4 OTHER SEISMIC CATEGORY I STRUCTURES*

#### 3.8.4.1 Description of the Structures

Seismic Category I structures, other than the containment and internal structures, are the following:

- Auxiliary building.
- Control building.
- Diesel-generator building.
- Intermediate building.
- Standby auxiliary feedwater building.
- Screen house (service water (SW) portion).

A complex of interconnected buildings surrounds the containment building (Figure 3.8-57). Though contiguous, these buildings are structurally independent of the containment building (Figure 3.8-45). However, several Seismic Category I structures are connected to nonseismic structures. The Seismic Category I auxiliary building is contiguous with the nonseismic service building on the west side. The Seismic Category I intermediate building adjoins the nonseismic turbine building to the north, and the auxiliary building to the south. The turbine building adjoins the Seismic Category I diesel-generator building to the north and the Seismic Category I control building to the south. The facade, a cosmetic rectangular structure that encloses the containment building, has all four sides partly or totally in common with the auxiliary and intermediate buildings.

#### 3.8.4.1.1 Auxiliary Building

The auxiliary building is a three-story rectangular structure, 70 ft 9 in. by 214 ft 5 in. It is located south of the containment and intermediate buildings and adjacent to the service building. The structure has a concrete basement floor that rests on a sandstone foundation at elevation 235 ft 8 in., and two concrete floors--an intermediate floor at elevation 253 ft and an operating floor at elevation 271 ft. The floors have a minimum thickness of 1.5 ft, and are supported by 2.5-ft thick concrete walls at the south, east, and part of the north sides of the building. The northwest corner of the building is adjacent to the circular wall of the containment building. The west concrete wall, which separates the auxiliary building and the spent fuel storage pool, is 6 ft thick.

The spent fuel storage pool is a rectangular swimming-pool-type concrete structure. Its bottom is at elevation 236 ft 8 in. Walls are 6-ft thick at the north and west sides and 3-ft thick at the east and south sides, which are below the ground surface and also serve as retaining walls.

The auxiliary building has two roofs constructed of steel truss and bracing systems and supported by frame bracing systems. The high roof (elevation 328 ft) covers the west part of the operating floor and the spent fuel storage pool. The low roof (elevation 312 ft) covers the east part of the operating floor. Insulated siding is used for the wall above the operating floor.

A platform that supports a component cooling surge tank and a heat exchanger rises from the operating floor to elevation 281.5 ft. The platform is supported by columns and bracings. There are also a number of 2.5-ft to 3.5-ft thick concrete shield walls on the floors.

The bottom elevation of the foundation mat is 233 ft 8 in., with the deepest foundation for the decay heat removal area at elevation 217 ft 0 in. with a sump at elevation 214 ft 0 in. Rock elevation in this area is at approximately elevation 236 ft 0 in. The west end of the superstructure of the auxiliary building is connected with a portion of the service building and on the northwest with the intermediate building. However, the foundation of the auxiliary building is independent of these building foundations.

#### 3.8.4.1.2 Control Building

The control building is located adjacent to the south side of the turbine building and is a 41-ft 11-3/4 in. by 54-ft 1-3/4-in. three-story structure with concrete foundation mat at elevation 253 ft. The foundation of the control building is supported on lean concrete or compacted backfill. The rock elevation in this area is at approximately elevation 240 ft 0 in. The foundation of the control building was excavated to the surface of the bedrock. The fill material was placed on the rock surface to a depth coincident with the control building foundation.

The bottom elevation of the deepest portion of the foundation mat is at elevation 245 ft 4 in., with a structural slab supported at elevation 250 ft 6 in. with a thickened slab for column footings. The common wall is reinforced with structural members, stiffeners, and siding to form a pressurization wall or "superwall." The portion of the common wall above elevation 289 ft 6 in. to the roof has 1/4 in. armor plate. The south and west sides have reinforced-concrete walls, and the roof is also reinforced concrete. The control room floor at elevation 289.75 ft and the relay room floor at elevation 271 ft are 6-in. thick reinforced-concrete slabs supported by steel girders that are tied to turbine building floors at the respective elevations. The basement is the battery room. The east wall of the control room, from elevation 289 ft 6 in. to the roof, has 1/4 in. armor plate covered by insulated siding. The relay room east wall is primarily insulated siding and some concrete block. The east wall has been modified during the Structural Upgrade Program to withstand the effects of tornado wind, tornado differential pressure, tornado missiles, and flooding of Deer Creek. The modification consists of a reinforced-concrete Seismic Category I structure adjoining the east wall of the relay room (see Section 3.3.3.3.6). The battery room is below grade.

#### 3.8.4.1.3 Diesel Generator Building

The diesel generator building is a one-story reinforced-concrete structure that has two cable vaults underneath the floor. The south wall, which is common with the turbine building, is reinforced to be a pressurization wall like the one described above in Section 3.8.4.1.2. The building roof has a built-up roof supported by four shear walls that sit on concrete spread footings.

The diesel generator building was modified as part of the Structural Upgrade Program to withstand tornado winds and missiles, external flooding, seismic loads, and extreme snow loads. A new reinforced-concrete north wall was constructed 4 ft north of the existing north wall. Reinforced-concrete wing walls were constructed that extended the east and west walls to meet the new north wall, enclosing the space between the existing and new north wall. The new wall includes missile-resistant watertight equipment and personnel doors. A new reinforced-concrete slab roof with a reinforced-concrete parapet was constructed covering the entire diesel generator building. The existing north wall and portions of the existing roof were left in place. The building as modified was designed to remain undamaged during and after an operating basis earthquake and remain functional during and after a safe shutdown earthquake.

#### 3.8.4.1.4 Intermediate Building

The intermediate building is located on the north and west sides of the containment building, and is founded on rock. The west end has a retaining wall where the floor at elevation 253 ft 6 in. is supported. The bottom of the retaining wall footing is at elevation 233 ft 6 in. Rock elevation in this area is at approximately elevation 239 ft 0 in. Foundations for interior columns are on individual column footings and embedded a minimum of 2 ft in solid rock. The building, which also encloses the cylindrical containment building, is north of the auxiliary building and is connected to the part of the auxiliary building that is under the high roof.

The building is a 136-ft 7-in. by 140-ft 11-in. steel frame structure with facade structures on each side. The facade structures are steel frame bracing systems covered with shadowall aluminum sidings. The concrete basement floor slab (elevation 253.5 ft) is supported by a set of 2-ft 10-in. square concrete columns and a concrete retaining wall on the west side. The columns have individual concrete footings embedded in the rock foundation. The top elevations of the footings vary from 238 ft to 236.5 ft.

In the north part of the building, there are three floors at elevations 278.33 ft, 298.33 ft, and 315.33 ft, and a high roof at elevation 335.5 ft. In the south part of the building there are two floors at elevations 271 ft and 293 ft, and the low roof at elevation 318 ft. All floors are made of composite steel girders and 5-in. thick concrete slabs. Built around the circular containment building, the floors extend completely through the west side of the intermediate building, a major portion of the north side and a small portion of the south side. There are no floors on the east side. The roofs are supported by steel roof girders. The floors and roofs are also supported vertically on a set of interior steel columns which are continuous from the basement floor to the roof. Concrete block walls surround all the floor space between the basement floor and the roofs.

The top of the four facade structures is at elevation 387 ft. There is no roof at the top, only a horizontal truss connecting the four sides to provide out-of-plane strength. One special characteristic of the west facade is that the horizontal floor or roof girders are connected not to the bracing joints but somewhere between joints. In such a design, the columns must transform significant shears and moments when the structure is subject to lateral loads.

#### 3.8.4.1.5 Standby Auxiliary Feedwater Building

The standby auxiliary feedwater building is a reinforced-concrete Seismic Category I structure with reinforced-concrete walls, roof, and base mat. The building is supported by 12 caissons which are socketed into competent rock.

The building was analyzed to obtain the seismic response to three simultaneous, independent, mutually perpendicular acceleration time-histories which enveloped the response spectrum of Regulatory Guide 1.60. The analysis considered soil/caisson interaction and soil liquefaction potentials. Equivalent seismic forces obtained from the analysis were distributed through the reinforced-concrete structure in proportion to the stiffness of the structural elements.

#### 3.8.4.1.6 Screen House

The screen house-service water (SW) building is comprised of two superstructures, one for the service water (SW) system and one for the circulating water system (the screen house portion). The service water (SW) portion of the building (both below and above grade) is a Seismic Category I structure.

The service water (SW) portion houses four Seismic Category I service water (SW) pumps and Seismic Category I electric switchgear. The screen house portion houses the traveling water screens and circulating water pumps.

The entire screen house-service water (SW) building is founded in or on bedrock with the exception of the basement of the electric switchgear portion which is founded approximately 4 ft above bedrock. Since the building is founded in bedrock the basement will not realize any spectral acceleration and the seismic loading is equivalent to the ground motion of 0.08g and 0.20g.

The basement is designed to be dewatered. The full height of the wall is designed for an external hydrostatic pressure plus a seismic load equal to a percentage of the dead load of the wall and the hydrostatic pressure. For the portion of the wall below grade and above bedrock an active earth pressure based on a saturated soil weight is applied.

Internal walls, such as pump baffles and the wing walls between the traveling screens, were designed for a full height hydrostatic pressure on either side plus a seismic load due to the water movement during a seismic event.

The service water (SW) portion of the screen house consists of four rigid frame bents in the east-west direction with bracing for wind and seismic loads in the north-south direction. The roof system is designed as a horizontal truss to transmit horizontal seismic loads to the frame columns and through the bracing to the foundation.

#### 3.8.4.1.7 Turbine Building

Even though the turbine building was not designed to be Seismic Category I, it is included in this section because of its connection to Seismic Category I structures.

The turbine building is a 257.5-ft by 124.5-ft rectangular building on the north side of the building complex. It has a concrete basement at elevation 253.5 ft, two concrete floors (a mezzanine floor at elevation 271 ft and an operating floor at elevation 289.5 ft). The roof includes a roof truss structure from elevation 342.66 ft to elevation 357 ft composed of top and bottom chords connected by vertical bracing. The roof and floors are supported by steel framing and bracing systems on all four sides of the building. The floors are also supported by additional interior framing at various locations under the floors.

Part of the south wall frame also serves as the north wall of the intermediate building. The north facade structure (from elevation 357 ft to elevation 387 ft) is actually on the top of the south frame of the turbine building. The west frame is the continuation of the west facade structure of the intermediate building. This west frame is also part of the service building. Except between buildings, the walls of the turbine building have insulated aluminum siding.

Inside the building and parallel to the south and north frames, there is an interior frame system supporting the crane from the basement elevation to elevation 330 ft. The crane frame is designed like the exterior frame system with vertical columns, horizontal beams, and cross bracing bolted to columns. Each interior column is welded to the corresponding exterior column at the joints and mid-points of columns by a series of girder connections.

The south frame of the turbine building is designed like the west facade structure of the intermediate building; that is, horizontal floor girders are connected to columns somewhere between joints.

#### 3.8.4.1.8 Service Building

The service building is a nonseismic structure. It is included in this section because it is contiguous with Seismic Category I structures.

The service building is located on the west side of the building complex. It extends from the south end of the auxiliary building, through the intermediate building, and ends a little before the north end of the turbine building. The building is a two-story steel structure with spread footings, steel columns, and concrete-steel framing floors and roof. The basement is at elevation 253.66 ft, the floor is at elevation 271 ft, and the roof is at elevation 287.33 ft. The walls between the service building and the other buildings as well as the partitions in the building are made of concrete blocks.

#### 3.8.4.1.9 Interconnected Building Complex

The auxiliary, intermediate, control, screen house, standby auxiliary feedwater, and diesel generator buildings are Seismic Category I structures, and the turbine and service buildings are nonseismic category structures (see Figure 3.8-57). In the original analysis, each Seismic Category I structure was treated independently. For the SEP reevaluation it was found that the interconnected nature of the buildings was an important feature, especially in view of the lack of detailed original seismic design information. Therefore, both Seismic Category I and nonseismic category buildings were included in the reanalysis model.

The auxiliary, intermediate, turbine, control, diesel-generator, and service buildings form an interconnected U-shaped building complex (Figure 3.8-58) that is mainly a steel frame structural system supported by concrete foundations or concrete basement structures. A typical steel frame is made of vertical continuous steel columns with horizontal beams and cross bracing. The connections are typically bolted. The braced frames serve as the major lateral load-resisting system. Several such steel frames connect various parts of different buildings, which make the building complex a complicated three-dimensional structural system.

#### 3.8.4.1.10 Canister Preparation Building (CPB)

The CPB superstructure is designed to meet the applicable requirements of 10 CFR 50, the Ginna UFSAR, and the Building Code of New York State. The CPB superstructure is a seismic II/I structure such that the building cannot adversely impact the transfer cask, Auxiliary Building, and DSC when fuel is present.

The CPB superstructure is designed to transmit full tornado loading without differential pressure to primary members. The secondary members, purlins and girts are allowed to deform and yield locally such that there is no loss of function. Under direct tornado wind loads there may be local building cladding failures by this design and therefore tornado differential pressure is not a design load condition due to venting made available as a result of these localized failures.

The CPB Building Overhead Crane shall be supported to Seismic II/I criteria without consideration for live loads.

The CPB finished floor elevation is 269' - 2" to permit movement of the transporter into the facility from existing grade elevations and allow the 125-Ton crane to lift the transfer cask clear of the transporter trunnion supports.

### 3.8.4.2 Applicable Codes, Standards, and Specifications

The structural codes governing the original design of major Seismic Category I structures for Ginna Station and the corresponding currently applicable codes are listed in Section 3.8.2.1.

The impact of the code changes was evaluated in *Reference 25* (see Section 3.8.2.1). Several elements and regions were identified in the Seismic Category I structures that needed reevaluation. Additional analyses were performed (*Reference 30*) to determine the acceptability of the structures. The summary of these results is presented in Section 3.8.2.1.2.

### 3.8.4.3 Loads and Load Combinations

The loads and load combinations used in the original design of Ginna Station, the currently applicable loads and load combinations, and a comparative evaluation of these two sets were studied by the Franklin Research Center (*Reference 25*). The loads and load combinations that were not considered in the original design but had a potential effect on the structural acceptability were identified and additional analyses were performed to evaluate these changes and the results were reported in *References 27* and *29* (see also Section 3.8.2.1.2).

### 3.8.4.4 Design and Analysis Procedures

#### 3.8.4.4.1 Original Design and Analysis Procedures

A brief description of the dynamic analysis performed for the original design of Ginna Station is in the following.

**Auxiliary Building**

The steel superstructure above elevation 271 ft of the auxiliary building was evaluated for equivalent horizontal seismic loads based upon either the maximum spectral response or the spectrum value corresponding to the first harmonic frequency of the structure. This superstructure was designed (*Reference 46*) originally to withstand a wind loading of 18 $lb/ft^2$.

**Control Building**

The original seismic design of the control building was based on the operating-basis earthquake as follows:

Structural steel columns were designed for flexural moments resulting from a horizontal load equivalent to 10% of the axial load applied at the mid-span of the column.

Concrete walls above grade were subjected to a horizontal reaction normal to the wall and applied at mid-span. The wall was treated as a fixed-base cantilevered beam. The equivalent seismic load was 10% of the wall weight.

**Intermediate Building**

The bracing system of the intermediate building is common to the turbine, service, and auxiliary buildings and the facade structure. The bracing was checked to demonstrate that it could resist equivalent seismic load components from the above structures.

**Diesel-Generator Building**

The diesel-generator building has concrete shear walls and steel-framed roof structures. The seismic design of the concrete shear walls considered both in-plane and normal equivalent static loads. Seismic accelerations were taken as the peak of the seismic response spectra for 5% of critical damping. The steel roof framing was designed for a horizontal equivalent safe shutdown earthquake seismic load, taken as the mass of the roof structure and superimposed loads times the peak seismic response for 2.5% damping. Column foundations were designed for an additional 20% of axial load to account for seismic effects.

**Turbine Building and Service Building**

The turbine and service buildings are nonseismic structures that are connected to Seismic Category I structures. For purposes of the original seismic design, coupling between the two classes of structures was not considered.

## 3.8.4.4.2 SEP Reevaluation Design and Analysis Procedures

The seismic design input for the SEP reevaluation of the Seismic Category I structures are described in Section 3.7. The seismic analyses of these structures performed by Lawrence Livermore National Laboratory for SEP reevaluation were as follows:

### *3.8.4.4.2.1 Mathematical Model*

In the original analysis, each Seismic Category I structure was treated independently. Because of the interconnected nature of the buildings the SEP reevaluation included the entire building complex in the reanalysis model.

The auxiliary, intermediate, turbine, control, diesel-generator, and service buildings form an interconnected U-shaped building complex (Figure 3.8-58) that is mainly a steel frame structural system supported by concrete foundations or concrete basement structures. A typical steel frame is made of vertical continuous steel columns with horizontal beams and cross bracing. The connections are typically bolted. The braced frames serve as the major lateral load-resisting system. Several such steel frames connect various parts of different buildings, which makes the building complex a complicated three-dimensional structural system. The compositions and interrelationships of the buildings in the complex are described in Appendix C to *Reference 30*.

The principal lateral force-resisting systems of the interconnected building complex are the braced frames. Several such systems tie all buildings together to act as one three-dimensional structural system. It was, therefore, necessary to model these buildings in a single three-dimensional model to properly simulate interaction effects. The model was developed based on the following assumptions.

**Rigid foundation.**

All buildings are founded on solid sandstone rock or on lean concrete or compacted backfill over rock and are assumed to have rigid foundations; thus, no soil-structure interaction effects are considered.

**Uncoupled horizontal and vertical responses**

There is no coupling between horizontal and vertical responses (i.e., only horizontal responses result from horizontal loadings and only vertical responses from vertical loadings). This is a reasonable assumption for this type of medium-height building that has regular frames and doors.

**Only horizontal ground motion in the dynamic analysis.**

For the dynamic analysis, the mathematical model was designed to have only horizontal responses because the major concern is the capacity of the lateral force-resisting system. Vertical response was calculated assuming no dynamic amplification. Because the structures were originally designed for vertical loads, such as dead and live loads, they are relatively stiff in the vertical direction and in most cases, are not considered to have significant dynamic amplification during vertical excitation. It is not necessary to simulate both vertical and horizontal behavior simultaneously.

**Rigid floors and roofs.**

All floors and roofs were assumed to be rigid in-plane because of the high stiffness for horizontal loads of the in-plane steel girders and concrete slabs. Each floor or roof has three degrees of freedom: two in horizontal translation and one in vertical (torsional) rotation. All points on a floor or roof were assumed to move as a rigid body. The center of gravity of each rigid floor or roof was selected as the representative node.

**Lumped masses.**

All structural and equipment masses were assumed to be lumped at the floor or roof elevations, then transformed to the centers of gravity of each rigid floor or roof.

**Hinge connections.**

Most bolted joints that connect bracing and beams to columns (and columns to base supports) were treated as pin or hinge connections based on reviews of pertinent drawings. The few exceptions are described in the discussion of the model for each building.

**Buckled and unbuckled bracing systems.**

Cross-bracing members, which are the primary elements of the lateral load-resisting system, are expected to buckle during compression cycles because of their large slenderness ratios. After a member buckles, it has zero or very small stiffness, but regains its capacity under tension. Such nonlinear behavior was approximately accounted for by considering two linear models: a half-area model that simulates buckled bracing and a full-area model that simulates unbuckled bracing.

In the half-area model, it was assumed that both cross-bracing members have only half the actual member cross-sectional area and can take both compression and tension during earthquake excitation. The full-area model was based on the assumption that bracings with the full cross-sectional area are effective in both compression and tension.

**Stick model for concrete wall structures.**

The control building, which has concrete walls and roof that are much stiffer than the other structures, was modeled as an equivalent beam. The two-story concrete substructure in the basement of the auxiliary building was treated similarly.

**Stiffness and mass effects of the diesel-generator and service buildings.**

The one-story diesel-generator building has four shear walls that have significant stiffness but minimal mass (only the roof mass needs to be considered; the other masses are on the rigid foundation). Therefore, the four shear walls were modeled as four elastic springs having the equivalent stiffness of the shear walls. The service building is a relatively flexible steel frame structure, and only its mass was included.

**Damping.**

A uniform damping of 10% of critical was assumed for the whole structural system based on the suggestion of NUREG/CR-0098 for bolt-connected steel structures under safe-shutdown earthquake loading.

The three-dimensional mathematical model for the building complex was prepared for the computer program SAP4 (*Reference 47*). All steel frames were modeled by beam elements. The model rigid diaphragms for all roofs and floors were represented by the rigid restraint option of SAP4. There are 17 such rigid diaphragms in the model that were treated this way.

The two-story concrete substructure of the auxiliary building and the control building were modeled by equivalent beams. The four shear walls of the diesel-generator building were represented by four elastic springs attached to the north frame of the turbine building at the diesel-generator building roof. The masses of the service building roof were lumped to the turbine and intermediate buildings. All other masses were lumped to the centers of gravity of floor or roofs.

The complete model had 686 nodal points, 44 dynamic degrees of freedom, 1213 beam elements, and 10 elastic springs.

### *3.8.4.4.2.2 Method of Analysis*

Figure 3.8-59 is a flow chart of the analytical procedure. The frequencies and mode shapes of the structural system were obtained by the subspace iteration method provided in SAP IV.

After the frequencies and mode shapes were obtained, the structural responses were computed by the response spectrum method. The seismic input was defined by the horizontal spectral curve of the safe shutdown earthquake specified in Regulatory Guide 1.60 for 10% structural damping and 0.2g peak ground acceleration.

Two structural models were analyzed, one with half the bracing area (half-area model) and one with the full bracing area (full-area model). For each model, two analyses were performed, one with the input excitation in the north-south direction, the other in the east-west direction. In each analysis and for each direction the modal responses were combined by the square root of the sum of the squares method. Responses to the north-south and east-west excitations were also combined by the square root of the sum of the squares method. Vertical responses were obtained by taking 13% (0.2g x 2/3) of the dead load responses.

### *3.8.4.4.2.3 Structural Evaluation*

**Auxiliary Building**

Based on the stresses calculated in the reanalysis, the concrete structure has adequate load margins to withstand seismic loads. However, the braced steel frames of the superstructure are more critical. The bracings in the east-west direction have stresses below yield, but the north-south bracings are near or exceed yield. The bracing at the northeast corner of the low roof has a safety factor (defined as $f_y/f$) of about 0.8. Alone this may be considered marginal, but this bracing is one of only two lateral load-resisting systems for the auxiliary building superstructure in the north-south direction. The other one is the bracing between the high and low roofs, and its stress is close to yield. Consequently, RG&E upgraded this bracing on the auxiliary building east wall as part of the Structural Upgrade Program.

**Intermediate Building and Facade Structures**

The braced frames in the low portion of the east and west facades are the relatively weak areas of the intermediate building and facade structures. The stresses in the cross bracings are at or a little over yield (safety factor of 0.9). The lateral load-resisting systems have more reserve capacity than do the braced steel frames of the auxiliary building discussed above. The vertical columns of the floors and nonstructural members, such as stairway structures between floors and sidings, provide additional lateral support to the structure.

The reanalysis indicated that the columns supporting intermediate floors may yield locally at locations where floors at different elevations meet at mid-points between joints. However, those columns still have sufficient moment-resisting capacity, and the column systems can be considered acceptable.

**Turbine Building**

The Lawrence Livermore National Laboratory evaluation concluded that the lateral load-resisting system for turbine building floors had stresses below yield. The cross-bracings above the operating floor in the south, north, and west walls had stresses that exceed yield. The bracings right above the control building superwall had the lowest safety factor (0.7). These bracings sustain high loads because of the relatively high stiffness of the superwall and the control building compared to the turbine building frames. Consequently, RG&E upgraded this bracing on the turbine building south wall as part of the Structural Upgrade Program.

**Control Building**

Excluding stress concentration effects, the maximum shear stress in the reinforced-concrete walls of the control building is approximately 200 psi.

Because the walls have No. 5 reinforcing steel bars (5/8-in. diameter) at 12-in. spacing (in both horizontal and vertical directions), the structure is considered to be adequate for resisting shear.

### 3.8.4.5 Masonry Walls

As a result of IE Bulletin 80-11, Masonry Wall Design, RG&E identified the masonry walls at Ginna Station that were considered to be safety-related. Through a series of analyses a number of masonry walls were determined to be able to withstand all applicable loads and load combinations. Other masonry walls were qualified based on providing restraining modifications or safety-related equipment protection.

#### 3.8.4.5.1 Applicable Walls

The masonry walls in the structures considered under this section were surveyed to determine if their failure could damage any safety-related systems, equipment, and attachments.

Figures 3.8-60 through 3.8-62 illustrate the location of the 37 masonry walls that are considered safety-related, i.e., whose potential failure must not endanger safe shutdown capability. The presence of a safety-related system or component within one wall height of these walls is sufficient to qualify the wall as safety-related. The 37 walls contain 56 panels, a panel being a wall division isolated for engineering analysis.

Twelve of the 37 safety-related walls are reinforced vertically. Of this total, seven are reinforced with one #3 bar on 32-in. centers. The remaining five are reinforced with two #3 bars on 16-in. centers. The joint reinforcement is DUR-O-WALL standard truss type on 8-in. centers or DUR-O-WALL "extra heavy" truss type on 16-in. centers. All except one safety-related masonry block walls are running bond masonry walls. One of the walls is composed of interlocking lead bricks.

#### 3.8.4.5.2 Loads and Load Combinations

The walls were reevaluated for the following loads and load combinations.

**Loads**

- Wind load.
- Seismic accelerations.
- Dead loads.
- Ambient temperature differentials.

**Load Combinations**

- D + (1.5 P + 1.0 T)[a]
- D + (1.25 P + 1.0 <u>T</u>)[a] + 1.25 W

---

a. Accident pressure and temperature loads will be considered only inside containment when wall configurations make differentials a possibility. No safety-related masonry walls satisfy this condition.

- $D + (1.0\ P + 1.0\ T)^a + 1.0\ E'$

Symbols used in the above equations are defined as follows:

D = Dead load of structure (a value of D ± 0.05 shall be used where it produces maximum stress)

P = Accident pressure load

T = Thermal loads based upon temperature transient associated with 1.5 times accident pressure

T′ = Thermal loads based upon temperature transient associated with 1.25 times accident pressure

$\underline{T}$ = Thermal loads based upon temperature transient associated with accident pressure

E′ = Safe shutdown earthquake load

W = Wind load

**Wind Loads**

| **Height Above Ground (ft)** | **Pressure Load (psf)** |
|---|---|
| 0-15 | 12 |
| 6-25 | 15 |
| 26-40 | 18 |
| 41-60 | 21 |
| 61-100 | 24 |
| 101-200 | 28 |

### 3.8.4.5.3 Stress Analysis

#### *3.8.4.5.3.1 Computer Program*

The computer program SAP 4 was used to calculate stresses. Wall geometry, boundary support conditions, material and physical properties, attachment loads, and response spectra information were input into the SAP 4 program. The program then performed static and dynamic analyses to determine stresses in the walls for the various load combinations.

The stresses determined by the SAP 4 program were then compared to allowable stresses using a special purpose post-processor program designed to combine stresses obtained from the static and dynamic analysis of the SAP 4 program and compare the resultant stresses against allowable values.

The analysis uses linear working stress principles. The uncracked moment of inertia is based on the unreinforced section. The cracked moment of inertia is calculated by equating the moment of the transformed tensile steel area about the centroid axis of the cross-section to the moment of the masonry compressive area. Section stiffness is calculated using Branson's equation.

Boundary conditions used in the analysis are applied to each wall so as to reasonably resemble the actual physical conditions.

#### *3.8.4.5.3.2 Seismic Analysis*

Seismic analysis of the Safety-related masonry walls was performed for the following three levels.

**Level 1 Safe Shutdown Earthquake (0.2g SSE)**

With Appendix A to Standard Review Plan 3.8.4 acceptance criteria.

**Level 2 Safe Shutdown Earthquake (0.17g SSE)**

(Site-specific SEP earthquake)

With Appendix A to SRP 3.8.4 acceptance criteria.

**Level 3**

Level 2 analysis with the exception that a 1.5 overstress factor for tension normal to the bed joint is used instead of the SRP value of 1.3 as acceptance criteria.

Seismic analysis was performed using the response spectrum method. Response spectra for the analyses were based on averaging the floor response spectra for the top and bottom elevations if the wall is supported at both locations. Otherwise, the floor response spectrum at the base of the wall is used. Response spectra were broadened by 15% to account for uncertainties in the analytical model compared with the physical structure. The assumed damping value of 7% is consistent with Appendix A to SRP 3.8.4.

The analysis takes into account the combined effects of all modes of vibration up to 33 Hz, which corresponds to the rigid range of the floor response spectra. For walls whose frequencies are greater than 33 Hz, the floor response accelerations at 33 Hz were used for the analysis.

Three directions of earthquake were considered in the analysis by evaluating walls for both vertical plus out-of-plane and vertical plus in-plane load combinations. The vertical plus out-of-plane load combination was found to be the limiting load case in the analysis.

#### 3.8.4.5.4 Interstory Drift

In-plane strain criteria used to verify the adequacy of the walls is discussed in "Recommended Guidelines for the Reassessment of Safety-Related Concrete Masonry Walls," prepared by the Owners and Engineering Informal Group on Concrete Masonry Walls, October 6, 1980. The acceptance criteria are based upon an uncoupled system (separate treatment of in-plane and out-of-plane loads). Evaluations indicate that the in-plane strains induced on the walls due to interstory drift are less than the allowables permitted in the majority of instances, regardless of whether a mechanism exists to induce the drift into the walls. In the remaining instances, the implied strains would exceed the acceptance criteria if a positive transfer mechanism existed. For these later instances, a specific case-by-case review was conducted of the wall configuration with respect to the surrounding structure, displacements, and drift inducement mechanics. From this review, it was judged that a sufficient mechanism does not exist to induce significant interstory in-plane drift. Masonry walls at Ginna are not relied upon to provide horizontal shear load resistance (i.e., shear walls). Out-of-plane interstory drift has no significant effect on the walls since they can be considered simply supported between stories.

#### 3.8.4.5.5 Multi-Wythe Walls

There are no safety-related multi-wythe or brick masonry walls.

#### 3.8.4.5.6 Block Pullout

The attachments to the walls are typically made with 3/8-in. drilled anchors. Calculations of the forces on an 8-in. nominal block, which would result from two such anchors located symmetrically and nonsymmetrically, were made. Treating the block as a rigid body, forces necessary to provide equilibrium were calculated. The applied forces resulted in bearing and shear stresses at the perimeters of the loaded block, which were not sufficient to pull the block from the remainder of the wall.

#### 3.8.4.5.7 Structural Acceptance Criteria - Allowable Stresses

##### *3.8.4.5.7.1 Normal Operating Conditions*

For normal operating conditions, allowable masonry working stress values are as specified in ACI 531-79. The allowable stresses are based on compressive strength of 700 psi on the gross area of the block. The value of $m_o$, the specified 28-day compressive strength of the mortar per ASTM C-270, is 750 psi.

##### *3.8.4.5.7.2 Safe Shutdown Earthquake*

The increase factors permitted by SRP 3.8.4 for load combinations containing SSE loads were used for evaluation with one exception. For tension normal to the bed joint, an increase factor of 1.5 versus 1.3 was used to qualify two walls. The 1.3 factor is exceeded by 10% for wall 3-17A-5 and 7% for wall 2-2I. This corresponds to increase factors of 1.43 and 1.38, based on the actual wall stresses, rather than 1.5. The allowable stresses identified in ACI 531 include a safety factor of 3. Therefore, the use of 1.43 and 1.38 as increase factors still provides margins of safety of 2.10 and 2.17 for the two walls and is judged to be acceptable for these limited cases.

#### 3.8.4.5.8 Evaluation Results

##### *3.8.4.5.8.1 General*

All masonry block walls at Ginna Station were inspected and found to be built in accordance with the original specifications and with appropriate inspection and construction techniques applicable at the time of construction. See Section 3.8.4.5.9.

Of the 56 safety-related panels, the modifications installed after the original IE Bulletin 80-11 evaluation resulted in 29 panels meeting current stress criteria.

In the analysis no credit was taken for either horizontal or vertical reinforcing. Of the 27 panels that required modification or further analysis, twelve contain vertical reinforcing and horizontal DUR-O-WALL joint reinforcement.

As noted in Section 3.8.4.5.1, one safety-related wall, 971-2M, is composed of 4-in. interlocking lead bricks. The wall, 2 ft 3 in. wide at the base and 5 ft 4 in. high, was analyzed taking no credit for the interlocking effect of the brick. The steel framing network surrounding the wall can adequately restrain the wall in one direction during an earthquake, and wall failure in the other direction will not affect any safety-related equipment. Wall 971-2M is therefore seismically acceptable. Thus, 26 panels remained for further analysis or modification.

A cracked section analysis was performed on one wall panel. Due to the minimum reinforcing available in the evaluated panel, no significant benefit was gained from the cracked section analysis. No walls have been qualified using cracked section analysis.

A seismic analysis of the 12 safety-related reinforced masonry block wall panels in the control building was conducted as documented in *Reference 48*. The methodology used to evaluate the walls in the inelastic range was previously used on the masonry walls at the San Onofre Nuclear Generating Station Unit 1 (SONGS 1). Correlation of this methodology to Ginna Station was confirmed by *Reference 49*.

From the elastic analysis, the seven spanning walls had stresses in the vertical rebar exceeding the criteria limit of 36 ksi by ratios ranging from 1.25 to 2.18. Therefore, all walls required qualification by the inelastic analysis methodology as discussed below.

### *3.8.4.5.8.2 Inelastic Analysis*

Spanning walls 971-1C and 971-6C and cantilever wall 973-4C were analyzed in detail. Spanning wall 971-1C is a 16 ft 10 in. high wall 38 ft 1 in. long between elevations 253 ft 8 in. and 271 ft 0 in. in the control building. It is reinforced with #3 bars at 32-in. centers and horizontally with DUR-O-WALL joint reinforcing. Spanning wall 971-6C is similar in construction and at the same elevation. Cantilever wall 973-4C has two layers of vertical rebars rather than being centrally reinforced as for the spanning walls.

The two walls were chosen because they represent the highest and lowest levels of overstress, thus enabling results for the other walls to be obtained by interpolation. The results of the two chosen walls indicated strains well within the criteria limits of masonry strain $E_m = 0.003$ and vertical steel strain ratio of $E_s/E_y = 45$.

With the interpolation of the result of the inelastic analysis of walls 971-1C and 971-6C, it was concluded that the remaining spanning walls will have similarly low material-strain ratios. Based on this it is considered that all spanning walls will perform satisfactorily under SSE loading with degrees of nonlinearity well within the capability of reinforced masonry.

The detailed model of the cantilever wall 973-4C was used for the nonlinear analysis. The results of the time histories showed that the masonry and steel strain ratios were well within the criteria limits.

Based on these analyses it is concluded that the reinforced masonry walls have ample ductility to resist the design SSE input motions.

### *3.8.4.5.8.3 Wall Modifications*

For the remaining 14 wall panels, RG&E used the following methods to ensure wall qualifications:

a. A wall was considered safety-related if equipment was located within one full wall height of the base of the wall. Rochester Gas and Electric Corporation investigated the justification of using less than one full wall height, if applicable, on a wall-by-wall basis. If it were concluded that the collapse mechanism is such that the equipment is not hit, no further evaluation would be performed.

b. If a wall failure could impact safety-related equipment, additional analysis would be performed to determine if the equipment would actually be damaged and inoperable. If the equipment could withstand the wall impact and remain operable, no modification would be performed.

c. Modifications to protect safety-related equipment potentially impacted by wall failure would be designed and installed so that wall failure has no safety consequences.

d. Wall modifications would be designed and installed such that the wall would meet the evaluation criteria.

The NRC evaluated RG&E's response to IE Bulletin 80-11, regarding masonry wall design adequacy and the commitments for the 14 wall panels requiring additional analysis or modification, and determined that RG&E has adequately addressed the concerns of IE Bulletin 80-11 (*Reference 50*).

The 14 wall panels have been qualified either by structural modifications to the panel to meet the evaluation criteria or by protection of the safety-related equipment subject to impact. Protective structures have been installed to protect the A and B main steam isolation valve operators and solenoid valves and the auxiliary feedwater check valves that were subject to impact by wall failure. The main steam isolation valve control cables have been rerouted so as not to be susceptible to damage from failed walls.

### 3.8.4.5.9 Materials, Quality Control, and Special Construction Techniques

The original Ginna Station project specifications identified the materials to be used for the construction of masonry walls as follows.

A. Concrete: ACI 318-63.

B. Steel: ASME Section III, Article CC-2000.

C. Brick: Facing brick shall conform to the requirements of ASTM Specifications C 216-65, Grade SW and Type FBS.

D. Concrete masonry units: Hollow, load-bearing units shall conform to ASTM C 90-665, Grade G-11. Interior non-load-bearing partitions shall be Haydite block.

E. Concrete masonry bed reinforcing: Reinforcing shall be Dur-O-Wall standard, truss design, or Hohmann & Barnard, Inc., Turs-Mesh, with a width 2 in. less than the nominal thickness of the wall. Reinforcing in exterior walls shall be galvanized in accordance with ASTM A 116-65, Class 1, specifications. Installation shall be in strict accordance with the manufacturer's recommendations.

F. Partition ties: 1-1/4 in. x 1/4 in. x 8 in. with 2-in. right-angle bends at either end, prime painted with 13-Y-5 zinc chromate primer as made by Mobil Chemical Company, Metuchen, New Jersey, or approved equivalent.

G. Anchors at columns: Anchors will be provided by others at 24-in. centers.

H. Control joints: Dur-O-Wall, wide flange, Rapid Control Joint.

I. Mortar:

    a. Mortar and mortar materials shall conform to the requirements of the property specifications of ASTM Specifications for Mortar for Unit Masonry C 270-64T, Type N.

        1. **Portland cement**: ASTM C 150-66, Type I or II.
        2. **Hydrated lime**: ASTM C 207-49, Type S, or Miracle Lime as made by G. & W. H. Corson, Plymouth Meeting, Pennsylvania.
        3. **Sand**: ASTM C 144-66T.
        4. **Water**: Water shall be clean and free of deleterious amounts of acids, alkali, or organic materials.
        5. **Mixing**: Mixing shall be done in a mechanical batch mixer. No more mortar shall be mixed at one time than can be used within 1.5 hours.

6. **Admixtures**: Salts and antifreeze compounds to lower the freezing point of mortar will not be permitted.

b. At the subcontractor's option, a prepared mortar may be used conforming to ASTM Specification C 91-66, Type II.

### *3.8.5 FOUNDATIONS*

The foundations of the interior containment structures, the auxiliary building, the screen house, and the intermediate building are founded on the bedrock of the Queenston formation, which is exhibited to be strong and fresh layers of shale, sandstone, and siltstone in the boring logs. The turbine building, control building, and the diesel generator building foundations were excavated and provided with lean concrete on compacted backfill to a depth whereby the elevation of the top of the fill material was coincident with the elevation of the bottom of the concrete foundation of that particular building.

The standby auxiliary feedwater building is on pilings to the bedrock. The technical support center is on the second floor of the all-volatile-treatment building, which is founded on a concrete mat.

The (CPB) foundation was designed as a Seismic Category I structure. In order to assure that suitable soil exists that is capable of supporting the CPB superstructure and the 125 ton crane an investigation revealed and determined that drilled caissons would be needed to prevent building displacement as the 125 ton crane tranverses into and out of the Aux building. The caissons also act to limit the horizontal and vertical seismic motions exerted during a postulated seismic event. The drilled caissons were installed into the bedrock beneath the CPB / crane foundations.

The major structures of Ginna Station have experienced no visible evidence of settlement since the construction of the station. During the SEP and evaluation of Topic II-4.F, the NRC concluded (*Reference 51*) that the settlement of foundations and buried equipment is not a safety concern for Ginna Station.

## REFERENCES FOR SECTION 3.8

1. T. C. Waters, N. T. Barrett, "Prestressed Concrete Pressure Vessels for Nuclear Reactors," Journal of the British Nuclear Energy Society, Vol. 2, 1963.

2. M. Bender, "A Status Report on Prestressed Concrete Reactor Pressure Vessel Technology," Nuclear Structural Engineering, 1965.

3. Bengt, B. Broms, "Crack Width and Crack Spacing in Reinforced Concrete Members," ACI Journal, October 1965.

4. Bengt, B. Broms and Leroy A. Lutz, "Effects of Arrangement of Reinforcement on Crack Width and Spacing of Reinforced Concrete Members," ACI Journal, November 1965.

5. Peter Gergeley and Leroy A. Lutz, "Maximum Crack Width in Reinforced Concrete Flexural Members," presented at ACI Symposium on Cracking of Concrete, March 7-10, 1966.

6. Suggested Design of Joints and Connections in Precast Structural Concrete, Report of the Joint ACI-ASCE Committee 5-12 (712) of the Task Committee on Precast Structural Concrete Design and Construction of the Committee on Masonry and Reinforced Concrete.

7. S. S. Morris and W. S. Garrett, "The Raising and Strengthening of Steenbrar Dam," Proceedings, I.C.E., Vol. 1, Part 1, No. 1; Discussion, Vol. 1, Part 1, No. 4, 1956.

8. O. C. Zienkiewica and R. W. Gerstner, "Stress Analysis and Special Problems of Prestressed Dams," Journal of the Power Division, ASCE, January 1961.

9. A. Eberhardt and J. A. Voltrop, "1300-Ton Capacity Prestressed Anchors Stabilize Dam," Journal of the Prestressed Concrete Institute, Vol. 10, No. 4, August 1965.

10. VSL Prestressed Rock and Aluvium Anchors, Losiner and Company, SA, March 1965.

11. F. S. Leonhardt, Prestressed Concrete Design and Construction, Wilhelm Ernst and Sons, Munich, 1964, Figure 9.4, page 271.

12. Y. Guyon, Prestressed Concrete, John Wiley and Sons, 1960.

13. "Problems of the Design and Construction of Earthquake Resistant Prestressed Concrete Structures," PCI Journal, June 1966.

14. J. D. Gilchrist, "The Stress Corrosion Cracking of High Tensile Steel Wire," Journal of Metallurgical Club, 1960-61.

15. S. Timoshenko, Strength of Materials, Part II, Second Edition, D. Ban Nostrand Company, Inc., 1952.

16. M. Hetenye, Beams on Elastic Foundations, University of Michigan Press, 1961.

17. A. Kalnins, "Analysis of Shells of Revolution Subjected to Symmetrical and Nonsymmetrical Loads," Journal of Applied Mechanics, September 1964.

18. ACI Committee 408, "Bond Stress - The State of the Art," ACI Journal Proceedings, Vol. 63, No. 11, November 1966.

19. R. E. Untrauer and R. L. Henry, "Influence of Normal Pressure on Bond Strength," ACI Journal Proceedings, Vol. 62, No. 5, May 1965.

20. S. Timoshenko and Woinowsky-Krieger, Theory of Plates and Shells, McGraw Hill Book Company, 1959.

21. S. Timoshenko and J. N. Goodier, Theory of Elasticity, McGraw Hill Company, Inc., 1951.

22. L. D. Kriz and C. H. Raths, "Connection in Precast Concrete Structures - Strength of Corbels," Journal of the Prestressed Concrete Institute, Vol. 10, No. 1, February 1965.

23. A. G. Young, "Design of Liners for Reactor Vessels," Group J., Paper 57, Prestressed Concrete Pressure Vessels, London, 1967.

24. Letter from R. W. Kober, RG&E, to D. M. Crutchfield, NRC, Subject: Containment Vessel Tendon Evaluation Program, R. E. Ginna Nuclear Power Plant, dated March 26, 1984.

25. Franklin Research Center Technical Evaluation Report, Design Codes, Design Criteria and Load Combinations, TER C5257-322, May 27, 1982. (Enclosure to letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Design Codes, Design Criteria, and Load Combinations, dated January 4, 1983.)

26. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Topic III-7.B, Design Codes, Design Criteria and Load Combinations, R. E. Ginna Nuclear Power Plant, dated April 21, 1982.

27. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic III-7.B, Design Codes, Design Criteria, and Load Combinations, dated May 27, 1983.

28. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topics II-2.A, III-2, III-4.A, and III-7.B, dated April 22, 1983.

29. Letter from J. E. Maier, RG&E, to D. M. Crutchfield, NRC, Subject: SEP Topic III-7.B, Load Combinations (Containment Liner Analysis), dated April 28, 1983.

30. U.S. Nuclear Regulatory Commission, Seismic Review of the Robert E. Ginna Nuclear Power Plant as Part of the Systematic Evaluation Program, NUREG/CR-1821, November 15, 1980. (Enclosure to letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Seismic Design, Construction and Component Integrity, dated January 7, 1981.)

31. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Integrated Plant Safety Assessment Report, Section 4.8, Section 4.11, and Section 4.17.1, dated August 22, 1983.

32. U.S. Nuclear Regulatory Commission, Structural Review of the Robert E. Ginna Nuclear Power Plant Under Combined Loads for the Systematic Evaluation Program, NUREG/CR-2580, December 1981.

33. Draft Report, Safety Evaluation Report on Containment Pressure and Heat Removal Capability, SEP Topic VI-3, and Mass and Energy Release for Possible Pipe Break Inside Containment, SEP Topic VI-2.D, for the R. E. Ginna Nuclear Power Plant, US NRC Docket No. 50-244, 1981.

34. Swanson Analysis Systems, Inc., ANSYS, Revision 3, Houston, Pennsylvania.

35. Structures Research Associates, FASOR - Field Analysis of Shells of Revolution, Laguna Beach, California.

36. G. A. Cohen, FASOR - A Program for Stress, Buckling, and Vibration of Shells of Revolution, Structures Research Associates, Laguna Beach, California.

37. T. L. Windstead, E. G. Burdette, and D. R. Armentrout, "Liner Anchorage Analysis for Nuclear Containments," Journal of Structural Division ASCE, Vol. 101, No. ST10, Proc. Paper 11635, October 1975, P. 2103.

38. U.S. Nuclear Regulatory Commission, Response of the Zion and Indian Point Containment Buildings to Severe Accident Pressures, NUREG/CR 2569, May 1982.

39. A. Kalnins, Computer Program for the Stress Analysis of Axisymmetric Thin, Elastic Shells, 1976.

40. J. G. Ollgaard, R. G. Slutter, and J. W. Fisher, "Shear Strength of Stud Connections in Lightweight and Normal Weight Concrete," AISC Engineering Journal, pp. 55-64, April 1971.

41. TRW Report, Design Data - Nelson Concrete Anchor.

42. D. R. Barna, Shear Strength of Deformed Bar Anchors, Nelson Stud Welding, April 1972.

43. C. G. Goble, "Shear Strength of Thin Flange Composite Specimens", AISC Engineering Journal, pp. 62-65, April 1968.

44. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: Integrated Plant Safety Assessment Report, Section 4.17.1, Containment Liner Insulation, dated August 1, 1983.

45. Letter from L. D. White, Jr., RG&E, to D. L. Ziemann, NRC, Subject: Systematic Evaluation - NRC Use of the Containment and Internals Structural Model, dated November 30, 1979.

46. Letter from L. D. White, Jr., RG&E, to D. L. Ziemann, NRC, Subject: Systematic Evaluation Program Seismic Review, dated May 22, 1979.

47. S. J. Sackett, Users Manual for SAP4, A Modified and Extended Version of the U. S. Berkeley SAP IV Code, UCID 18226, Lawrence Livermore National Laboratory, Livermore, CA, 1979.

48. Computech Engineering Services, Reinforced Masonry Wall Evaluation - Evaluation of Control Building Reinforced Walls, Report No. R562-N4, December 16, 1985. (Enclosure to letter from R. W. Kober, RG&E, to G. E. Lear, NRC, Subject: Seismic Masonry Wall Analysis, R. E. Ginna Nuclear Power Plant, dated December 19, 1985.)

49. Computech Engineering Services, Reinforced Masonry Wall Evaluation - Correlation of SONGS-1 Test Data, Report No. R562-N3, December 15, 1985. (Enclosure to letter from R. W. Kober, RG&E, to G. E. Lear, NRC, Subject: Masonry Wall Design, R. E. Ginna Nuclear Power Plant, dated January 14, 1986.)

50. Letter from D. C. DiIanni, NRC, to R. W. Kober, RG&E, Subject: Resolution of Masonry Wall Design Integrity IE Bulletin 80-11 for R. E. Ginna, dated December 12, 1986.

51. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: SEP Topics II-4.D, Stability of Slopes, and II-4.F, Settlement of Foundations and Buried Equipment, dated February 19, 1982.

52. Letter from J. A. Zwolinski, NRC, to R. W. Kober, RG&E, Subject: Safety Evaluation Containment Vessel Tendon Surveillance Program, dated August 19, 1985.

53. Letter from R. W. Kober, RG&E, to G. E. Lear, NRC, Subject: Containment Vessel Tendon Investigation, dated December 20, 1985.

54. Letter from M.C. Thadani, NRC, to J.E. Pacher, Ginna NPP, Subject: R.E. Ginna Nuclear Power Plant – Issuance of Amendment RE: Revision to Technical Specification Section 3.6.5, “Containment Air Temperature.” (TAC NO. MF0900), dated August 12, 2014. (License Amendment No. 116)

**Table 3.8-1a**
**COMPUTER PROGRAM SAND INPUT FOR CONTAINMENT SEISMIC ANALYSIS DIMENSIONS AND FORMULA**

## MEMBER DIMENSIONS

Each member is assumed to have uniform area with cross section as at the mid-point of the member

$$\text{Radius} = \sqrt{(R1^2 - h^2)}$$

(Equation Formula T-3.8-1)

$R_1{}^2 = 660^2 = 435{,}600$

$R_2{}^2 = 630^2 = 396{,}900$

## Table 3.8-1b
## COMPUTER PROGRAM SAND INPUT FOR CONTAINMENT SEISMIC ANALYSIS DIMENSION CALCULATIONS

| Member[a] | h | $h^2$ | $R1^2-h^2$ | $R2^2-h^2$ | External Radius | Internal Radius | Thickness |
|---|---|---|---|---|---|---|---|
| 13-12 | 575 | 330,600 | 105,000 | 66,300 | 324.0 | 257.5 | 66.5 |
| 12-11 | 405 | 164,000 | 271,600 | 232,900 | 521.2 | 482.6 | 38.6 |
| 11-10 | 235 | 55,230 | 380,370 | 341,670 | 616.6 | 584.6 | 32.0 |
| 10-9 | 77 | 5,930 | --- | 391,000 | 672 | 625.3 | 46.7 |
| 9-8 | | | | | 672 | | 42.0 |
| 8-7 | | | | | 672 | | 42.0 |
| 7-6 | | | | | 672 | | 42.0 |
| 6-5 | | | | | 672 | | 42.0 |
| 5-4 | | | | | 672 | | 42.0 |
| 4-3 | | | | | 672 | | 42.0 |
| 3-2 | | | | | 672 | | 42.0 |
| 2-1 | | | | | 672 | | 42.0 |

a See Figure 3.8-10

## Table 3.8-1c
## COMPUTER PROGRAM SAND INPUT FOR CONTAINMENT SEISMIC ANALYSIS NATURAL FREQUENCIES AND RESPONSE

| Mode | Frequency (Hz) | Period (Sec) | Modal Effective Mass (x $10^6$ ) | Response Accelerations (2% Damping) 0.08g | Response Accelerations (2% Damping) 0.20g |
|---|---|---|---|---|---|
| 1 | 6.95 | 0.144 | 18.46 | 0.14 | 0.36 |
| 2 | 19.19 | 0.052 | 4.78 | 0.09 | 0.22 |
| 3 | 34.44 | 0.029 | 0.30 | 0.08 | 0.20 |
| 4 | 38.01 | 0.026 | 0.92 | 0.08 | 0.20 |
| 5 | 54.63 | 0.018 | 0.51 | 0.08 | 0.20 |
| 6 | 64.82 | 0.015 | 0.05 | 0.08 | 0.20 |
| | | Total | 25.02 x $10^6$ lbs | | |

## Table 3.8-2
## MAJOR STRUCTURES FOR WHICH PRESTRESSED ROCK ANCHORS WERE USED

**DAMS**

Little Goose Lock & Dam

Snake River, Oregon, Washington and Idaho

Designed October 1963 by U. S. Army Engineers District

Walla Walla, Washington

Wanapum Hydro Station - Washington, 1962

Enestina Dam, Brazil - 1951-1954

Allt-Wa-Lairige Dam, Scotland - 1954-1956

Tourtemegne Dam, Switzerland - 1957-1958

Swallow Falls, South Africa - 1956-1958

Catagunya Dam, Tasmania - 1959-1961

Meadowbanks Dam, Tasmania - 1964

**BRIDGES**

Feather River Suspension Bridge

Oroville, California

Designed by California Department of Water Resources

**TIE BACKS**

Montreal Subway

Designed by Bealieu-Trudeau and Associates, Montreal

New York State's University Teaching Hospital in

Syracuse, New York

Designed by DiStasion and Van Buren

Washington Hilton Hotel

Designed by Wayman C. Wing

University of California

San Francisco Medical Center

Designed by Reid and Tarics

New York Life Insurance Company

New York City

Designed by Edwards and Hjorth

**SPECIAL STRUCTURES**

Test Facility for Saturn Rocket

Engines at Edwards Air Force Base

Designed by Corps of Engineers, Los Angeles

Research by Aero-Jet General Corporation

**Table 3.8-3**
**PROPERTIES AND TESTS FOR CONTAINMENT ANCHOR AND TENDON CORROSION INHIBITOR**

**Physical Properties**

| **Item** | **Range** | **Method** |
|---|---|---|
| Specific gravity | 0.88-0.90 | ASTM D-287 |
| Weight/gal | 7.35-7.50 lb | --- |
| Pour point | 110°F-120 °F | ASTM D-97 |
| Flash point (COC) | 400 °F, Minimum | ASTM D-92 |
| Viscosity at 130 °F | 575-635 SSU | ASTM D-88 |
| Viscosity at 150 °F | 135-145 SSU | ASTM D-88 |
| Viscosity at 210 °F | 65-75 SSU | ASTM D-88 |
| Penetration (cone) at 77 °F | 328-367 Sec | ASTM D-937 |
| Thermal conductivity | 0.12 Btu/hr/ft$^2$/ °F/ft Thickness (approximate) | --- |
| Specific heat (heat capacity) | 0.51 Btu/lb/°F (approximate) | --- |
| Shrinkage factor from 150 °F to 75 °F | 3.5% - 4.5% | ---- |
| Accelerated Corrosion Test Results | | |
| Humidity cabinet (JAN-H-792) | 300 hr | ASTM D-1748-62T |
| Salt spray cabinet | 75 hr | ASTM B-117-62 (Salt Fog Test) |

## Table 3.8-4
## ALLOWABLE STRESSES

| Load[a] Combination | Actual Maximum Tensile Stress (ksi) | Average Shear Stress Capability [b] (ksi) | Actual Average Shear Stress (ksi) | Ultimate Tensile Stress Capability [c] (ksi) |
|---|---|---|---|---|
| **a** | 38.0 | 0 | 0 | 38.0 |
| **b** | 31.6 | 10.5 | 4.4 | 37.0 |
| **c** | 25.4 | 14.1 | 8.7 | 33.8 |

a. Load (a) C = 0.95 D + 1.5 P + 1.0 T
   Load (b) C = 0.95 D + 1.25 P + 1.0 T′ + 1.25 E
   Load (c) C = 0.95 D + 1.0 P + 1.0 $\underline{T}$ + 1.0 E′

b. For the given tensile stress.

c. For the given shear stress.

## Table 3.8-5a
## CONTAINMENT STRUCTURE STRESSES - LOADING #1 DEAD LOAD

| | Stress Resultants | | Stress Couples | | | |
|---|---|---|---|---|---|---|
| Location in Feet Up From Base Element No. | Meridional $N\phi$ | Hoop $N\theta$ | Meridional $M\phi$ | Hoop $M\theta$ | Meridional Shear $V\phi$ | Radial Displacement $\delta R$ |
| Base 0 | -70.9 | 0 | 0 | 0 | 0 | 0 |
| 3 | -69.4 | 0 | 0 | 0 | 0 | 0 |
| 6 | -67.8 | 0 | 0 | 0 | 0 | 0 |
| 10 | -65.2 | 0 | 0 | 0 | 0 | 0 |
| 15 | -63.3 | 0 | 0 | 0 | 0 | 0 |
| 20 | -60.5 | 0 | 0 | 0 | 0 | 0 |
| 30 | -55.7 | 0 | 0 | 0 | 0 | 0 |
| 40 | -50.5 | 0 | 0 | 0 | 0 | 0 |
| 60 | -40.3 | 0 | 0 | 0 | 0 | 0 |
| 75 | -32.7 | 0 | 0 | 0 | 0 | 0 |
| 85 | -27.6 | 0 | 0 | 0 | 0 | 0 |
| 90 | -25.0 | 0 | 0.0 | 0 | 0 | 0 |
| 95 | -22.5 | 0 | +20.0 | 0 | 0 | 0 |
| Springline 99 | -20.4 | +20.4 | +27.8 | 0 | 0 | 0 |
| 102 | -19.4 | +18.3 | +31.0 | 0 | 0 | 0 |
| 105 | -18.5 | +16.2 | +32.3 | 0 | 0 | 0 |
| Dome Anchor 108 | -17.5 | +14.1 | +31.0 | 0 | 0 | 0 |
| -111 | -16.8 | +12.2 | +26.5 | 0 | 0 | 0 |
| 111 | -16.8 | +12.2 | +28.0 | 0 | 0 | 0 |
| +111 | -16.8 | +12.2 | -1.5 | 0 | 0 | 0 |
| 114 | -16.1 | +10.5 | 0.0 | 0 | 0 | 0 |
| 117 | -15.4 | +8.7 | 0.0 | 0 | 0 | 0 |
| 123 | -14.3 | +5.5 | 0.0 | 0 | 0 | 0 |
| 130 | -13.2 | +2.2 | 0.0 | 0 | 0 | 0 |
| Apex | -10.2 | -10.2 | 0.0 | 0.0 | 0.0 | 0.0 |

## Table 3.8-5b
## CONTAINMENT STRUCTURE STRESSES - LOADING #2 FINAL PRESTRESS - 636 K/ TENDON

$$N\phi = \frac{160x636}{108.5\pi} = 299k/in. -$$

| Location in Feet Up From Base Element No. | Stress Resultants: Meridional Nϕ | Stress Resultants: Hoop Nθ | Stress Couples: Meridional Mϕ | Stress Couples: Hoop Mθ | Meridional Shear Vϕ | Radial Displacement δ R |
|---|---|---|---|---|---|---|
| Base 0 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 3 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 6 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 10 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 15 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 20 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 30 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 40 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 60 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 75 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 85 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 90 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 95 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| Springline 99 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 102 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 105 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| Dome Anchor 108 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| -111 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| 111 | -299.0 | 0 | 0 | 0 | 0 | 0 |
| +111 | 0 | 0 | 0 | 0 | 0 | 0 |
| 114 | 0 | 0 | 0 | 0 | 0 | 0 |

$$N\phi = \frac{160x636}{108.5\pi} = 299k/in. -$$

| Location in Feet Up From Base Element No. | Stress Resultants<br>Meridional Nϕ | Hoop Nθ | Stress Couples<br>Meridional Mϕ | Hoop Mθ | Meridional Shear Vϕ | Radial Displacement δ R |
|---|---|---|---|---|---|---|
| 117 | 0 | 0 | 0 | 0 | 0 | 0 |
| 123 | 0 | 0 | 0 | 0 | 0 | 0 |
| 130 | 0 | 0 | 0 | 0 | 0 | 0 |
| Apex | 0 | 0 | 0 | 0 | 0 | 0 |

## Table 3.8-5c
## CONTAINMENT STRUCTURE STRESSES - LOADING #3 OPERATING TEMPERATURE - WINTER

*$N\theta$ = kry = 116.5 (651) 12 y/1000 = 912 y′c k/ft $\delta r$ = -0.143*

| | Stress Resultants | | Stress Couples | | | |
|---|---|---|---|---|---|---|
| **Location in Feet Up From Base Element No.** | **Meridional $N\phi$** | **Hoop $N\theta$** | **Meridional $M\phi$** | **Hoop $M\theta$** | **Meridional Shear $V\phi$** | **Radial Displacement $\delta R$** |
| Base 0 | 0.0 | +130.2 | 0.0 | 99.5 | -6.6 | 0.000 |
| 3 | 0.0 | +95.6 | -9.7 | 99.5 | -0.3 | .038 |
| 6 | 0.0 | +65.0 | -3.6 | 99.5 | 4.0 | -.075 |
| 10 | 0.0 | +27.3 | +19.8 | 99.5 | 7.2 | -.113 |
| 15 | 0.0 | 0.0 | +59.7 | 99.5 | 8.4 | -.143 |
| 20 | 0.0 | -14.6 | +100.2 | 99.5 | 7.6 | -.159 |
| 30 | 0.0 | -19.2 | +160.4 | 99.5 | 4.3 | -.164 |
| 40 | 0.0 | +11.8 | +188.0 | 99.5 | 1.5 | -.156 |
| 60 | 0.0 | 0.0 | +192.3 | 99.5 | -0.4 | -.144 |
| 75 | 0.0 | 0.0 | +185.6 | 99.5 | 0.0 | -.142 |
| 85 | 0.0 | 0.0 | +186.0 | 99.5 | 0.0 | -.143 |
| 90 | 0.0 | +20.0 | +149.1 | 99.5 | +0.8 | -.121 |
| 95 | 0.0 | +34.6 | +157.3 | 99.5 | +3.1 | -.105 |
| Springline 99 | 0.0 | +48.3 | +173.8 | +99.5 | +5.9 | -.090 |
| 102 | 0.0 | -24.8 | +28.1 | 28.2 | -1.0 | -.093 |
| 105 | 0.0 | -12.1 | +31.9 | 28.2 | +1.0 | -.072 |
| Dome Anchor 108 | 0.0 | -3.7 | +31.8 | 28.2 | +1.2 | -.058 |
| -111 | 0.0 | 0.0 | +28.2 | 28.2 | 0.0 | -.052 |
| 111 | 0.0 | 0.0 | +28.2 | 28.2 | 0.0 | -.052 |
| +111 | 0.0 | 0.0 | +28.2 | 28.2 | 0.0 | -.052 |
| 114 | 0.0 | 0.0 | +28.2 | 28.2 | 0.0 | -.052 |
| 117 | 0.0 | 0.0 | +28.2 | 28.2 | 0.0 | -.052 |
| 123 | 0.0 | 0.0 | +28.2 | 28.2 | 0.0 | -.052 |
| 130 | 0.0 | 0.0 | +28.2 | 28.2 | 0.0 | -.052 |

***$N\theta$ = kry = 116.5 (651) 12 y/1000 = 912 y′c k/ft $\delta r$ = -0.143***

| | Stress Resultants | | Stress Couples | | | |
|---|---|---|---|---|---|---|
| **Location in Feet Up From Base Element No.** | **Meridional $N\phi$** | **Hoop $N\theta$** | **Meridional $M\phi$** | **Hoop $M\theta$** | **Meridional Shear $V\phi$** | **Radial Displacement $\delta$ R** |
| Apex | 0.0 | 0.0 | +28.2 | 28.2 | 0.0 | -.052 |

## Table 3.8-5d
## CONTAINMENT STRUCTURE STRESSES - LOADING #4 OPERATING TEMPERATURE - SUMMER

| | Stress Resultants | | Stress Couples | | | |
|---|---|---|---|---|---|---|
| Location in Feet Up From Base Element No. | Meridional Nϕ | Hoop Nθ | Meridional Mϕ | Hoop Mθ | Meridional Shear Vϕ | Radial Displacement δ R |
| Base 0 | 0.0 | -130.2 | 0.0 | 0.0 | +6.6 | 0.000 |
| 3 | 0.0 | -38.3 | +16.1 | 0.0 | +4.2 | +.101 |
| 6 | 0.0 | -30.1 | +25.9 | 0.0 | +2.4 | +.110 |
| 10 | 0.0 | -19.1 | +31.6 | 0.0 | +0.6 | +.122 |
| 15 | 0.0 | -15.5 | +30.9 | 0.0 | -0.7 | +.126 |
| 20 | 0.0 | -2.7 | +25.7 | 0.0 | -1.3 | +.140 |
| 30 | 0.0 | +2.7 | +12.5 | 0.0 | -1.2 | +.146 |
| 40 | 0.0 | +2.7 | +3.3 | 0.0 | -0.6 | +.146 |
| 60 | 0.0 | 0.0 | -1.4 | 0.0 | 0.0 | +.143 |
| 75 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| 85 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| 90 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| 95 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| Springline 99 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| 102 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| 105 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| Dome Anchor 108 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| -111 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| 111 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| +111 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| 114 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| 117 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| 123 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| 130 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |
| Apex | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | +.143 |

## Table 3.8-5e
## CONTAINMENT STRUCTURE STRESSES - LOADING #5 INTERNAL PRESSURE

**p = 60psig δRD = 0.383 in. δR = 0.492 in.**

| | Stress Resultants | | Stress Couples | | | |
|---|---|---|---|---|---|---|
| **Location in Feet Up From Base Element No.** | **Meridional Nϕ** | **Hoop Nθ** | **Meridional Mϕ** | **Hoop Mθ** | **Meridional Shear Vϕ** | **Radial Displacement δ R** |
| Base 0 | 227.0 | +79.6 | -30.0 | 0.0 | +55.3 | .009 |
| 3 | 227.0 | +127.4 | +106.0 | 0.0 | +36.2 | .149 |
| 6 | 227.0 | +199.4 | +190.6 | 0.0 | +20.9 | .226 |
| 10 | 227.0 | +282.2 | +243.0 | 0.0 | +6.2 | .314 |
| 15 | 227.0 | +363.1 | +243.6 | 0.0 | -4.8 | .401 |
| 20 | 227.0 | +418.8 | +205.7 | 0.0 | -9.7 | .460 |
| 30 | 227.0 | +469.0 | +102.8 | 0.0 | -9.5 | .514 |
| 40 | 227.0 | +473.2 | +28.9 | 0.0 | -5.2 | .518 |
| 60 | 227.0 | +454.2 | +10.8 | 0.0 | 0.0 | .498 |
| 75 | 227.0 | +454.0 | -7.1 | 0.0 | 0.0 | .492 |
| 85 | 227.0 | +438.0 | -3.9 | 0.0 | 0.0 | .480 |
| 90 | 227.0 | +428.0 | +34.7 | 0.0 | -0.4 | .470 |
| 95 | 227.0 | +354.0 | +7.7 | 0.0 | -12.8 | .388 |
| Springline 99 | 227.0 | +322.0 | -60.5 | 0.0 | -21.6 | .353 |
| 102 | 227.0 | +210.0 | -126.7 | 0.0 | -18.2 | .346 |
| 105 | 0.0 | +182.0 | -199.1 | 0.0 | -25.0 | .301 |
| Dome Anchor 108 | 0.0 | +229.0 | 19.8 | 0.0 | +3.1 | .368 |
| -111 | 0.0 | +243.0 | +10.3 | 0.0 | +3.3 | .402 |
| 111 | 227.0 | +243.0 | +10.3 | 0.0 | +3.3 | .402 |
| +111 | 227.0 | +243.0 | +10.3 | 0.0 | +3.3 | .402 |
| 114 | 227.0 | +243.0 | +4.3 | 0.0 | +2.0 | 402 |
| 117 | 227.0 | +238.0 | +0.2 | 0.0 | 0.8 | .393 |
| 123 | 227.0 | +230.0 | 0.0 | 0.0 | 0.0 | 388 |
| 130 | 227.0 | 227.0 | 0.0 | 0.0 | 0.0 | .383 |
| Apex | 227.0 | 227.0 | 0.0 | 0.0 | 0.0 | .383 |

**Table 3.8-5f**
**CONTAINMENT STRUCTURE STRESSES - LOADING #6 ACCIDENT TEMPERATURE - P = 60 PSIG, T = 286°F**

| | Stress Resultants | | Stress Couples | | | |
|---|---|---|---|---|---|---|
| Location in Feet Up From Base Element No. | Meridional Nϕ | Hoop Nθ | Meridional Mϕ | Hoop Mθ | Meridional Shear Vϕ | Radial Displacement δ R |
| Base 0 | 8.0 | -1.5 | 0.0 | 0.0 | 1.2 | 0.000 |
| 3 | 8.0 | -0.6 | 2.5 | 0.0 | -0.8 | .001 |
| 6 | 8.0 | +1.2 | 4.3 | 0.0 | 0.5 | .003 |
| 10 | 8.0 | +3.0 | 5.5 | 0.0 | 0.1 | .005 |
| 15 | 8.0 | +5.0 | 5.5 | 0.0 | -0.1 | .007 |
| 20 | 8.0 | +6.0 | 4.6 | 0.0 | -0.2 | .008 |
| 30 | 8.0 | +6.7 | 2.3 | 0.0 | -0.2 | .009 |
| 40 | 8.0 | +6.7 | 0.6 | 0.0 | -0.1 | .009 |
| 60 | 8.0 | +6.7 | -0.2 | 0.0 | 0.0 | .009 |
| 75 | 8.0 | +6.7 | 0.0 | 0.0 | 0.0 | .009 |
| 85 | 8.0 | +25.8 | -80.0 | 0.0 | 0.0 | .030 |
| 90 | 8.0 | +54.1 | -85.7 | 0.0 | +0.9 | .061 |
| 95 | 8.0 | +102.4 | -66.8 | 0.0 | +6.8 | .114 |
| Springline 99 | 8.0 | +120.7 | -28.4 | 0.0 | +13.7 | .134 |
| 102 | 8.0 | +54.0 | -0.3 | 0.0 | -5.6 | .179 |
| 105 | 8.0 | +84.4 | +8.7 | 0.0 | -1.0 | .229 |
| Dome Anchor 108 | 8.0 | +103.7 | +8.2 | 0.0 | +0.9 | .261 |
| -111 | 111.0 | 111.0 | +5.0 | 0.0 | +1.1 | .273 |
| 111 | 111.0 | 111.0 | 0.0 | 0.0 | 0.0 | .273 |
| +111 | 111.0 | 111.0 | 0.0 | 0.0 | 0.0 | .273 |
| 114 | 111.0 | 111.0 | 0.0 | 0.0 | 0.0 | .273 |
| 117 | 111.0 | 111.0 | 0.0 | 0.0 | 0.0 | .273 |
| 123 | 111.0 | 111.0 | 0.0 | 0.0 | 0.0 | .273 |
| 130 | 111.0 | 111.0 | 0.0 | 0.0 | 0.0 | .273 |
| Apex | 111.0 | 111.0 | 0.0 | 0.0 | 0.0 | .273 |

**Table 3.8-5g**
**CONTAINMENT STRUCTURE STRESSES - LOADING #7 ACCIDENT TEMPERATURE - P = 90 PSIG, T = 312°F**

| | Stress Resultants | | Stress Couples | | | |
|---|---|---|---|---|---|---|
| Location in Feet Up From Base Element No. | Meridional Nϕ | Hoop Nθ | Meridional Mϕ | Hoop Mθ | Meridional Shear Vϕ | Radial Displacement δ R |
| Base 0 | 8.0 | -1.5 | 0.0 | 0.0 | 1.2 | 0.000 |
| 3 | 8.0 | -0.6 | 2.5 | 0.0 | 0.8 | .001 |
| 6 | 8.0 | +1.2 | 4.3 | 0.0 | 0.5 | .003 |
| 10 | 8.0 | +3.0 | 5.5 | 0.0 | 0.1 | .005 |
| 15 | 8.0 | +5.0 | 5.5 | 0.0 | -0.1 | .007 |
| 20 | 8.0 | +6.0 | 4.6 | 0.0 | -0.2 | .008 |
| 30 | 8.0 | +6.7 | 2.3 | 0.0 | -0.2 | .009 |
| 40 | 8.0 | +6.7 | 0.6 | 0.0 | -0.1 | .009 |
| 60 | 8.0 | +6.7 | -0.2 | 0.0 | 0.0 | .009 |
| 75 | 8.0 | +6.7 | 0.0 | 0.0 | 0.0 | .009 |
| 85 | 8.0 | +35.0 | -90.0 | 0.0 | 0.0 | .040 |
| 90 | 8.0 | +61.4 | -97.6 | 0.0 | 1.0 | .069 |
| 95 | 8.0 | +97.9 | -76.1 | 0.0 | 7.7 | .109 |
| Springline 99 | 8.0 | +134.5 | -32.3 | 0.0 | +15.6 | .149 |
| 102 | 8.0 | +59.8 | -0.3 | 0.0 | -6.4 | .200 |
| 105 | 8.0 | +95.6 | +10.0 | 0.0 | -1.1 | .259 |
| Dome Anchor 108 | 8.0 | +119.9 | +9.8 | 0.0 | +1.0 | .299 |
| -111 | +126.0 | +126.0 | +5.7 | 0.0 | 1.3 | .309 |
| 111 | +126.0 | +126.0 | 0.0 | 0.0 | 0.0 | .309 |
| +111 | +126.0 | 126.0 | 0.0 | 0.0 | 0.0 | .309 |
| 114 | +126.0 | +126.0 | 0.0 | 0.0 | 0.0 | .309 |
| 117 | +126.0 | +126.0 | 0.0 | 0.0 | 0.0 | .309 |
| 123 | +126.0 | +126.0 | 0.0 | 0.0 | 0.0 | .309 |
| 130 | +126.0 | +126.0 | 0.0 | 0.0 | 0.0 | .309 |

| | Stress Resultants | | Stress Couples | | | |
|---|---|---|---|---|---|---|
| Location in Feet Up From Base Element No. | Meridional $N\phi$ | Hoop $N\theta$ | Meridional $M\phi$ | Hoop $M\theta$ | Meridional Shear $V\phi$ | Radial Displacement $\delta$ R |
| Apex | +126.0 | +126.0 | 0.0 | 0.0 | 0.0 | .309 |

## Table 3.8-5h
## CONTAINMENT STRUCTURE STRESSES - LOADING #8 0.10G EARTHQUAKE - HORIZONTAL + VERTICAL COMPONENT

| | Stress Resultants | | Stress Couples | | | |
|---|---|---|---|---|---|---|
| Location in Feet Up From Base Element No. | Meridional Nϕ | Hoop Nθ | Meridional Mϕ | Hoop Mθ | Meridional Shear Vϕ | Radial Displacement δ R |
| Base 0 | 70.3 | 0 | 0 | 0 | 0 | 0 |
| 3 | +68.3 | 0 | 0 | 0 | 0 | .002 |
| 6 | +66.3 | 0 | 0 | 0 | 0 | .003 |
| 10 | +63.6 | 0 | 0 | 0 | 0 | .005 |
| 15 | +60.2 | 0 | 0 | 0 | 0 | .007 |
| 20 | +56.9 | 0 | 0 | 0 | 0 | .010 |
| 30 | +50.3 | 0 | 0 | 0 | 0 | .016 |
| 40 | +46.7 | 0 | 0 | 0 | 0 | .021 |
| 60 | +31.6 | 0 | 0 | 0 | 0 | .034 |
| 75 | +23.3 | 0 | 0 | 0 | 0 | .044 |
| 85 | +18.4 | 0 | 0 | 0 | 0 | .050 |
| 90 | +16.1 | 0 | 0 | 0 | 0 | .053 |
| 95 | +14.0 | 0 | 0 | 0 | 0 | .055 |
| Springline 99 | +12.3 | 0 | 0 | 0 | 0 | .058 |
| 102 | +11.2 | 0 | 0 | 0 | 0 | .059 |
| 105 | +10.0 | 0 | 0 | 0 | 0 | .062 |
| Dome Anchor 108 | +9.1 | 0 | 0 | 0 | 0 | .062 |
| -111 | +8.2 | 0 | 0 | 0 | 0 | .063 |
| 111 | +8.2 | 0 | 0 | 0 | 0 | .063 |
| +111 | +8.2 | 0 | 0 | 0 | 0 | .063 |
| 114 | +7.4 | 0 | 0 | 0 | 0 | .064 |
| 117 | +6.5 | 0 | 0 | 0 | 0 | .064 |
| 123 | +4.9 | 0 | 0 | 0 | 0 | .063 |
| 130 | +3.5 | 0 | 0 | 0 | 0 | .059 |
| Apex | 0 | 0 | 0 | 0 | 0 | 0 |

**Table 3.8-6a**
**CONTAINMENT STRUCTURE LOADING COMBINATIONS - LOAD NUMBERS 1 THROUGH 48**

| Load Combinations | Load No. | DL | VP | $OT_W$ | $OT_S$ | IP P=60 | $AT_{60}$ | $AT_{90}$ | E a=0.1g |
|---|---|---|---|---|---|---|---|---|---|
| Normal Operation | 1 | 1.0 | 1.0 | 1.0 | | | | | |
| (MODES 1 and 2) | 2 | 1.0 | 1.17 | 1.0 | | | | | |
| | 3 | 1.0 | 1.0 | | 1.0 | | | | |
| | 4 | 1.0 | 1.17 | | 1.0 | | | | |
| | 5 | 1.0 | 1.0 | 1.0 | | | | | 2.0 |
| | 6 | 1.0 | 1.17 | 1.0 | | | | | 2.0 |
| | 7 | 1.0 | 1.0 | | 1.0 | | | | 2.0 |
| | 8 | 1.0 | 1.17 | | 1.0 | | | | 2.0 |
| | 9 | 1.0 | 1.0 | 1.0 | | | | | -2.0 |
| | 10 | 1.0 | 1.17 | 1.0 | | | | | -2.0 |
| | 11 | 1.0 | 1.0 | | 1.0 | | | | -2.0 |
| | 12 | 1.0 | 1.17 | | 1.0 | | | | -2.0 |
| Test | 13 | 1.0 | 1.0 | 1.0 | | 1.15 | | | |
| | 14 | 1.0 | 1.17 | 1.0 | | 1.15 | | | |
| | 15 | 1.0 | 1.0 | | 1.0 | 1.15 | | | |
| | 16 | 1.0 | 1.17 | | 1.0 | 1.15 | | | |
| Accident Pressure Condition "d" | 17 | 1.0 | 1.0 | 1.0 | | 1.0 | 1.0 | | |
| | 18 | 1.0 | 1.17 | 1.0 | | 1.0 | | | |
| | 19 | 1.0 | 1.0 | | 1.0 | 1.0 | 1.0 | | |
| | 20 | 1.0 | 1.17 | | 1.0 | 1.0 | 1.0 | | |
| | 21 | 1.0 | 1.0 | 1.0 | | 1.0 | 1.0 | | 0.8 |
| | 22 | 1.0 | 1.17 | 1.0 | | 1.0 | 1.0 | | 0.8 |
| | 23 | 1.0 | 1.0 | | 1.0 | 1.0 | 1.0 | | 0.8 |
| | 24 | 1.0 | 1.17 | | 1.0 | 1.0 | 1.0 | | 0.8 |
| | 25 | 1.0 | 1.0 | 1.0 | | 1.0 | 1.0 | | -0.8 |

| Load Combinations | Load No. | DL | VP | $OT_W$ | $OT_S$ | IP P=60 | $AT_{60}$ | $AT_{90}$ | E a=0.1g |
|---|---|---|---|---|---|---|---|---|---|
| | 26 | 1.0 | 1.17 | 1.0 | | 1.0 | 1.0 | | -0.8 |
| | 27 | 1.0 | 1.0 | | 1.0 | 1.0 | 1.0 | | -0.8 |
| | 28 | 1.0 | 1.17 | | 1.0 | 1.0 | 1.0 | | -0.8 |
| Condition "a" | 29 | 1.0 | 1.0 | 1.0 | | 1.5 | | 1.0 | |
| | 30 | 1.0 | 1.17 | 1.0 | | 1.5 | | 1.0 | |
| | 31 | 1.0 | 1.0 | | 1.0 | 1.5 | | 1.0 | |
| | 32 | 1.0 | 1.17 | | 1.0 | 1.5 | | 1.0 | |
| Condition "b" | 33 | 1.0 | 1.0 | 1.0 | | 1.25 | | 1.0 | 1.0 |
| | 34 | 1.0 | 1.17 | 1.0 | | 1.25 | | 1.0 | 1.0 |
| | 35 | 1.0 | 1.0 | | 1.0 | 1.25 | | 1.0 | 1.0 |
| | 36 | 1.0 | 1.17 | | 1.0 | 1.25 | | 1.0 | 1.0 |
| | 37 | 1.0 | 1.0 | 1.0 | | 1.25 | | 1.0 | -1.0 |
| | 38 | 1.0 | 1.17 | 1.0 | | 1.25 | | 1.0 | -1.0 |
| | 39 | 1.0 | 1.0 | | 1.0 | 1.25 | | 1.0 | -1.0 |
| | 40 | 1.0 | 1.17 | | 1.0 | 1.25 | | 1.0 | -1.0 |
| Condition "c" | 41 | 1.0 | 1.0 | 1.0 | | 1.0 | 1.0 | | 2.0 |
| | 42 | 1.0 | 1.17 | 1.0 | | 1.0 | 1.0 | | 2.0 |
| | 43 | 1.0 | 1.0 | | 1.0 | 1.0 | 1.0 | | 2.0 |
| | 44 | 1.0 | 1.17 | | 1.0 | 1.0 | 1.0 | | 2.0 |
| | 45 | 1.0 | 1.0 | 1.0 | | 1.0 | 1.0 | | -2.0 |
| | 46 | 1.0 | 1.17 | 1.0 | | 1.0 | 1.0 | | -2.0 |
| | 47 | 1.0 | 1.0 | | 1.0 | 1.0 | 1.0 | | -2.0 |
| | 48 | 1.0 | 1.17 | | 1.0 | 1.0 | 1.0 | | -2.0 |

## Table 3.8-6b
## CONTAINMENT STRUCTURE LOADING COMBINATIONS - KEY TO SYMBOLS

**KEY**

| **Loading Number Fundamental Load** | **Symbol** | **Meaning** |
|---|---|---|
| No. 1 | DL | Dead Load |
| No. 2 | VP | Vertical Prestress |
| No. 3 | $OT_W$ | Operating Temperature - Winter |
| No. 4 | $OT_S$ | Operating Temperature - Summer |
| No. 5 | IP | Internal Pressure<br>(P=60 psig) |
| No. 6 | $AT_{60}$ | Accident Pressure + Temperature<br>(P=60 psig; T = 286 °F) |
| No. 7 | $AT_{90}$ | Accident Pressure + Temperature<br>(P=90 psig; T =312 °F) |
| No. 8 | E | Design Earthquake<br>(horizontal acceleration 0.10g) |

## Table 3.8-7
## CONCRETE COVER REQUIRED FOR REINFORCING STEEL

| | | Minimum Cover | |
|---|---|---|---|
| **Location** | **Type of Steel** | **Actual** | **ACI 318** |
| Dome | Principal (18S) | 11-1/2 in | 2-1/4 in. |
| | Crack control | 2 in | 1-1/2 in. |
| Cylinder | Hoop (18S) | 2-3/8 in | 2-1/4 in |
| | Vertical (14S & other) | 4-5/8 in | 1-3/4 & 1 1/2-in |
| Base ring | Bottom reinforcing | 3 in. | 3 in. |
| | Top reinforcing | 1-1/2 in | 1-1/2 in. |
| Base slab | Bottom reinforcing | 3 in. | 3 in. |
| | Top reinforcing | 1-1/2 in. | 1-1/2 in. |

## Table 3.8-8
## ELASTOMER PADS PROPERTIES

| | |
|---|---|
| **Original Physical Properties** | |
| Tear resistance, ASTM D625 $D_6$C C, psi of thickness, minimum 180 | 180 |
| Hardness, ASTM D676, points | 55 ± 3 |
| Tensile strength, ASTM D412, minimum psi | 2500 |
| Ultimate elongation, minimum % | 400 |
| **Change in Physical Properties (Oven Aging 70 hr at 212 °F in accordance with ASTM D573)** | |
| Hardness, points change | 0 to +15 |
| Tensile strength, % change | ±15 |
| Ultimate elongation, maximum % | -40 |
| **Extreme Temperature Characteristics** | |
| Compression set under constant deflection, (22 hr at 158 °F) ASTM D395 (Method B), maximum, % | 25 |
| Low temperature brittleness, ASTM D745, no breaks above | -20 °F |
| **Ozone Cracking Resistance** | |
| Exposure to 100 parts per 100 million of ozone in air by volume at a strain of 20% and a temperature of 100 °F ± 2° in a test otherwise conforming to ASTM D1149. (Samples shall be solvent-wiped before test to remove any traces of surface impurities). Time within which no cracks develop, minimum hours | 100 |
| **Oil Sell, ASTM Oil No. 3** | |
| 70 Hours at 212 °F, ASTM D471, volume change, maximum, % | +80 |
| Shear modulus, psi | 138 ± 10% |

## Table 3.8-9
## ROCK ANCHOR A - UPLIFT TEST WITH JACKING FRAME, MAY 19, 1966

| | | Pier Dials | | | | Rock Surface Pegs | | |
|---|---|---|---|---|---|---|---|---|
| Time | Load Kips | NE Corner (in.) | SW Corner (in.) | Head Dial (in.) | Average Deformation Top of Pier (in.) | North (in.) | Intermedite (in.) | South (in.) |
| 0840 | 0 | .300 | 0 | .700 | 0 | 7-1/4 | 7-5/8 | 9-3/4 |
| 0955 | 20 | .304 | .005 | .705 | .0045 | | | |
| 1010 | 40 | .308 | .009 | .709 | .0085 | | | |
| 1025 | 60 | .311 | .012 | .714 | .0115 | | | |
| 1040 | 80 | .318 | .019 | .723 | .0185 | | | |
| 1055 | 100 | .354 | .031 | .752 | .0425 | 7-1/4 | 7-9/16 | 9-5/8 |
| | | | | | LIFT OFF APPARENT | | | |
| 1105 | 110 | .380 | .039 | .767 | .0595 | | | |
| | 80 | .349 | .025 | 739 | .037 | | | |
| | 60 | .334 | .016 | .724 | .025 | | | |
| | 40 | .326 | 010 | .715 | .018 | | | |
| | 20 | .318 | .003 | .706 | .0105 | | | |
| | 0 | .312 | -.002 | .699 | .005 | 7-1/4 | 7-9/16 | 9-5/8 |

# Table 3.8-10
# DESIGN CODE COMPARISON

**(Summary of Code Changes with the Potential to Significantly Degrade Perceived Margin of Safety)**

**(AISC 1963 Versus AISC 1980)**

| Referenced Subsection AISC 1980 | Referenced Subsection AISC 1963 | Structural Elements Potentially Affected | Comments |
|---|---|---|---|
| 1.5.1.2.2 | --- | Beam end connection where the top flange is coped and subject to shear, or failure by shear along a plane through fasteners or by a combination of shear along a plane through fasteners plus tension along a perpendicular plane. | See case study 1 for details. |
| 1.9.1.2 and Appendix C | 1.9.1 | Slender compression unstiffened elements subject to axial compression or compression due to bending when actual width-to-thickness ratio exceeds the values specified in subsection 1.9.1.2. | New provisions added in the 1980 Code, Appendix C. See case study 10 for details. |
| 1.10.6 | 1.10.6 | Hybrid girder - reduction in flange stress. | New requirements added in the 1980 Code Hybrid girders were not covered in the 1963 Code. See case study 9 for details. |
| 1.11.4 | 1.11.4 | Shear connectors in composite beams. | New requirements added in the 1980 Code regarding the distribution of shear connectors. (Equation 1.11-7). The diameter and spacing of the shear connectors are also subject to new controls. |
| 1.11.5 | --- | Composite beams or girders with formed steel deck. | New requirement added in the 1980 Code. |
| 1.14.2.2 | --- | Axially loaded tension members where the load is transmitted by bolts or rivets through some but not all of the cross-sectional elements of the members. | New requirement added in the 1980 Code. |
| 1.15.5.2, 1.15.5.3, 1.15.5.4 | --- | Restrained members when flange or moment connection plates for end connections of beams and girders are welded to the flange of I or H shaped columns. | New requirement added in the 1980 Code. |
| 2.9 | 2.8 | Lateral bracing of members to resist lateral and torsional displacement. | Scale<br>A 0.0 $M/M_p < 1.0$;<br>C 0.0 $M/M_p > -1.0$ |

**(Summary of Code Changes with the Potential to Significantly Degrade Perceived Margin of Safety)**

**(AISC 1963 Versus AISC 1980)**

| Referenced Subsection | | | |
|---|---|---|---|
| **AISC 1980** | **AISC 1963** | **Structural Elements Potentially Affected** | **Comments** |
| | | | See case study 7 for details. |

# Table 3.8-11
# ACI 318-63 VERSUS ACI 349-76 CODE COMPARISONS

| Reference Subsection | | | |
|---|---|---|---|
| ACI 349-76 | ACI 318-63 | Structural Elements Potentially Affected | Comments |
| 7.10.3 | 805 | Columns designed for stress reversals with variation of stress from $f_y$ in compression to 1/2 $f_y$ in tension | Splices of the main reinforcement in such columns must be reasonably limited to provide for adequate ductility under all loading conditions. |
| 11.13 | --- | Short brackets and corbels which are primary load-carrying members | As this provision is new, any existing corbels or brackets may not meet these criteria and failure of such elements could be nonductile type failure. Structural integrity may be seriously endangered if the design fails to fulfill these requirements. |
| 11.15 | --- | Applies to any elements loaded in shear where it is inappropriate to consider shear as a measure of diagonal tension and the loading could induce direct shear type cracks | Structural integrity may be seriously endangered if the design fails to fulfill these requirements. |
| 11.16 | --- | All structural walls - those which are primary load carrying, e.g., shear walls and those which serve to provide protection from impacts of missile-type objects | Guidelines for these kinds of wall loads were not provided by older codes; therefore, structural integrity may be seriously endangered if the design fails to fulfill these requirements. |
| Appendix A | --- | All elements subject to time-dependent and position-dependent temperature variations and restrained so that thermal strains will result in thermal stresses | For structures subject to effects of pipe break, especially jet impingement, thermal stresses may be significant. Scale A for areas of jet impingement or where the conditions could develop causing concrete temperature to exceed limitation of A.4.2. |
| Appendix B | --- | All steel embedments used to transmit loads from attachments into the reinforced-concrete structure | New appendix; therefore, considerable review of older designs is warranted. Since stress analysis associated with these conditions is highly dependent on definition of failure planes and allowable stress for these special conditions, past practice varied with designers' opinions. Stresses may vary significantly from those thought to exist under previous design procedures. |

| Reference Subsection: ACI 349-76 | ACI 318-63 | Structural Elements Potentially Affected | Comments |
|---|---|---|---|
| Appendix C | --- | All elements whose failure under impulsive and impactive loads must be precluded | New appendix; therefore, consideration and review of older designs is considered important. Since stress analysis associated with these conditions is highly dependent on definition of failure planes and allowable stress for these special conditions, past practice varied with designers' opinions. Stresses may vary significantly from those thought to exist under previous design procedures. |

## Table 3.8-12
## ACI 301-63 VERSUS ACI 301-72 (REVISED 1975) COMPARISON

No significant changes were found in the ACI 301 Code comparison.

## Table 3.8-13
## ACI 318-63 VERSUS ASME B&PV CODE, SECTION III, DIVISION 2, 1980 CODE COMPARISON

| Referenced Subsection | | | |
|---|---|---|---|
| **Sec. III 1980** | **ACI 318-63** | **Structural Elements Potentially Affected** | **Comments** |
| CC-3421.5 | --- | Containment and other elements transmitting in-plane shear. | New concept. There is no comparable section in ACI 318-63, i.e. no specific section addressing in-plane shear. The general concept used here (that the concrete, under certain condition, can resist some shear, and the remainder must be carried by reinforcement) is the same as in ACI 318-63. Concepts of in-plane shear and shear friction were not addressed in the old codes and therefore, a check of the old designs could show some significant decrease in overall prediction of structural integrity. |
| CC-3421.6 | 1707 | Regions subject to peripheral shear in the region of concentrated forces normal to the shell surface. | These equations reduce to $V_c = 4\sqrt{f'_c}$ when membrane stresses are zero, which compares to ACI 318-63 (Sections 1707 (c) and (d)) which address “punching” shear in slabs and footings with the $\phi$ factor taken care of in the basic shear equation (Section CC-3521.2.1, Equation 10). |

# Table 3.8-14
# ASME B&PV CODE, SECTION III, DIVISION 2, 1980 (ACI 359-80) VERSUS ACI 318-63 CODE COMPARISION

| Sec. III 1980 | AIC 318-63 | Structural Elements Potentially Affected | Comments |
|---|---|---|---|
| CC-3421.6 | | | Previous code logic did not address the problem of punching shear as related to diagonal tension, but control was on the average uniform shear stress on a critical section. See case study 13 for details. |
| CC-3421.7 | 921 | Regions subject to torsion. | New defined limit on shear stress due to pure torsion. The equation relates shear stress from a biaxial stress condition (plane stress) to the resulting principal tensile stress and sets the principal tensile stress equal to $6\sqrt{f'_c}$. Previous code superimposed only torsion and transverse shear stresses. |
| CC-3421.8 | --- | Bracket and corbels. | New provisions. No comparable section in ACI 318-63; therefore, any existing corbels or brackets may not meet these criteria, and failure of such elements could be nonductile type failure. Structural integrity may be seriously endangered if the design fails to fulfill these requirements. |
| CC-3440(b),(c) | --- | All concrete elements which could possibly be exposed to short-term high thermal loading. | New limitations are imposed on short-term thermal loading. No comparable provisions existed in the ACI 318-63. |
| CC-3532.1.2 | --- | Where biaxial tension exists. | ACI 318-63 did not consider the problem of development length in biaxial tension fields. |

## Table 3.8-15
## LIST OF STRUCTURAL ELEMENTS TO BE EXAMINED

| Structural Elements To Be Examined | Code Changes Affecting These Elements | |
|---|---|---|
| | New Code | Old Code |
| Beams | AISC 1980 | AISC 1963 |
| Composite Beams | | |
| 1. Shear connectors in composite beams | 1.11.4 | 1.11.4 |
| 2. Composite beams or girders with formed steel deck | 1.11.5 | ---[a] |
| Hybrid Girders | | |
| Stress in flange | 1.10.6 | 1.10.6 |
| Compression Elements | AISC 1980 | AISC 1963 |
| With width-to-thickness ratio higher than specified in 1.9.1.2 | 1.9.1.2 and Appendix C | 1.9.1 |
| Tension Members | AISC 1980 | AISC 1963 |
| When load is transmitted by bolts or rivets | 1.14.2.2 | --- |
| Connections | AISC 1980 | AISC 1963 |
| Beam ends with top flange coped, if subject to shear | 1.5.1.2.2 | --- |
| Connections carrying moment or restrained member connection | 1.15.5.2, 1.15.5.3, 1.15.5.4 | --- |
| Members designed to operate in an inelastic regime | AISC 1980 | AISC 1963 |
| Spacing of lateral bracing | 2.9 | 2.8 |
| Short brackets and corbels having a shear span-to-depth ratio of unity or less | ACI 349-76, 11.13 | ACI 318-63 |
| Shear walls used as a primary load-carrying member | ACI 349-76, 11.16 | ACI 318-63 |
| Precast concrete structural elements, where shear is not a measure of diagonal tension | ACI 349-76, 11.15 | ACI 318-63 |
| Concrete regions subject to high temperatures | ACI 349-76 | ACI 318-63 |
| Time-dependent and position-dependent temperature variations | Appendix A | --- |
| Columns with spliced reinforcement subject to stress reversals; $f_y$ in compression to 1/2 $f_y$ in tension | ACI 349-76, 7.10.3 | ACI 318-63, 805 |
| Steel embedments used to transmit load to concrete | ACI 349-76, Appendix B | ACI 318-63 |

| Structural Elements To Be Examined | Code Changes Affecting These Elements | |
|---|---|---|
| | New Code | Old Code |
| Elements subject to impulsive and impactive loads whose failure must be precluded | ACI 349-76, Appendix C | ACI 318-63 |
| Containment and other elements, transmitting in-plane shear | B&PV Code Section III, Division 2, 1980, CC-3421.5 | ACI 318-63 |
| Region of shell carrying concentrated forces normal to the shell surface (See case study 13 for details) | B&PV Code, Section III, Division 2, 1980, CC-3421.6 | ACI 318-63, 1707 |
| Region of shell under torsion | B&PV Code Section III, Division 2, 1980, CC-3421.7 | ACI 318-63, 921 |
| Elements subject to short-term high temperature loading | B&PV Code Section III, Division 2, 1980, CC-3440(b), (c) | ACI 318-63 |
| Elements subject to biaxial tension | B&PV Code, Section III Division 2, 1980, CC-3532.1.2 | ACI 318-63 |
| Brackets and corbels | B&PV Code, Section III, Division 2, 1980, CC-3421.8 | ACI 318-63 |
| Roof[b] | --- | --- |
| Extreme environmental snow loads are provided by SEP Topic II-2.A. Regulatory Guide 1.102 (Position 3) provides guidance to preclude adverse consequences from ponding on parapet roofs. Failure of roofs not designed for such circumstances could generate impulsive loadings and water damage, possibly extending to Seismic Category I components of all floor levels. | | |

a. Dash (---) indicates that no provisions were provided in the older code.

b. Not shown in tabular summary of code comparisons.

## Table 3.8-16
## MASSES, MOMENT OF INERTIA (I), FLEXURAL AREA (A), AND SHEAR AREA ($A_s$) FOR THE LLNL MODEL

| Node | Element | Mass, lb-sec$^2$/in. | I, in. (x $10^9$) | A, in. (x $10^4$) | $A_s$, in. (x $10^4$) |
|---|---|---|---|---|---|
| 13 | | 2480.4 | | | |
| | 12 | | 5.202 | 12.15 | 6.074 |
| 12 | | 4952.8 | | | |
| | 11 | | 15.35 | 12.17 | 6.086 |
| 11 | | 4952.8 | | | |
| | 10 | | 21.80 | 12.08 | 6.038 |
| 10 | | 7007.2 | | | |
| | 9 | | 40.09 | 19.03 | 9.516 |
| 9 | | 6491.06 | | | |
| | 8 | | 36.44 | 17.18 | 8.590 |
| 8 | | 5972.0 | | | |
| | 7 | | 36.44 | 17.18 | 8.590 |
| 7 | | 5972.0 | | | |
| | 6 | | 36.44 | 17.18 | 8.590 |
| 6 | | 5972.0 | | | |
| | 5 | | 36.44 | 17.18 | 8.590 |
| 5 | | 5972.0 | | | |
| | 4 | | 36.44 | 17.18 | 8.590 |
| 4 | | 5972.0 | | | |
| | 3 | | 36.44 | 17.18 | 8.590 |
| 3 | | 5972.0 | | | |
| | 2 | | 36.44 | 17.18 | 8.590 |
| 2 | | 5972.0 | | | |
| | 1 | | 36.44 | 17.18 | 8.590 |
| 1 | | 5972.0 | | | |

**Table 3.8-17**
**MODAL FREQUENCIES FOR THE LAWRENCE LIVERMORE NATIONAL LABORATORY CONTAINMENT SHELL MODEL**

| Mode | Frequency |
|---|---|
| 1 | 6.97 |
| 2 | 18.87 |
| 3 | 21.47 |
| 4 | 37.75 |
| 5 | 53.91 |
| 6 | 54.60 |
| 7 | 70.23 |
| 8 | 80.89 |
| 9 | 84.70 |
| 10 | 92.38 |

**Table 3.8-18**
**RESPONSE VALUES FOR REGULATORY GUIDE 1.60 HORIZONTAL (0.17g) AND VERTICAL (0.11g) SPECTRA INPUT**

| | Horizontal | | Vertical |
|---|---|---|---|
| Element | Moment (lb-in. x $10^9$) | Shear (lb x $10^6$) | Axial (lb x $10^6$) |
| 12 | 0.102 | 0.60 | 0.204 |
| 11 | 0.391 | 1.70 | 0.603 |
| 10 | 0.842 | 2.68 | 0.985 |
| 9 | 1.41 | 3.89 | 1.50 |
| 8 | 2.12 | 4.90 | 1.94 |
| 7 | 2.95 | 5.71 | 2.32 |
| 6 | 3.88 | 6.40 | 2.65 |
| 5 | 4.90 | 6.97 | 2.94 |
| 4 | 5.97 | 7.42 | 3.18 |
| 3 | 7.09 | 7.76 | 3.37 |
| 2 | 8.24 | 7.98 | 3.48 |
| 1 | 9.42 | 8.08 | 3.55 |

**Table 3.8-19**
**PEAK HARMONIC AMPLITUDES OF THE SEISMIC LOAD ON CYLINDER AND DOME OF THE CONTAINMENT SHELL**

| Elevation[a] (in.) | Load Amplitude (psi) |
|---|---|
| 0 | 0 |
| 73 | 0.334 |
| 219 | 0.736 |
| 365 | 1.138 |
| 511 | 1.508 |
| 657 | 1.908 |
| 803 | 2.310 |
| 949 | 2.712 |
| 1095 | 5.310 |
| 1188 | |
| **$\phi$ (rad)** | **Load Amplitude (psi)** |
| 1.57 | 3.944 |
| 1.80 | 2.074 |
| 2.20 | 2.907 |
| 2.62 | 4.602 |
| 3.14 | |

a. Elevation measured from mid-surface of base slab.

**Table 3.8-20**
**MATERIAL PROPERTIES FOR STEEL, CONCRETE, AND FOAM INSULATION**

| | Steel Liner | Concrete | Insulation | Reinforcement Steel |
|---|---|---|---|---|
| Young's modulus (psi) | $29 \times 10^6$ | $4.3 \times 10^6$ | --- | $29 \times 10^6$ |
| Poisson's ratio | 0.3 | 0.25 | --- | --- |
| Coefficient of thermal expansion of (in./in.-°F) | $6.3 \times 10^{-6}$ | $5.5 \times 10^{-6}$ | --- | --- |
| Density ($lb/ft^3$) | 490 | 150 | 4 | --- |
| Coefficient of thermal conductivity, Btu/hr ft, °F | 26 | 0.44 | 0.022 | --- |
| Specific heat Btu/lbm °F | 0.11 | 0.160 | 0.30 | --- |
| Thickness (in.) | 0.375 | 43.30 | 1.25 | |
| $\sigma_Y$ (psi) Steel and $f'_c$ (psi) Concrete | 32,000 | 5,000 | --- | 40,000 |

## Table 3.8-21
## MAXIMUM DISPLACEMENTS OF 5/8-INCH S6L STUDS IN THE INSULATION TERMINATION REGION

| Stud Capacity Qu (kips) (1) | Buckled Panel Stress (ksi) (2) | Maximum Stud Displacement Δ (in.) (3) | Ultimate Stud Displacement (in.) (4) | Max/Ultimate Displacement % (5) |
|---|---|---|---|---|
| 10.6 | 26 | 0.141 | 0.167 | 84 |
| 8.3 | 26 | 0.148 | 0.167 | 89 |
| 10.6 | 29 | 0.159 | 0.167 | 95 |
| 8.3 | 29 | 0.166 | 0.167 | 99 |

## Table 3.8-22
## MAXIMUM DISPLACEMENT OF STUDS IN GENERAL DOME

| | | | | | | Membrane liner Strains (in./in.) | | | |
|---|---|---|---|---|---|---|---|---|---|
| Stud Capacity Qu (kips) | Stress Limit in Unbuckled Panels (ksi) | Maximum Stud Displacement Δ (in.) | Ultimate Stud Displacement (in.) | Max/Ultimate Displacement % | Liner Lateral Displacement (in.) | Membrane Compression | Membrane and Bending Compression | Membrane and Bending Tension | Column (8)/ $\in_y$ |
| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) | (10) |
| | | | | | | | | | |
| *5/8-In. Diameter S6L Studs at 24 In.* | | | | | | | | | |
| 10.6 | 26 | 0.113 | 0.167 | 68 | 1.67 | 0.0096 | 0.0558 | 0.0366 | 35 |
| 8.3 | 26 | 0.150 | 0.167 | 90 | 1.92 | 0.0097 | 0.0626 | 0.0433 | 39 |
| 10.6 | 29 | 0.170 | 0.167 | 102 | 2.03 | 0.0088 | 0.0597 | 0.0422 | 37 |
| 8.3 | 29 | >0.300 | 0.167 | >>100 | NA | NA | NA | NA | NA |
| | | | | | | | | | |
| *3/4-In. Diameter Headed Studs at 51 In.* | | | | | | | | | |
| 31.1 | 5.8 | 0.00343 | 0.341 | 1 | 0.42 | 0.000177 | 0.000767 | 0.000413 | 0.5 |
| 31.1 | 12 | 0.0388 | 0.341 | 11 | 1.41 | 0.00020 | 0.0024 | 0.0019 | 1.5 |
| $\in_y$ = 48/30000 = 0.0016 in./in. | | | | | | | | | |

## Table 3.8-23
## LOAD DEFINITIONS

| | |
|---|---|
| D | Dead loads or their related internal moments and forces (such as permanent equipment loads). |
| E or $E_O$ | Loads generated by the operating-basis earthquake. |
| E′ or $E_{ss}$ | Loads generated by the safe shutdown earthquake. F |
| | Loads resulting from the application of prestress. |
| H | Hydrostatic loads under operating conditions. |
| $H_a$ | Hydrostatic loads generated under accident conditions, such as post-accident internal flooding. ($F_L$ is sometimes used to designate the post-LOCA internal flooding). |
| L | Live loads or their related internal moments and forces (such as movable equipment loads). |
| $P_a$ | Pressure load generated by accident conditions (such as those generated by the postulated pipe break accident). |
| $P_o$ or $P_v$ | Loads resulting from pressure due to normal operating conditions. |
| $P_s$ | All pressure loads which are caused by the actuation of safety relief valve discharge including pool swell and subsequent hydrodynamic loads. |
| Rs or $R_r$ | Pipe reactions under accident conditions (such as those generated by thermal transients associated with an accident). |
| $R_o$ | Pipe reactions during startup, normal operating, or shutdown conditions, based on the critical transient or steady-state condition. |
| $R_a$ | All pipe reaction loads which are generated by the discharge of safety relief valves. |
| $T_a$ | Thermal loads under accident conditions (such as those generated by a postulated pipe break accident). |
| $T_o$ | Thermal effects and loads during startup, normal operating, or shutdown conditions, based on the most critical transient or steady-state condition. |
| $T_s$ | All thermal loads which are generated by the discharge of safety relief valves. |
| W | Loads generated by the design wind specified for the plant. |
| W′ or $W_t$ | Loads generated by the design tornado specified for the plant. Tornado loads include loads due to tornado wind pressure, tornado-created differential pressure, and tornado-generated missiles. |
| $Y_j$ | Equivalent static load on the structure generated by the impingement of the fluid jet from the broken pipe during the design-basis accident. |
| $Y_m$ | Missile impact equivalent static load on the structure generated by or during the design-basis accident, such as pipe whipping. |
| $Y_r$ | Equivalent static load on the structure generated by the reaction on the broken pipe during the design-basis accident. |