3.8 DESIGN OF CATEGORY I STRUCTURES

This section presents information on the design of seismic Category I structures. The seismic Category I structures consist of the following:

a) Reactor Building (comprising a free standing steel containment vessel, a containment internal structure and a reinforced concrete Shield Building).

b) Reactor Auxiliary Building

c) Fuel Handling Building

d) Component Cooling Water System Structure

All seismic Category I structures are housed in a common structure called the Nuclear Plant Island Structure, which is a rectangular box-like reinforced concrete structure 380 ft. long, 267 ft. wide and extending 64.5 ft. below grade.

The seismic Category I structures are discussed in Subsection 3.8.1 through 3.8.4 and shown on Figure 3.8-1. The Nuclear Plant Island Structure is supported on a common reinforced concrete mat foundation which in turn lies on top of Pleistocene clay. The concrete mat foundation is discussed in Subsection 3.8.5.

3.8.1 CONCRETE CONTAINMENT

The Containment System does not utilize a concrete containment. The primary containment is a free standing steel pressure vessel which is surrounded by a reinforced concrete Shield Building. The Shield Building is designed as a seismic Category I structure and is discussed under Subsection 3.8.4. The steel containment and the Reactor Building internal structure are described in Subsection 3.8.2 and 3.8.3, respectively.

3.8.2 STEEL CONTAINMENT

3.8.2.1 Description of the Containment

The containment vessel, including all its penetrations, is a low leakage steel shell which is designed to withstand the postulated loss of coolant accident and to confine the postulated release of radioactive material. Systems directly associated with the containment vessel are the Containment Spray System, the Containment Cooling System and the Containment Isolation System. Plans and sectional views are shown on Figures 1.2-17, 18, 19, 20, 21, 22, 24, and 3.8-2.

The containment vessel is a cylindrical steel pressure vessel with hemispherical dome and ellipsoidal bottom which houses the reactor pressure vessel, the reactor coolant piping, the pressurizer, the quench tank, the reactor coolant pumps, the steam generators, and the safety injection tanks. It is completely enclosed by the reinforced concrete Shield Building. An annular space is provided between the walls and domes of the containment vessel and the concrete Shield Building to permit construction operations and in-service inspection. The containment vessel is an independent free standing structure with a net free volume of approximately 2,680,000 cubic ft., rigidly fixed at its base near the elevation of its bottom spring line. The containment vessel is supported on a concrete base that is placed after the cylindrical shell and the ellipsoidal bottom have been constructed and post weld heat treated. Both the Shield Building and the containment vessel is supported on a common foundation mat. With the exception of the concrete placed underneath and near the knuckles at the sides of the vessel, there is no structural ties between the containment vessel and the Shield Building above the foundation slab. Therefore, there is virtually unlimited freedom for differential movement between the containment vessel and the Shield Building above the top of the concrete base at elevation - 1.50 ft. MSL. Concrete floor fill as described in Subsection 3.8.3.1 is placed above the ellipsoidal shell bottom after the vessel has been post weld heat treated, to anchor the vessel.

The cylindrical portion of the steel containment shell has a minimum thickness of 1.903 in. on an inside radius of 70 ft. The polar crane girder support plates are welded to the shell liner at approximately six ft. on center. Except for some miscellaneous platform framing and some minor seismic restraints, no major floor framing or seismic restraint supports are attached to the shell liner. Immediately below the crane girder, a heating and ventilating duct approximately 6'-6" wide x 8'-O" deep, running the entire containment circumference, is structurally supported at 45 places and attached to the shell by means of welded clips. The containment shell was also used to support temporary construction loads from the pedestal cranes. The 1.903 in. minimum shell plate thickness increases to a minimum of four in. adjacent to all penetrations and openings. The inside radius of the hemispherical dome is 70'-15/32 in. with a dome plate of 0.95 in. thick connected to the cylindrical portion of the shell at the tangent line by means of a full penetration weld. The containment spray piping is attached to the dome by means of welded clips as is the dome inspection walkway and platforms. The containment vessel is protected from external missiles by the Shield Building. Protection from internal missiles is provided by the primary and secondary shield walls and other containment internal structures.

The function of the containment piping penetration assemblies is to provide for passage of process, service, sampling and/or instrumentation pipe lines (or in the case of the fuel transfer assembly, new or spent fuel) into the reactor containment vessel, while maintaining the desired containment integrity and providing a leak-tight seal with adequate provisions for movement between the pipe lines (or fuel transfer tube) and the containment structure during operation (startup, shutdown, power, testing, emergency and accident condition).

All containment penetrations have the following design characteristics in order to maintain the desired containment integrity:

a) penetrations are capable of withstanding the maximum internal pressure which would occur due to the postulated rupture of any pipe inside the containment vessel,

b) penetrations are capable of withstanding the jet forces associated with the flow from a postulated rupture of the pipe in the penetration or adjacent to it, while still maintaining the integrity of the containment, and

c) penetrations are capable of safely accommodating all thermal and mechanical stresses which may be encountered during all modes of operation and test.

The materials used for penetrations, including the personnel access air locks, the equipment access hatch, the piping and duct penetration sleeves land the electrical penetration sleeves conform with the requirements set forth by the ASME Boiler* and Pressure Vessel Code. In accordance with this code, the penetration materials meet the necessary nil ductility transition temperature impact values as specified in Subsection 3.8.2.6.

All piping penetrations shown and tabulated on Figure 3.8-3 penetrate the Shield Building as well as the containment vessel. The containment vessel is provided with capped spare penetrations for possible future requirements.

These penetrations are divided into six general classes:

a) Type I - "Hot" piping (penetrations which accommodate thermal movement).

b) Type II - General piping (penetrations which are subject to only relatively small thermal movement or stress).

c) Type III - Moderate temperature and pressure (refer to Figure 3.8-3) piping

d) Types IV and V - piping penetrations in the concrete mat structure

e) Type VI - Fuel transfer tube penetration

Type I piping penetrations convey fluids at elevated temperature and pressure (main steam and feedwater). Because of the differential movement of the various structures due to seismic loading and the thermal expansion between the process line anchor (at the flued head) and the containment vessel, bellows expansion joints are required. The bellows expansion joints provide the required flexibility in such installations without imposing undue stresses on the containment wall or in the piping system.

The penetration unit consists of a multiple fluid head to which are welded a length of process pipe, guard pipe, primary and secondary bellows expansion joints. The expansion joint serves two general functions:

a) provides for the passage of process pipes into the containment vessel while maintaining a leak-tight seal, and

b) absorbs relative movements between the containment vessel, Shield Building and/or process piping.

The guard pipe protects the bellows element against direct steam impingement in case of process line rupture. To permit leakage surveillance, a special multiply-ply bellows element is used, so constructed as to permit a pressure test.

A bellows, located between the shield wall and the flued head, seals the penetration where it passes through the concrete shield wall, permitting the annulus between the containment and Shield Building to be maintained at a negative pressure. The end of the guard pipe is provided with an impingement ring to protect the primary expansion bellows from jet forces due to an internal pipe rupture.

In the case of a process line rupture or pipe whip, the guard pipe will protect the rest of the penetration assembly and other penetrations in the near proximity from steam impingement, jet forces, or deflection while directing the released fluids into the containment. The guard pipe shall be designed to withstand the various pressures, temperatures and loads associated with the intended service.

The trunion on the multiple flued head serves as an anchor against forces and moments generated by piping configuration within the containment vessel. The trunion is supported by the Reactor Auxiliary Building structure.

Type II penetrations are provided for pipelines carrying low temperature and low pressure fluids, where failure of the process pipe would not over pressurize the annulus between the Shield Building and the Containment vessel.

The principal consideration in this design is the provision of a leaktight seal between the pipe and the containment vessel. This is accomplished by use of sleeves welded into the steel containment vessel by the vessel fabricator. The process line is welded directly to a sleeve penetrating the containment vessel. The sleeve and containment shell are designed to carry the forces and moments due to normal operating conditions and due to a pipe rupture.

In addition to the basic Type II penetrations there are the following modified Type II penetrations:

a) multiple lines - penetrations whose design allows for the passage into the containment of multiple instrument lines through one penetration assembly,

b) penetration whose design allows for the passage of a process line through different Shield Building and containment vessel elevations, and

c) penetrations whose design consist of a valve located in the annulus, as well as inside and outside the containment vessel, which allows any bypass leakage through the penetration to be directed to the annulus, and accommodates for any expansion or contraction in the process line portion of the penetration with an expansion joint.

Type III penetrations (Figure 3.8-3) are provided for moderate pressure and temperature piping, where there is a possibility of a pipe rupture overpressurizing the annulus and/or associated impingement effects and where relatively large thermal movements are involved, separate penetration nozzles are used. In this design, a multiple flued head is used to provide a leaktight seal between the penetration nozzle, the Shield Building sleeve and the process pipe, and at the same time, minimize fatigue problems that could arise due to thermal expansion of the process line and thermal transients. In case of a rupture of the process pipe in the area of the annulus, the penetration acts to direct the fluid back into the containment vessel, thus preventing overpressurization of the annulus. The penetration designed to accommodate all forces and moments due to both thermal expansion and pipe rupture. Modified Type III penetrations penetrate the containment vessel bottom head.

Type IV and V penetrations, shown in Figure 3.8-4, provide for moderate pressure and temperature piping in the containment concrete mat structure. The penetrations are designed to eliminate the possibility of a process line rupture from emptying the containment of recirculation water during and after a loss-of-coolant accident (LOCA). The penetration assembly consists of a process pipe guided through an oversized guard pipe and welded to the capped ends of the guard pipe with an expansion joint in the portion of the guard pipe outside of the containment. The design will accommodate any differential expansion of the various equipment and structures involved.

Type VI penetration, shown in Figure 3.8-5, is the fuel transfer tube which is provided to transport fuel assemblies between the refueling cavity and the spent fuel pool during refueling operations of the reactor. The penetration assembly consists of a smaller pipe within a segmented larger one. The inner pipe acts as the transfer tube and is fitted with a double gasketed blind flange at the refueling cavity end (Reactor Building) and a standard gate valve at the spent fuel pool end (Fuel Handling Building). This arrangement prevents leakage through the transfer tube in the event of an accident. The outer pipe provides a leaktight seal between the transfer tube and the containment penetration nozzle. Bellows expansion joints are provided with the outer pipe to compensate for any differential movement between the two pipes and/or the various structures.

Electrical penetrations are manufactured and tested in accordance with the intent of IEEE 317-1972, Electrical Penetration Assemblies in Containment Structures for Nuclear Fueled Power Generating Stations. Compliance with Regulatory Guide 1.63 is discussed in Subsection 8.3.1.

Cartridge type penetrations are used for all electrical conductors passing through the steel containment vessel and the Shield Building. The penetration cartridges are hollow cylinders through which the conductors pass. Each cartridge is provided with a pressure connection to allow pressurization for testing and cartridges containing power cables have thermocouples for temperature monitoring. Figure 3.8-6 shows typical electrical penetrations. The cartridges are installed in penetration sleeves welded into the wall of the containment vessel and cast in the concrete Shield Building. Sealing between the cartridges and the sleeves is accomplished by welding. The cartridge headers through which the cables pass are hermetically sealed. All materials used in the design are selected for resistance to all possible environmental conditions.

Each cartridge is sealed and tested at the factory for leakage. The only seals that have to be made in the field are the welds mounting the cartridges into the sleeves. Where necessary, the outer sides of the penetration headers are protected against contamination by accumulations of dirt and moisture.

Sufficient cable slack is provided in the annulus to allow for differential expansion and relative movement during an earthquake between the containment vessel and the Shield Building. Cable protection sleeves are provided in the annulus at each penetration.

A 14 ft. diameter equipment hatch is provided for equipment access. This is a welded steel assembly, with a double gasketed flanged and bolted cover. Provision is made to pressurize the space between the double gaskets to 44 psig.

Two personnel air locks are provided. These are welded steel assemblies. Each lock has two double gasketed doors in series. Provision is made to pressurize the space between the gaskets. The doors are mechanically interlocked to ensure that one door cannot be opened until the second door is sealed. Provision is made for deliberately overriding the interlock by the use of special tools and procedures under strict administrative control. Each door is equipped with quick acting valves for equalizing the pressure across the doors. The doors are not operable unless the pressure is equalized. Pressure equalization is possible from every point at which the associated door can be operated. The valves for the two doors are properly interlocked so that only one valve can be opened at one time, and only when the opposite door is closed and sealed. Each door is designed so that with the other door open, it can withstand and seal against design and testing pressures of the containment vessel. There is visual indication outside each door showing whether the opposite door is open or closed and whether its valve is open or closed. In addition, limit switches are provided to indicate remotely whether doors are open or closed. Provisions are made outside each door for remotely closing and latching the opposite door so that in the event that one door is accidentally left open it can be closed by remote control. The air locks have nozzles installed which permit pressure testing of the lock at any time. Gaskets are made of materials resistant to radiation.

A communications system and an interior lighting system are installed. These systems are capable of operating from the emergency power supply as discussed in Subsections 9.5.2 and 9.5.3, respectively.

3.8.2.2 Applicable Codes, Standards and Specifications

The following codes, standards and specifications were used in the design, fabrication, erection and testing of the containment vessel.

a) American Society of Mechanical Engineers (ASME) - 1971 Edition

   1) Boiler and Pressure Vessel Code, Section II, "Material Specifications"

   2) Boiler and Pressure Vessel Code, Section III, "Nuclear Vessels"

   3) Boiler and Pressure sure Vessel Code, Section VIII, "Unfired Pressure Vessels"

   4) Boiler and Pressure Vessel Code, Section IX, "Welding Qualifications"

b) American Society of Testing and Material (ASTM) - 1971

   1) ASTM A36 - Structural Steel

c) American Institute of Steel Construction (AISC)

   1) Specification for the Design, Fabrication and Erection of Structural Steel for Buildings - July 1, 1970 edition

d) American National Standards Institute (ANSI)

   1) N5.12 - 1974 - Protective Coatings (Paints) for Light Water Nuclear Industry

   2) N101.2 - 1971 - Protective Coatings (Paints) for Light Water Nuclear Reactor Containment Facilities

   3) N45.4 - 1972 - Leakage Rate Testing of Containment Structure for Nuclear Reactors

   4) N45.2.9 - 1974 - "Requirements for Collection, Storage and Maintenance of Quality Assurance Records for Nuclear Power Plants."

   5) N45.2.12 - 1973 - "Requirements for Auditing of Quality Assurance Programs for Nuclear Power Plants."

e) Steel Structures Painting Council - 1963

   1) SSPC-SP-10 Near White Blast Cleaning

   2) SSPC-PA-1 Shop, Field and Maintenance Painting

f) Ebasco Services Incorporated

   1) Specification Ebasco LOU-1564-717 "Steel Containment Vessel" October 1970.

   2) Specification Ebasco 73-65, 873-73 and 873-75 "Nondestructive Testing Procedures"

   3) Ebasco Quality Assurance Manual for Nuclear Power Stations - 6/1/70

g) Federal Register - 1972

   1) 10 CFR Part 50 "Licensing of Production and Utilization Facilities" - Appendix J "Primary Reactor Containment Leakage Testing for Watercooled Power Reactors."

h) NRC Regulatory Guides

   1) Regulatory Guide 1.57 (June 1973), "Design Limits and Loading Combinations for Metal Primary Reactors Containment System Components"

The containment vessel was designed, fabricated, erected, and tested in accordance with the requirements of Section III, Subsection NE of the ASME Code for Class "MC" Components, 1971 Edition, up to and including Summer 1971 Addenda, and Code cases 1431, 1454-1 and 1517 as approved by Regulatory Guides 1.84 and 1.85. The design, fabrication and erection of supports and bracing and similar structures not within the scope of the ASME Code conform to the requirements of the AISC specifications, except that the welding, welding procedures and welders qualifications are in accordance with the ASME Code Section IX.

The design, fabrication, erection and testing of the containment vessel shall also conform to the ASME Boiler and Pressure Vessel Code, Section II "Material Specifications," and Section VIII "Unfired Pressure Vessels."

The containment vessel is code stamped in accordance with Paragraph NE-8000 of Section III of the ASME Boiler and Pressure Vessel Code.

The containment vessel metal materials allowable stresses are in compliance with those specified in the ASME Code Section III Appendix I for the design temperatures which are in effect for the loading conditions. All of the material used for fabricating the containment vessel complies with Article NE-2000 of Subsection NE of the ASME Code, Section III, Division I. The impact testing, as a minimum requirement, is as specified in Section III of the ASME code, Paragraph NE-2321 or NE-2322. Charpy V-notch specimens (SA-370 - Type A) have been used for all impact testing. Weld test plates have been made and impact tested in accordance with Subsection NE of Section III of the ASME Code. Production test plates have been impact tested in accordance with the ASME Code, Section III, Paragraph MB-2300.

All main butt welded seams (Categories A and B of ASME Section III) have been 100 percent radiographically inspected as per ASME Section VIII.

Corner welds and fillet attachments have been inspected as per ASME Section III, Subsection NE for Nuclear Containment Vessels.

The containment vessel has been pressure tested in accordance with the rules of ASME Boiler and Pressure Vessel Code, Section VIII, UG-100 and Section III Subsection NE.

In addition to the ASME Codes, the testing of the pressure vessel meets the requirements of ANSI N45.4 and 10CFR50 Appendix J. The design requirements in addition to the ASME Code, meet the requirements of Ebasco Specification LOU 1564.717. Quality control procedures for the containment vessel meet the requirements of ANSI N45.2.9 and ANSI 45.2.12 as well as the Ebasco Quality Assurance Manual for Nuclear Power Stations.

3 8.2.3 Loads and Load Combinations

The vessel was designed to exhibit a general elastic behavior under accident and earthquake conditions of loading. No permanent deformations due to primary stresses have been permitted in the design under any condition of loading. The design of the Containment vessel was based on permissible stresses as set forth in the applicable codes. The structure will safely function within the normal design limits as specified in Section III of the ASME Boiler and Pressure Vessel Code Article NE-3000 "Design" and Regulatory Guide 1.57 (June 1973), Design Limits and Loading Combinations for Metal Primary Reactor Containment System Components. A fatigue evaluation will not be performed for the containment vessel design.

Circumferential compressive stresses in the vicinity of the bottom knuckle resulting from internal pressures were held below the critical buckling stresses with appropriate margins of safety (refer to Subsections 3.8.2.4 and 3.8.2.5).

The possibility of differential motion between the containment vessel base and adjacent interior and exterior concrete was considered. Provisions are made to insure that relative motion of these elements do not occur.

The areas of the vessel adjacent to penetrations that are not subjected to externally applied loads were designed by the area replacement method in accordance with NE-3331 of Section III of the ASME Code.

Jet impingement forces were calculated on the basis of velocity attenuation as a function of distance between the point of pipe rupture and the object being impinged upon. The velocity attenuation was based on Abramovich's theory of turbulent jets.(1) The containment vessel shell was designed to withstand the forces determined from these calculations.

Where external loads and moments were applied to penetrations the secondary and local stresses were treated in accordance with the Welding Research Council Bulletin 107[2] and the basic stress intensity limits shall be in accordance with ASME Code, Section III, as noted in Table 3.8-1. Where substantial thermal or mechanical loads other than pressure loads exist, analyses were performed using the Yale computer program developed by Professor A Kalnins.[3] The program was used to analyze the following areas:

a) discontinuity stresses at embedment,

b) head to shell discontinuities,

c) discontinuities around the crane girder, and

d) areas where the membrane stress spectrum is disturbed due to change in geometry or loading conditions.

A containment vessel of thickness suitable to meet the specified internal pressure requirements is capable of withstanding an external pressure differential of 0.65 psig in accordance with UG-28 of Section VIII of the ASME Boiler and Pressure Vessel Code. Since the ASME Code charts have a safety factor of four, the collapsing pressure for the containment vessel will be about four times greater than the design external pressure differential. The containment vessel is vented as required to eliminate pressure fluctuations caused by air temperature changes during various operating modes. This is accomplished through ventilation purge connections which are normally closed while the reactor is in operation.

Automatic vacuum relief devices are used to prevent the containment vessel from exceeding the external design pressure in accordance with the requirements of Article 16, Section III of the ASME Boiler and Pressure Vessel Code. The Automatic Vacuum Relief System consists of two redundant 24 in. penetrations connecting the annulus to the containment. Each system includes one 24 in. butterfly valve with pneumatic operator and one 24 in. check valve located on the containment side of the penetration in series. Each butterfly valve is actuated by a separate pressure controller which senses the differential pressure between the containment and the annulus. Each butterfly valve is provided with an air accumulator of minimum capacity to allow the valve to open at least two times after failure of instrument air. The check valve is set to open when the pressure of the upstream (annulus) side of the valve is 1.1 in. WG above the pressure of the downstream (containment) side of the valve.

→(EC-706, R304; EC-18219, R304)

The butterfly valve will actuate automatically. It is set to open before containment pressure decreases 10 in. WG below annulus pressure. The valve can only be manually closed after containment pressure increases above the butterfly valve actuation setpoint. The combined pressure drop at rated flow through the two valves in either line will not exceed the design external pressure differential of 0.65 psig with any prevailing atmospheric pressure.

←(EC-706, R304; EC-18219, R304)

Pressures, temperatures, design loads and the corresponding nomenclature for the design of the containment vessel and penetrations are listed below:

a)	Test overpressure	49.5 psig
b)	Design Internal Pressure (Pgl) Temperature (Tgl)	39.6 psig @ 263 F (as and defined by 1971 Edition of ASME III)
c)	Leakage Rate Test Pressure	44 psig
d)	Containment pressure (PcL) and Temperature (Tcl) after loss of coolant accident	44 psig @ 263 F
e)	Maximum external to internal pressure differential and temperature after cooling of the containment by the containment spray coolant system and actuation of the vacuum breaker system.	0.65 psi @ 120 F
f)	Process line maximum operating pressure (Po) and temperature (To)	See Figure 3.8-3
g)	Penetration guard pipe (for process line), ambient pressure (Pg) and temperature (Tg)	14.7 psia @ 120 F
h)	Penetration guard pipe (for process line), internal pressure (Pgl) and temperature (Tgl) after (LOCA)	39.6 psig and 263 F
i)	Process line design pressure (Pd) and temperature (Td)	See Figure 3.8-3
j)	Penetration guard pipe internal pressure (Pgr) and temperature (Tgr) after rupture of the process line inside the penetration, (Pgr), (Tgr) equal (Po), (To) respectively.	See Figure 3.8-3
k)	Normal external to internal pressure differential and containment ambient temperature.	0.15 psi @ 120 F
l)	Containment ambient pressure (Pc) and ambient temperature (Tc)	14.7 psia @ 120 F
m)	Normal annulus pressure (Pa) and temperature (Ta)	14.5 psia @ 120 F

n) Annulus pressure (Pal) and temperature (Tal) after loss of coolant accident (LOCA)

When t is 5 min., (Pal) is 13.6 psia, and (Tal) is 120 F

When t is 60 min., (Pal) is 14.6 psia, and (Tal) is 180 F

o) Stress (W) in penetration due to weight of pipe — 1.5 ksi

p) Jet impingement (J) with 0.15 psi pressure and 120 F temperature — Determined on a case basis

The containment vessel was designed for loads in accordance with the ASME Code, including those specified herein.

a) Live Loads:

1) the live load on the equipment hatch opening,

2) the weight of contained air during tests,

3) crane loads,

4) loads at penetrations,

5) floor load of 150 psf in the personnel locks,

6) floor load on dome walkway of 50 psf,

7) load on access ladder of 500 lbs,

8) the shield structure concrete dome temporary construction loads, and

9) scaffolding or other temporary construction loads.

b) Dead Loads:

The dead load from the ventilation ducts, crane dead loads and dead loads of vessel and appurtenances.

c) Seismic Loads:

1) The operating basis earthquake seismic loads are lateral forces equal to the seismic coefficients shown on attached Figure 3.8-7, multiplied by the

permanent gravity loads applied to the vessel and a vertical upward or downward force equal to 7.5 percent of the permanent gravity loads, assumed to be acting simultaneously with each other.

2) The safe shutdown earthquake seismic loads are lateral forces equal to the seismic coefficients shown on attached Figure 3.8-8, multiplied by the permanent gravity loads applied to the vessel and a vertical force upward or downward equal to 15.0 percent of the permanent gravity loads, assumed to be acting simultaneously with each other.

3) Stresses due to seismic loads are combined with stresses caused by the maximum hypothetical accident, dead loads and appropriate indicated loads to obtain the total stresses.

4) The seismic loads include the seismic effects due to the inertia of the mass of the air locks and equipment hatches and to the effects of the air locks vibrating as independent systems.

d) Wind Loads:

The portion of the containment vessel which was exposed above grade prior to the completion of the shield structure was designed for the wind loads on the projected area of the circular shape in accordance with the height zones below:

Height Above Grade Feet	Wind Load lb/ft^2
0-30	30
30-49	40
50-99	50
Above 100	60

Tabulated wind pressures include the reduction for the circular shape of the vessel.

The load combinations that were considered in the design of the containment vessel were in compliance with Article NE-3000 of the ASME Code for Class "MC" Components Section III, Division I and have included the following:

Case 1 - Construction at post weld heat treatment

Case 2 - Acceptance test at ambient temperature and wind load

Case 3 - Preoperation test at ambient temperature

Case 4 - Normal operating condition at temperature range of 30 F to 150 F

Case 5 - Cold shutdown at temperature range of 30 F to 120 F

Case 6 - Accident condition with operating basis earthquake seismic loads

Case 7 - Accident condition with safe shutdown earthquake seismic loads

Case 8 - Accident condition with operating basis earthquake seismic loads and thermal plus pipe rupture loads

Case 9 - Accident condition with safe shutdown earthquake seismic loads on piping plus pipe rupture loads

Case 10 - Temporary roof construction loads with safe shutdown earthquake seismic loads.

Table 3.8-2 shows the load combinations for each of the ten cases. Table 3.8-3 shows the load combinations used to determine stresses at the junction of the containment vessel knuckle region and column.

3.8.2.4 Design and Analysis Procedures

The analysis of the steel containment consists of two parts: the overall analysis of the containment and the local analyses. The local analyses include such items as the air lock and penetrations.

3.8.2.4.1 Shell Analysis

Stresses in the vessel shell remote from penetrations or other appurtenances are analyzed as described below. Shell stresses adjacent to appurtenances are analyzed along with the appurtenance design.

Stresses resulting from each specified load condition are calculated separately at critical locations and combined to obtain total meridional and circumferential stresses at each point. Stress intensities are then determined and compared to specified allowable stresses.

In accordance with the maximum shear stress failure criterion and thin shell theory, stress intensities are found as follows:

a) since shear stress is much less than circumferential or meridional stress, shear stresses are neglected in calculating stress intensities, and

b) since radial stress is much less than circumferential or meridional stress, for calculating stress intensities $\delta r = 0$.

For latitudinal stress $\delta\theta$ and meridional stress $\delta\phi$ of like sign (refer to Figure 3.8-9); the stress intensity is the larger of $|\ (\delta\theta) - \delta r = |\ (\delta\theta)\ |$ or $|\ (\delta\phi - \delta r\ | = |\ (\delta\phi)\ |$. For $\delta\theta$ and $\delta\phi$ of unlike signs, the stress intensity is equal to $|\ (\delta\theta) - (\delta\phi)\ |$.

In addition to the stress intensity evaluation, compressive buckling loads are investigated in the construction, normal operating, and accident conditions. An axial load in the longitudinal direction will induce a meridional membrane stress $\delta\phi$ (local buckling) in the cylindrical portion of the containment vessel shell. In the cylindrical portion of the vessel:

2Π r $N\phi$ + F = 0 (from equilibrium)

where:

F	= axial load	
t	= shell thickness	
r	= shell radius	
$N\phi$	= meridional force per unit length	$= \dfrac{-F}{2\pi\ r}$
$\sigma\phi$	= meridional stress (longitudinal)	$= \dfrac{N\phi}{t} = \dfrac{-F}{2\pi\ rt}$

From Timoshenko's <u>Theory of Plates and Shells,</u>(4)

where:

$$\frac{N\phi}{r_1} + \frac{N\Theta}{r_2} = -Z$$

where:

r_1	=	radius in longitudinal direction
r_2	=	radius in latitudinal direction
Z	=	radial load = 0 for a cylinder with $r_1 = \infty$ and $r_2 = r$
$N\theta$	=	latitudinal (hoop force) per unit length

therore: $N\theta$ = 0.

In the spherical portion (dome), an axial load in the longitudinal direction will induce a meridional buckling stress ($\sigma\phi$) and a latitudinal buckling stress ($\sigma\theta$). In the spherical portion:

$2\pi r_o\ N\phi\ \mathrm{Sin}\phi + F = 0$ (for equilibrium(4))

where:

r_o	=	radius at point of interest
ϕ	=	refer to Figure 3.8-10.

r_o = r sinϕ $\qquad N\phi = \dfrac{-F}{2\ \ r_o\ \mathrm{Sin}\phi}$

$\sigma\phi = \dfrac{N\phi}{t} \qquad \sigma\phi = \dfrac{-F}{2\pi\ \ r\,t\ \mathrm{Sin}^2\phi}$

$$\frac{N\phi}{r_1} + \frac{N\Theta}{r_2} = -Z$$

Z = 0 for a sphere $r_1 = r_2$

$$\frac{N\Theta}{r} = \frac{N\phi}{r} \text{ and } N\Theta = -N\phi$$

Figure 3.8-10 shows the equation used for the total vertical axial load to be considered at any location.

An axial load in the latitudinal (circumferential) direction will induce a meridional (load buckling) stress in the cylindrical portion of the containment vessel:

$$\sigma\phi = \frac{Mr}{I} \frac{Mr}{\pi r^3 t} = \frac{M}{\pi r^2 t}$$

$$N\phi = \frac{M}{\pi r^2}$$

where:

M	=	moment at point of interest
I	=	moment of inertia of cylindrical shell at point of interest
r	=	shell radius
t	=	shell thickness

Since N ϕr_1 + N /r_2 = - Z and Z is equal to zero when $r_1 = \infty$ and $r_2 = r$, then $N\theta = 0$.

In the spherical portion (dome), an axial latitudinal load will induce a longitudinal buckling stress ($\sigma\phi$) as well as a latitudinal buckling stress ($\sigma\theta$):

$$N\phi_{max} \operatorname{Sin}\phi\, t = \frac{Mr_o}{I} = \frac{Mr_o}{3}$$

πr_o

(assuming a stress distribution in accordance with Beam Flexure Theory)

$$\sigma\phi = \frac{N\phi}{t} = \frac{Mr_o}{\pi r^3{}_o t \sin\phi} = \frac{M}{\pi r^2 t \sin^3\phi}$$

where:

r_o	=	radius at point of interest
r	=	radius of dome
ϕ	=	see Figure 3.8-11

Since $N\phi/r_1$ + N /r_2 = - Z and Z is equal to zero when $r_1 = r_2$,
then $\sigma\theta = -\sigma\phi$.

Figure 3.8-11 shows the equations used for the total horizontal axial load to be considered at any location.

The ellipsoidal bottom head is analyzed for loading conditions which produce buckling. The loading conditions investigated are specified in Table 3.8-3. The analysis and design of tile knuckle region of the bottom head (refer to Figures 3.8-12, 13 and 14) during construction is performed using conventional methods as well as CB&I's computer programs 781 and 405. The computer model and results of the analyses are shown on Figure 3.8-15 and presented in Table 3.8-17 and 18.

CB&I's program 781 is employed to analyze the portion of the ellipsoidal bottom head in the region of the ethafoam zone around the outside of the containment. The computer model assumes the shell fixed against deflection at the bottom of the embedment zone. The ethafoam 220 is treated as a series of springs on the outside of the steel shell. The spring rate is based on published data from Dow Company. The concrete inside the steel shell is not accounted for in the computer model. The shell model extends to a point in the cylindrical shell where a membrane state may be assumed to exist. At this point, boundary conditions approximating membrane forces due to pressure, dead load and seismic shear forces are applied. Seismic shear forces are applied in a sine series expansion. Load cases both with and without the specified thermal gradients are considered. Three thermal gradients (times of 20 min, 50 min, and 2 hr) are selected as representative for analyzing this loading condition. The analyses consider only the largest seismic load. Table 3.8-4 shows the containment vessel temperature gradient used in the analysis.

3.8.2.4.2 Air Lock Seismic Analysis

The containment vessel design for earthquake includes the seismic effects of the air locks vibrating as an independent system. The seismic effect of this independent vibration is then added vectorially to all other seismic effects.

In the analysis, the vibration driving force on the air locks is determined by accelerations derived from the response spectra curves, shown in Subsection 3.7.2. The vibrating driving forces is considered to be independent of the vibration modes of the composite containment vessel shield building and foundation system.

For analytical purposes the locks are assumed to vibrate in three independent directions as shown on Figure 3.8-16. Case I will result in forces and moments being applied to the containment in the meridional plane. Case II will result in forces and moments being applied to the containment in the circumferential plane. Case III will result in a radial thrust being applied to the containment shell.

Once the natural frequency of the lock is calculated for the longitudinal, circumferential, and radial direction of the containment vessel, it is possible to determine the fundamental period and thus the response acceleration to be applied to the lock. The response acceleration is calculated for the insert to shell junction and then applied to the lock loads to find stresses in the shell.

For longitudinal and circumferential direction: (Case I & II)

$$K = \frac{M}{\Theta}$$

where: K = spring constant of shell

$$\omega = \left[\frac{K}{I}\right]0.5$$

ω = angular frequency of lock

$$\omega = \left[\frac{M}{\Theta I_o}\right]0.5$$

Io = mass moment of inertia of lock about point of support on shell

M = moment at shell

$$T = \frac{2\pi}{\omega} \text{ (sec)}$$

θ = unit rotation at shell to insert junction

T = fundamental period of lock

The spring constant for the longitudinal and circumferential direction is determined by applying a unit deflection (one radian) at the shell and determining M using Reference 5 for the β based on the proper parameters for the junction.

For the radial direction: (Case III)

$$K = \frac{P}{w}$$

Where: W = weight of lock plus insert

$$T = 2\pi \frac{W}{K_g}^{0.5}$$

g = 386.4 in/sec^2

P = load

w = unit deflection at shell to insert junction

Stresses in the shell due to the air locks vibrating as an independent system under horizontal and vertical earthquake have been determined by the use of Reference 2.

A horizontal earthquake acting perpendicular to the lock (Case I), will result in meridional shear and moment being applied to the shell. A horizontal earthquake acting perpendicular to the lock (Case II) results in a circumferential shear and moment being applied to the shell. An earthquake acting parallel to the lock (Case III) subjects the shell to a radial thrust.

Stresses are checked at three locations; at the neck to insert junction, at the insert to shell junction, and at 1/2 $[Rt]^{(0.5)}$ from any local stress area.

Stresses are calculated in the insert and in the shell. An equivalent stress intensity is calculated (per maximum shear theory) and compared to the ASME allowables.

3.8.2.4 3 Penetrations

The penetrations are analyzed for compliance with the ASME Code. Area replacement is calculated using Code methods. Welds for nozzles employing partial penetration attachment are analyzed using Code rules. Nozzles with specified loads are investigated for pipe wall stresses and for stresses in the vessel shell.

Inserts for penetrations larger than two in. pipe size are checked for area replacement in accordance with Paragraph NE-3331 of ASME Section III.

The pipe wall of nozzles with loads specified as thermal plus, seismic are analyzed for primary stresses. A description of the methods employed is discussed below.

Stresses in the vessel shell resulting from loads applied to penetrations are calculated using CB & I computer programs 1036M and 1027.

Loads are applied in specified combinations on a penetration of interest and on adjacent penetrations that are on cardinal lines of the central penetration within a distance of $2\sqrt{RT}$ where R is the radius of the shell and T is the thickness of the shell. This limit is chosen since the results of this type of analysis are questionable for greater distances. The combinations of specified loadings follow. Each load is considered reversible for purposes of determining maximum stress intensity.

Pressure produces a complex of stress in the shell and penetration at their intersection. As a rational means of estimating these stresses paragraph NB 3331 (b) of Section III, though not specifically applicable to Class MC vessels, has been used as a guide. This paragraph assumes that in the vicinity of a penetration reinforced in accordance with ASME rules, maximum membrane pressure stress will not exceed 1.0 Sm and the maximum surface stress will not exceed 1.5 Sm.

By inspection of the load combinations shown in Table 3.8-1, several observations were noted to reduce the number of loading cases by using similar combinations that produced more conservative values.

In the analysis of stress intensities in the combination insert plates, a comparison is made among the original loading combinations. For several of the loading combinations with the same allowable stress limits, similar pipe rupture loads, seismic loads, or thermal loads, and similar pressures, a typical load case is chosen as a conservative representative of the other similar combinations. In the analysis of the inserts, using CBI program 1036M, only the containment pressure (Pcl) and the internal pressure (Pgl) are significant. The process line pressures and guard pipe pressures are only considered in analyzing the penetration necks. By using Pcl (the most significant and the most conservative pressure) the number of different combinations was reduced. This number was further reduced by observing that in each group of similar loading combinations, the combination with the greatest number of of applied loads was the most conservative and was representative of all similar load combinations containing fewer applied loads (i.e. a load case that combines axial loading with moment was more conservative than a load combination that only included axial loading).

In analyzing stresses in the penetration necks due to external loads and initial shell stresses, comparisons again are made to reduce the number of significant load combinations. It is noted that by definition, design internal pressure is greater than guard pipe pressure, that containment pressure after loss of coolant accident (LOCA) is greater than containment ambient pressure, and that annulus pressure after LOCA is greater than normal annulus pressure. Also, the similarities in the given combinations and stress allowables make it possible to use load combinations with the largest value of pressure as a conservative case thus reducing the original number of combinations.

CB&I's 1036M program for interactions of penetrations assumes a round insert for analysis. According to Welding Research Council Bulletin 107, this is a conservative analysis.

3.8.2.4.4 Program 405

This is a CB&I program used for the analysis of a ring with a constant moment of inertia and modulus of elasticity. The loads are in the plane of the ring. The mathematics are based upon the Hardy-Cross Column Analysis for rings.[6] The loads can be moments, tangential, or radial to the ring. The printout is coefficients at incremental distances around the ring. The printout titles for the output are as follows:

X	=	angle and degrees as measured from a reference axes
V	=	a radial shear with force units acting in a radial direction through the ring
T	=	an axial thrust in the ring with units of force
M/R	=	a coefficient with units of force when multiplied by the radius to the centroid will equal a moment
EI/RR	=	a coefficient which when multiplied by the radius squared will equal the rotation of the ring at the point
REI/RRR	=	a coefficient when multiplied by the radius cubed equals the radial deflection of the point
CEI/RRR	=	a coefficient when multiplied by the radius cubed will equal the tangential deflection of the point

The following assumptions and limitations are considered:

a) ring has a uniform cross section and is made of homogeneous material,

b) depth of ring relative to its radius is too small to significantly influence the elementary flexural theory for straight beams,

c) unit stresses do not exceed proportional limit,

d) deflections from shear and axial stress are negligible,

e) deflections are so small, the basic geometry remains essentially unchanged,

f) when the ring is attached to a cylindrical shell, distortion of the ring with respect to the shell is so small the only significant reactions in the shell are membrane type shears, and

g) when the ring is attached to a cylindrical shell, the shell is held and loaded in such a way that membrane shear patterns are limited to the usual beam shear S=VCQ/I or torsional shear S_1 = Torsional moment/2 π R or any combination of the two (Vc = total shear perpendiculars to axis of a thin walled cylinder, Q = moment of area, I = moment of inertia and R = radius to centroid of ring).

3.8.2.4.5 Program 1027

This CB&I program determines the stress intensities in a sphere or cylinder at a maximum of 12 points around an externally loaded round or square attachment. Stresses resulting from external loads are superimposed on an initial pressure stress situation. The program computes stresses at three levels of plate thicknesses: outside, inside, and centerline of plate. The 12 points investigated are shown in Figure 3.8-17: four points at the edge of attachment, at $1/2\sqrt{RT}$ (where R is the radius of the shell and T is the thickness of the shell) from the edge of attachment and at the edge of reinforcement.

The program determines three components for each stress intensity:

σx = normal stress parallel to the vessels longitudinal axis

$\sigma\phi$ = normal stress in a circumferential direction

T = shear stress

The program has an option, whereby the penetration load will be considered reversible or nonreversible in direction. Under the reversible option, only the data associated with the most severe loading situation is printed.

Most of the analysis and notation used in the program is taken directly from the Welding Research Council Bulletin #107. Use of the program requires complete familiarity with this publication.

The program contains extrapolations of the curves for cylinders in Welding Research Council Bulletin # 107 for γ up to 570.

3.8.2.4.6 Program 781

The Shells of Revolution Program is the CB&I Program 781. The program calculates the stresses and displacements in thin walled elastic shells of revolution when subjected to static edge, surface and/or temperature loads with arbitrary distribution over the surface of the shell. The geometry of the shell must be symmetric, but the shape of the median is arbitrary. It is possible to include up to three branch shells with the main shell in a single model. In addition the shell wall may consist of four layers of different orthotropic materials, and the thickness of each layer and the elastic properties of each layer may vary along the median.

The 781 program numerically integrates the eight ordinary first order differential equations of thin shell theory derived by H. Reissner.(7) The equations are derived such that the eight variables are chosen which appear on the boundaries of the axially symmetric shell so that the entire problem can be expressed in these fundamental variables.

CB&I's program is an extensively revised Kalnins Program.(3) The program has been altered such that a four x four force-displacement relation can be used as a boundary condition as an alternative to the usual procedure of specifying forces or displacements. This force-displacement relation can be used to describe the forces at the boundary in terms of displacements at the boundary, or the displacements at the boundary in terms of forces or some compatible combination of the two. In this manner, it is possible to study the behavior of a large complex structure.

It is also possible to introduce a "spring matrix" at the end of any part of the stress model. This matrix must be expressed in the form, force' = spring matrix x displacement. In this manner it is possible to model the restraint of the sand cushion in the transition zone at the point of embedment. In addition to the above changes, the Kalnins Program has been modified to increase the size of the problem that can be considered and to improve the accuracy of the solution.

3.8.2.4.7 Program 1036M

This CB&I program determines the stress intensities in a "Jumbo" insert plate (a reinforcing plate with multiple penetrations) in a cylindrical vessel at eight points around one of these penetrations due to the loading on that penetration plus the loadings on the four adjacent penetrations all as superimposed on an initial stress situation. It does this at three levels of plate thickness: outside, inside, and centerline of plate. The eight points investigated are shown in Figure 3.8-18. The four points on radius R are at the junction of the penetration and the insert plate. The other four points are other points of interest, normally, they will be at the midpoints in the clear space between penetrations or at the edge of reinforcing. Although five penetrations are considered, each point is analyzed as though it were only influenced by two (the central penetration plus the penetration on the same axis as the point concerned).

The program also determines three components for each stress intensity:

σx = normal stress parallel to the vessels longitudinal axis.

$\sigma\phi$ = normal stress in a circumferential direction.

T = shear stress.

Each of these is composed of three subcomponents, one due to the central penetrations loading, one due the loading on the next adjacent penetration, and an initial stress component (input).

The program has an option whereby the penetration loads will be considered reversible or nonreversible in direction. Under the reversible option only the data associated with the most severe loading situations is printed out.

Most of the analysis and notation used in the program is taken directly from the Welding Research Council Bulletin (WRC) #107. Use of the program requires complete familiarity with this publication.

The analysis in WRC 107 is for a single penetration. This program analyzes the several penetrations individually, using WRC 107 techniques verbatim, and then through superposition obtains the composite results. The adjacent penetrations must be on a cardinal line of the central penetration in order to use WRC 107 methods. This has required a very conservative extension of the WRC 107 analysis. WRC 107 analysis applies only to the points on the penetration to shell juncture. This program makes stress determinations at points removed from the junction by fictitiously extending the radius of any penetration to any point at which a stress determination is desired. This disregards the statement in WRC 107 that "these stresses attenuate very rapidly at points removed from the penetration to shell juncture". Furthermore, in some cases, the moment induced stresses at both the juncture and at points removed from the juncture are increased by 20 percent per discussion in WRC 107. Table 3.8-5 shows the cases for the calculation of the parameters (per WRC 107) and stresses.

The program contains extrapolations of the curves in WRC 107 for T up to 600. The program is limited to the domains and range of Figures 1A through 4C in WRC 107 ($0 < \beta \leq 0.5$ and $5 \leq T \leq 600$).

3.8.2.4.8 Program 1392

Stresses in pipe are computed using CB&I's Program 1392 by considering the pipe as a beam. Pressure stresses are added in by superposition in the appropriate directions. Within the limits of reinforcement or next to the shell, the circumferential pressure stress is the membrane stress in the circumferential direction due to pressure in the vessel (PMSϕ)- Outside the limits of reinforcement or away from the shell, the circumferential pressure stress is the membrane stress in the circumferential direction due to pressure in the pipe (PMϕ). The stresses are computed for locations A,B,C and D (refer to Figure 3.8-19) and stress intensities are then computed as per WRC 107.

Referring to Figures 3.8-19 and 20:

$$\text{Sxat A} = \frac{P}{A} + \frac{ML}{S} + \frac{prm}{2t_n};\ \text{Sx at C} = \frac{P}{A} + \frac{Mc}{S} + \frac{prm}{2t_n}$$

$$\text{Sx at B} = \frac{P}{A} + \frac{ML}{S} + \frac{prm}{2t_n};\ \text{Sx at C} = \frac{P}{A} + \frac{Mc}{S} + \frac{prm}{2t_n}$$

$$S\phi \text{ at A, B, C, D} = \frac{\text{prm}}{\text{t}_\text{n}} \text{ if element is away from shell}$$

$$= \text{PMS}\phi \text{ if element is within limits of reinforcing}$$

$$T \text{ at A, B,} = \frac{\text{V}_\text{c}}{\pi\, \text{r}_\text{m} t_n} + \frac{\text{M}_{\text{Ty}}}{\text{J}}\text{; T at C, D} = \frac{\text{V}_\text{L}}{\pi\, \text{r}_\text{m} t_n} + \frac{\text{M}_{\text{Ty}}}{\text{J}}$$

Stress intensities are determined as follows:

a) Is Sx and Sϕ have like signs, the maximum of

$$\text{SI} = 1/2 \left[\text{S}\phi + \text{Sx} + \sqrt{(\text{S}\phi - \text{Sx})^2} + 4\text{T}^2 \right] \text{or} \sqrt{(\text{S}\phi - \text{Sx})^2} + 4\text{T}^2$$

b) If Sx and Sϕ have unlike signs,

$$SI = \sqrt{(\text{S}\phi - \text{Sx})}^2 + 4^2$$

c) If T = 0, the maximum of
SI = Sϕ, Sx or |Sϕ - Sx|

3.8.2.5 Structural Acceptance Criteria

3.8.2.5.1 Hemispherical Head and Cylindrical Shell

The load combinations for the containment are specified in Subsection 3.8.2.3 and summarized on Table 3.8-2 and 3. The allowable stresses for each of these load cases are summarized on Table 3.8-6. Tables 3.8-7 through 14 compare the calculated stresses with the allowable stresses for those critical sections of the containment shown in Figure 3.8-21. These tables represent load combinations one through seven and 10.

Membrane stresses in excess of the basic ASME code allowable are addressed in the 1971 Edition of Section III in Subparagraph NB 3213.10. This subparagraph has been interpreted to require no investigation or further evaluation of membrane stresses below 1.1 x basic code allowable (i.e., a stress level of 10 percent above basic allowable is acknowledged as acceptable).

Paragraph NE 3322 provides further information on this subject. Design by formula is permitted by this paragraph in the absence of substantial mechanical loads. Later (1974) code editions define substantial loads as those which cumulatively result in stresses that exceed 10 percent of the primary stresses induced by pressure (i.e., a design by formula could neglect loads producing 10 percent of the stress resulting from pressure).

In summary, primary membrane stress levels as high as 10 percent above basic allowable for the condition being evaluated, are acceptable and require no further justification.

Load combination cases eight and nine include an accident condition with seismic loads (OBE or SSE) and pipe rupture loads. As specified in Subsection 3.8.2.4, pipe rupture loads are investigated as local effects. The various load combinations and stress limits for each type of penetration are given in Table 3.8-1.

The allowable stresses were determined by the following methods:

a) Allowable Buckling Stresses for Unstiffened Hemispherical Head

Compressive stress resultants in the top head are compared to the allowable stresses obtained from the paragraphs entitled, "Biaxial Compression-Equal Unit Forces" and "Biaxial Compression-Unequal Unit Forces" of the Welding Research Council Bulletin #69[8]. Using these allowables for the spherical dome is based on the assumption that the dome acts as a cylinder with the radius equal to the radius of the dome.

Three cases are considered (refer to Figure 3.8-22):

1) For a uniaxial compressive stress resultant and for biaxial unequal tensile and compressive stress resultants:

$$N\phi \quad \text{allowable} = 1.8 \times 10^{6} \frac{t^{2}}{R}$$

where: t = shell thickness = 0.95 in.

R = dome radius = 840 in.

Therefore $N\phi$ allowable = 1934 lb/in and $\sigma\phi = \frac{N\phi}{t} = 2036$ psi.

2) For biaxial equal compressive stress resultants:

$N\phi$ allowable = $0.9 \times 10^{6} \frac{t^{2}}{R}$

$N\phi$ allowable = 967 lb/in.

$\sigma\phi = \sigma\theta = 1018$ psi

3) For biaxial unequal stress resultants: This case is treated as the summation of the uniaxial condition with equal stress.

$$\frac{N\Theta - N\phi}{1.8 \times 10^{6} \dfrac{t^{2}}{R}} + \frac{N\phi}{0.9 \times 10^{6} \dfrac{t^{2}}{R}} \leq 1$$

$N\theta + N\phi = 1934$ lb/in.

$\sigma\theta + \sigma\theta \leq 2036$ psi

b) Allowable Buckling Stresses for Cylindrical Vessel (refer to Figure 3.8.-23)

1) Meridional or Axial Stress

The maximum allowable compressive stress used in the design of cylindrical shell subjected to loadings that produce longitudinal compressive stress is in accordance with ASME Section VIII paragraph UG-23 (b). The maximum allowable compressive stress value is determined utilizing Figure UCS-28.2 of Section VIII. With a starting point of $L_1/100t = 4.41$, where L_1 and t are the inside radius of the cylindrical shell and thickness, respectively, the maximum allowable compressive stress value ($\delta\phi$) is equal to 4200 psi.

2) Circumferential Stress

Generally speaking, cicircumferential compression results from external pressure loading. The criteria of ASME Section VIII paragraph UG-28 is used to analyze circumferential buckling. These rules provide a safety factor of 4.0 against shell buckling.

From Figure UCS-28.2 of Section VIII, using L/Do and Do/t, the value of B is determined to be 1140. To compute the maximum allowable working pressure Pa, the following formula is used:

$$Pa = \frac{B}{Do/t} = 1.29 \text{ psi}$$

To compute the allowable circumferential stress, the following formula is used:

$$\sigma\Theta = \frac{PaR}{t} = 570 \text{ psi}$$

where:

R = shell radius

c) Allowable Buckling Stresses for Cylindrical Vessel during Post Weld Heat Treatment

Using E. 0. Bergman's formula(9) for the allowable buckling stress in a cylinder:

$\delta\Theta$ = longitudinal compressive stress $\frac{PD}{t}$

where:

$$P = \frac{E}{16}\left(\frac{t}{Le}\right)^2 = 3.7092$$

E = modulus of elasticity

Le = radius = 70 ft.

D = diameter = 140 ft.

t = shell thickness = 1.9 in.

Therefore $\sigma\theta$ = 3280 psi.

To calculate the allowable circumferential compressive stress:

$$\sigma\theta = S/2$$

where:

$$S = \frac{.3\ E\ t^{1.5}}{LD^{.5}}$$

L = vertical direction length (tangent line to tangent line)
= 135 ft.

t = thickness of cylindrical shell = 1.9 in.

D = diameter of cylindrical shell = 140 ft

Therefore S = 595 psi and $\sigma\theta$ = 300 psi.

3.8.2.5.2 Ellipsoidal Bottom Head

The allowable buckling stresses for an internal pressure on the ellipsoidal bottom head is determined as follows(8):

For $\frac{t}{R_1} \leq .0067$, the allowable buckling stress is: $\sigma\theta = \frac{E}{16}\frac{t}{R_1}$

where:

E = modulus of elasticity

t = head thickness

R_1= radius of curvature

For $.0067 < t/R_1 \leq .0175$, the allowable buckling stress is determined from Figure 3.8-24.

The allowable buckling stresses for an external pressure on the ellipsoidal bottom head is determined as follows:

For $\frac{t}{R_2} \leq .0067$, the allowable buckling stress is: $\sigma\phi = \frac{E}{16}\frac{t}{R_2}$

where:

R2 = cone radius

For $.0067 < \frac{t}{R_2} \leq .0175$, the allowable buckling stress is determined from Figure 3.8-24.

The allowable buckling stresses for the ellipsoidal head is given in Table 3.8-15.

The allowable buckling stresses for the ellipsoidal head during post weld heat treatment are calculated as follows:

For $\frac{t}{R} \leq .0067$, $\sigma = \frac{E_1}{16}\frac{t}{R}$

For $.0067 < \frac{t}{R} \leq .0175$, σ is equal to $\frac{E_1}{E_2}$ of the allowable stress values from Figure 3.8-24 with a maximum value of $\frac{E_1}{16}(.0067)$. E_1 is the modulus

of elasticity at 1200 F (11.6×10^6 psi) and E_2 is the modulus of elasticity at room temperature (29×10^6 psi).

The allowable buckling stresses for the ellipsoidal head during post weld heat treatment is given in Table 3.8-16. Tables 3.8-17 and 18 show the allowable shell stresses compared to the actual stresses in the knuckle region between columns and at columns, respectively. The columns are only used to support the vessel during construction. The allowable stress in the ethafoam zone of the ellipsoidal head is equal to 3 Sm, where Sm is the design stress intensity of the material (17500 psi). A summary of the stresses in this zone is shown in Figures 3.8-25, 26, 27 and 28.

3.8.2.5.3 Allowable Weld Stresses

All weld metal joining or attaching pressure parts will meet specified Charpy V notch impact test requirements.

a) ASME Allowable Weld Stresses

For full fusion welds, the allowable weld stresses are in accordance with Subsection NE of the ASME Code, Section III. The allowable is the same as the parent metal.

For partial depth groove welds, the allowable weld stress on the effective depth is calculated by multiplying an inspection factor by a load factor by Sm of weaker material. The inspection factor used is 0.8. The load factor used is 1.0 for loads perpendicular to the axis of the weld, 0.875 for any combination of perpendicular and parallel loads, and 0.75 for a load parallel to the axis of the weld. For simplicity, an allowable stress of 0.8 x 0.75 x Sm = 0.6 Sm is used for all partial groove welds except where a higher allowable is required and is permissible as above.

For fillet welds, the allowable stress is per UW-18(d) of ASME Section VIII. The allowable stress is equal to 0.55 Sm (of weaker material) on a minimum leg equal to 0.55/0.707 Sm on throat.

b) AISC Allowable Weld Stresses

The allowable stress for fillet welds and groove welds are in accordance with paragraph 1.5.3 of AISC.

3.8.2.6 Materials, Quality Control and Special Construction Techniques

In compliance with Article NE-2000 of Subsection NE of the ASME Code, Section III, Division I, the code materials used in the fabrication of the containment vessel are given in Table 3.8-19.

All of the carbon steel materials complied with the requirements of the applicable ASME Code material specification for low temperature service, except that the impact testing, as minimum requirement, was as specified in Section III of the ASME Code, Paragraph NE-2321 or NE- 2322.

Charpy V-notch specimens (SA-370 - Type A) were used for all impact testing at a maximum temperature of O°F.

Weld test plates are made and impact tested in accordance with Subsection NE of Section III of the ASME Code employing a test temperature of O°F and using the same material and thickness range as defined by the ASME Code.

Only those types of low hydrogen electrodes and combinations of wire and flux that produce welds that at least meet the impact values of the carbon steel parent material specified above are permitted in the construction. Certified test reports of actual tests for each heat and lot of welding materials are in accordance with NE- 2130, NE-2140, NB- 2420, and NB-2430, employing an impact test temperature of O°F maximum.

Production test plates are made and impact tested in accordance with the ASME Code, Section III Paragraph NB-2300, employing a test temperature of O°F. These tests plates are made in the following locations;

Cylinder - One set for each course on vertical seams

Dome - One set for each course on radial seam

Bottom - One set for 25 percent of knuckle radial seams

After fabrication all surfaces are cleaned to remove all oil, grease, dirt, loose rust, loose mill scale, and other foreign substances. The removal of oil and grease is accomplished before the mechanical cleaning, using mineral spirits or other paraffin-free solvents having a flash point higher than 1OO°F. Clean cloths and clean fluids are used to avoid leaving a film of greasy residue. The use of chipping tools which produce cuts, burrs, and other forms of excessive roughness are not permitted.

All carbon steel surfaces are blast cleaned prior to painting in accordance with Steel Structures Painting Council Specification SSPC-SP-10 "Near White Blast Cleaning." The interior and exterior surfaces are painted with a prime coat of Carbo-Zinc 11 capable of resisting the postweld heat treatment. A finish coat of Phenoline 305 is used for its resistance against the loss of coolant accident condition without flaking or peeling and for ease of decontamination.

All longitudinal and circumferential welds in the shell of the containment vessel are of the double bevel butt type. All joints in any accessories subject to the ASME Code and all the joints are in accordance with ASME Code, Section III Subsection NE. Welding details at nozzles are in accordance with ASME Code, Section III Subsection NE. All pressure boundary double butt welds are 100 percent radiographed. Magnetic particle or liquid penetrant tests are performed on the final layer of nonradiographable welds which are pressure retaining parts of the vessel. When radiography is required, joints are designed so that they could be radiographed without difficulty or the need for special techniques. All mandatory provisions of the ASME Code are followed. All welders, welder operators and welding procedures are qualified in strict accordance with and meet the requirements of Section IX of the ASME Code.

The procedures, design, methods and sequence of welding are reviewed prior to performance of welding.

Prior to welding, all protective coatings other than deoxaluminate or equivalent, are chemically or mechanically removed from all areas within two inches from a seam to be welded.

The submerged arc and gas metal arc welding processes are used for automatic welding.

The flux cored arc welding process is allowed; however, cored wire designed for operation without the use of externally supplied shielding gas (i.e., air wire) is not allowed. Such wire is also not used with arc shielding gas.

Welding procedure qualification test results demonstrate that the resulting weldments are compatible with the base materials and meet the minimum specified mechanical properties of the base material.

The low hydrogen type electrodes are used for manual shielded, metal arc welding. Welding procedure qualification test results demonstrate that the resulting weldments are compatible with the base materials and meet the minimum specified mechanical properties of the base materials.

Preheating at 200°F minimum is applied to all seams whose thickness exceeded one inch regardless of surrounding air temperature. For seams whose thickness is less than one inch and for stainless steel, preheating at 1OO°F is applied if the surrounding air temperature falls below 50°F.

Nondestructive testing, in addition to the containment pressure and leak rate tests, is performed after the post weld heat treatment operation by means of magnetic particle or liquid penetrant examinations on welds that join the containment penetrations and access openings to the containment wall.

All radiographic examinations on the welds are made prior to the post weld heat treatment operation in accordance with Section III, Subsection NE of the ASME Code, with controlled heating and cooling rates during post weld heat treatment which provides a safeguard against propagation of defects.

Repairs (if required to meet acceptance standards) are examined by the original nondestructive testing examination acceptance method without further post weld heat treatment in accordance with ASME Code Case 1454, provided the limitations of this case are met.

Repairs (if required to meet acceptance standards and if beyond the limitations of ASME Code Case 1454) are examined by the original nondestructive testing examination acceptance method prior to post weld heat treatment.

Post weld heat treatment is performed as required by and in accordance with the ASME Code. For field post weld heat treatment, after the vessel shell and ellipsodial bottom has been completely erected and welded, and the top temporarily closed with a diaphragm, it is externally insulated with a temporary blanket type insulation suitable for the post weld heat treatment operation, and is attached mainly by banding. Temporary supports, covers and insulation, required for effecting the post weld heat treatment operation are attached with a minimum of welding to the vessel.

Thermocouples of the iron-constantan type are used to monitor temperatures during the post weld heat treatment operation, and are so located as to indicate representative temperatures of areas of the vessel (refer to Figures 3.8-29 and 30). Thermocouples are used to monitor temperatures of the vessel shell and bottom head and to serve as control points on the top head (which does not require heat treatment because of lesser thickness) during the heat treatment cycle. The thermocouples are attached by welding to the outside surfaces of the vessel. The hot or measuring junctions of the thermocouples are protected by special sleeves which are welded to the part being monitored.

The heating of the vessel is done with luminous flame oil burners firing through openings in the bottom and sides of the vessel and arranged in such a way that the heat will be evenly distributed throughout the vessel during the heatup and holding periods without flame impingement on any part of the vessel. Combustion products are exhausted through an opening or openings in the top of the vessel.

Temperatures at the thermocouple locations are simultaneously recorded against time on a direct reading strip chart or charts using multiple point potentiometer type instruments.

Heatup rates, holding temperatures and times, cooldown rates and temperature gradient restrictions are in accordance with Section III, Subsection NB, of the ASME Code.

During heatup from ambient to holding temperature, the vessel will become approximately 15 in. larger in diameter. Special attention is given to the temporary peripheral supporting columns during the post weld heat treatment cycle. In order to prevent the development of excessive stresses at the columns-to-vessel connections and in the columns themselves, provisions are made for the bases of the columns to move radially outward during the heatup period and inward during cooldown.

Upon completion of the post weld heat treatment operation, the insulation and other temporary items are removed and temporary attachment weldments ground smooth.

Vessel tolerances are in accordance with ASME Section III Paragraph NE-1120 with the following exceptions:

a) the difference between the maximum and minimum diameter at any cross section did not exceed 0.5 percent of the nominal diameter of the vessel,

b) the diameter at any cross section did not deviate more than .25 percent of the nominal diameter, and

c) out of plumb did not exceed 0.25 percent measured between tangent lines after making allowances for out of roundness as specified above.

For penetration tolerances, after welding and post weld beat treatment, out of round tolerances are as follows:

a) Two in. diameter through 12 in. diameter: 0.125 in. difference in maximum and minimum diameter,

b) 14 in. diameter through 24 in. diameter: 0.250 in. difference in maximum and minimum diameter, and

c) over 24 in. diameter through 66 in. diameter: 0.375 in. difference in maximum and minimum diameter.

During the erection of the containment vessel, it is supported by 24 temporary steel pipe column assemblies welded directly to the vessel shell, the temporary supports are removed after the containment vessel is completely constructed and post weld heat treated and after a portion of permanent support concrete is placed. The supports are cut no closer than 1/4 in. from the surface of the shell plate and the remaining support material and welds are removed by chipping and grinding smooth with the shell face.

A placing and grouting procedure is used to fill void areas beneath the containment vessel. The placing and grouting procedure results in a continuous support of the vessel.

The Q/A program for the containment vessel, as stated above, is in accordance with NE-4000 and NE-5000 of the ASME Code, Section III, Div. 1.

3.8.2.7 Testing and Inservice Inspection Requirements

All testing is performed after concrete is placed under and within the containment vessel. The vessel is not pressurized until the ambient temperature is 30 F or above. A halogen sniffer leech test is performed for all bottom head welds that are embedded in concrete. Penetrations that will be eventually embedded in concrete will be provided with blockouts so that the concrete may be placed after the testing is performed.

All temporary blind flanges, blanking off plates and gasketing required to seal the containment vessel for testing purposes are removed following the successful performance of tests. Ends of pipe and cable sealing details are properly prepared for connections. All testing connections are properly sealed and are permanently left in place for future testing.

All tests and retests on the containment vessel as listed below are made upon completion of vessel erection and after placement of concrete.

a) Soap Bubble Tests - Any leak detected by the following soap bubble tests are repaired prior to proceeding with further tests. Upon completion of the containment vessel, a soap bubble test at five psig is made of all welds and seals. The tests are also made on each door of the personnel air locks. A second soap bubble test of all welds and seals are conducted at the pressure of 39.6 psig pneumatic, upon completion of the overload pressure test defined below. Soap bubble tests are performed by applying a thick soap solution to all welds and seals after pressurizing, checking for bubbles or dry flaking as indications of leaks.

b) Leak Testing of Personnel Locks - With the containment vessel at atmospheric pressure, the air locks are pressurized with air to the pressure of 49.5 psig pneumatic. All welds and seals are observed for visual signs of distress or noticeable leakage. The air lock pressure is then reduced to the pressure of 39.6 psig pneumatic and a soap bubble test performed. All leaks and questionable areas are clearly marked for identification and subsequent repair. During the over pressure test, the inner door is blocked with hold down devices to prevent unseating of the seals. If leaks are detected, the internal pressure of the air locks are reduced to atmospheric pressure and all leaks are repaired, after which the air locks are pressurized to the pressure of 39.6 psig pneumatic with air and all areas suspected or known to have leaked during the previous test are retested by the soap bubble method. This procedure is repeated until no leaks are discernible by this means of testing.

   If a personnel lock had undergone initial tests before it is attached to the containment vessel, a soap bubble test of the seals and welds at the pressure of 39.6 psig pneumatic is performed after the lock has been made an integral part of the vessel.

c) Overload Pressure Test - After successful completion of the initial soap bubble test, a pressure test is made on the containment vessel and on each of the personnel locks at a pressure of 49.5 psig pneumatic and held for a minimum of one hour. The inner, as well as outer doors, of the personnel locks are tested at this pressure.

d) Leak Rate Test - After a successful completion of the soap bubble and overload pressure tests, a leak rate test at a pressure of 44 psig pneumatic is performed on the containment vessel with the personnel air lock inner doors closed and atmospheric pressure in the locks. Pressure is maintained for whatever length of time required to demonstrate full compliance with the air-tightness requirements. Continuous readings are taken at least once an hour and continued over a minimum period of 24 hours until it is satisfactorily shown that the leakage rate in any 24 hour period does not exceed 0.2 percent of the total contained weight of air at test pressure at ambient temperature. If it can be demonstrated to the satisfaction of the commission that the leakage rate can be accurately determined during a shorter test period, the agreed upon shorter period may be used (Refer to Subsection 6.2.6.1). Leakage is determined by the "Absolute Method" in accordance with 10CFR50 Appendix J and ANSI N45.4, whereby the actual mass of dry air within the containment is calculated as a function of time.

e) The equipment used for the integrated leak rate tests consists of but is not limited to the following (per 1OCFR50 Appendix J and ANSI N45.4):

1) Absolute pressure is measured by utilizing precision pressure indicators (one is redundant).

2) A barometer and thermometer for measuring outside pressure and temperature is provided.

3) In order to account for temperature effects, resistance temperature detectors (RTD's) are located within the containment.

These detectors are placed spatially within a calculated fractional volume. The average temperature, therefore, is the weighted average.

4) Relative humidity or dewpoint detectors are located the same as the RTD's discussed above.

5) Leakproof stuffing boxes are provided for all lines passing through the vessel shell.

The accuracy of the detectors are as follows:

a)	Pressure	±/- 0.02 percent of reading (0-100 psia scale)
b)	Temperature	±/- 0.1 F
c)	Humidity	±/- 2.5 percent RH or 1.0 F dewpoint

6) A supplemental test is performed whereby the leak rate measurements are validated independently. This validation is performed for a sufficient duration to accurately establish validation following the integrated leak rate test (ILRT) measurements.

f) Operational Testing - After installation, each lock including all latching mechanism and interlock is given an operational test consisting of repeated operation of each door and mechanism to determine whether all parts are operating smoothly without binding or other defects. All defects encountered are corrected and retested. The process of testing and correcting are continued until no defects are detectable. Provisions are also made whereby the doors are tested after each opening.

In-service periodic leakage rate tests of the containment vessel and leak tests of the testable penetrations are conducted to verify their continued leak-tight integrity. This testing is described in Subsection 6.2.6.

→ (DRN 99-0820)

A steel shell pressure containment vessel, designed, fabricated, inspected and pressure tested in accordance with the ASME Boiler and Pressure Vessel Code and protected by the concrete Shield Building will offer continued structural integrity over the life of the unit. The vessel receives a code stamp from an authoritative body (ASME) and represents the most recent developments in the techniques of pressure vessel design and fabrication that are backed up by years of research, testing and successful in-service experience. Beginning with the second ISI interval, the containment vessel will be inspected in accordance with ASME B&PV Code Section XI, Subsection IWE, as modified by 10CFR50.55a.

← (DRN 99-0820)

Testing procedures for the containment, as stated above, are in accordance with the preoperational structural proof test as indicated in NE-6000 of the ASME Code, Section III, Div. 1.

3.8.3 CONCRETE AND STEEL INTERNAL STRUCTURES OF STEEL CONTAINMENT

3.8.3.1 Description of the Internal Structures

The steel containment encloses several structures and structural components which comprise the internal structures of the containment. The main concrete components are the primary and secondary shield walls, a refueling canal, enclosures around the pressurizer and the regenerative heat exchanger. The major internal steel structures are the reactor vessel, steam generators and the reactor coolant pump support framing along with pipe, duct and tray restraints operating floor framing and miscellaneous platform framing. All internal components and framing are designed to seismic Category I requirements. The internal structures are supported on concrete floor fill, which is placed in the bottom of the steel containment vessel. The concrete floor fill, concrete primary and secondary shield walls and the concrete enclosures form compartments within which the entire Reactor Coolant System is located (Figure 3.8-31).

3.8.3.1.1 Primary Shield Wall

A shield wall is provided around the reactor pressure vessel (RPV) extending from the concrete fill at its base up to the RPV flange section and interconnected with the refueling cavity walls and secondary shield walls (see Figure 3.8-32). It is a rectangular heavily reinforced concrete structure, having an internal diameter of 17.54 ft. with a total height above the cavity base of approximately 42 ft. The minimum thickness is 5.67 ft. The reactor vessel is supported by the primary shield wall on a ring girder. Penetrations are provided through the shield wall for the reactor vessel nozzles and Reactor Coolant System. A tunnel through the bottom of the primary shield wall is provided for ventilation and for inspection of the bottom of the reactor vessel.

The following are the functions of the primary shield wall:

a) provide biological shield during normal operation (refer to Section 12.1),

b) provide a missile shield to prevent any missiles generated within the Reactor Coolant System from impinging upon the reactor vessel (refer to Section 3.5),

c) provide a support structure for the reactor vessel and intermediate platforms, and

d) provide support for pipe whip restraints (refer to Section 3.6).

3.8.3.1.2 Secondary Shield Wall

The secondary shield wall consists of two half-cylindrical heavily reinforced concrete walls, which enclose the steam generators, reactor coolant pumps and reactor coolant piping, on each side of the reactor vessel (see Figure 3.8-31). It extends from the top of the concrete fill at elevation -11.0 ft MSL to elevation +62.25 ft. MSL. The inside radius of the half-cylindrical wall is 27.0 ft., with a wall thickness of four ft. Blowout openings are provided at the bottom of the walls to minimize the potential pressure buildup which might arise during a LOCA. These blowout openings also facilitate drainage of spray water into and out of the area within the secondary shield. A concrete pedestal resting upon the concrete floor fill, supports the steam generator at elevation +8.67 ft. MSL.

Sufficient cooling air is supplied on the inside face of the primary shield to maintain the concrete temperature under operating conditions (see Subsection 3.8.3.3.1).

The following are the functions of the secondary shield wall:

a) provide shielding to reduce radiation levels in the steel containment vessel to levels which will permit normal access,

b) provide missile shield to protect steel containment vessel from missiles generated within the Reactor Coolant System as well as to protect the Reactor Coolant System from any missiles generated from other equipment within the internal structure,

c) provide lateral supports for steam generators and reactor coolant pumps,

d) provide a support structure for the operating floor, and intermediate platforms, and

e) provide support for piping.

3.8.3.1.3 Refueling Canal

The refueling canal connects the reactor cavity and the spent fuel storage pool of the Fuel Handling Building through the fuel transfer tube. The refueling cavity will be flooded with borated water during refueling. The refueling canal walls vary from three to six ft. in thickness and extend from elevation +2.5 ft. MSL to elevation +46.0 ft. MSL. The minimum thickness of the floor is 6.5 ft. A stainless steel liner completely covers the interior face of the cavity for the purpose of rendering the refueling cavity completely watertight. The refueling canal is shown in Figures 3.8-32 and 3.8-33.

The following are the functions of the concrete refueling cavity:

a) provide biological shielding and a watertight compartment in which to carry out the refueling process and

b) provide a support structure for the operating floor, intermediate platform and the missile shield above the reactor pressure vessel

3.8.3.1.4 Enclosures Around Pressurizer and Regenerative Heat Exchanger

The pressurizer enclosure consists of reinforced concrete walls (2.5 ft. thick) a supporting floor and a roof (elevation +72.0 ft. MSL forming a compartment abutting the secondary shield wall. It is supported by the secondary shield wall and columns from the concrete floor fill. Vent openings are provided in the walls and supporting floor slab for pressure relief in the event of a LOCA. The pressurizer enclosure provides biological shielding during normal operation and also serves to contain potential missiles which may be generated as a result of safety/relief valve failure (see Figure 3.8-31).

The regenerative heat exchanger enclosure consists of reinforced concrete walls (two ft. thick) a supporting floor and a roof (elevation +46.0 ft. MSL) forming a compartment abutting the secondary shield wall. It is supported by the secondary shield wall and columns from the concrete fill. The enclosure provides biological shielding during normal operation (see Figure 3.8-31).

3.8.3.1.5 Steel Internal Structures

The reactor vessel is supported on a 21.25 ft. diameter centerline to centerline built up ring girder, 5.0 ft-deep by 2.75 ft. wide. The load is transferred from the reactor vessel to the built up steel ring at the four reactor vessel cold legs and the loads are then transferred from the steel ring to the supporting concrete by means of embedded floor plates and embedded ring plates (see Figure 3.8-34).

➔(EC-8438, R307)

The steam generator is supported on a sliding base. The sliding base rests on bearings which permit free movement of the steam generator during thermal expansion of the primary coolant loop. The sliding base also restrains the steam generator in the event of a pipe rupture. The steam generator also has an upper support whose primary function is to take the loads that occur during LOCA at snubber and key locations and in turn transfer these loads to the internal concrete walls (see Figures 3.8-35 and 3.8-36). The mechanical/structural loads associated with the dynamic effects of a LOCA in the RCS hot leg and cold leg piping have been eliminated with the application of Leak-Before-Break (LBB) (Reference Section 3.6.3). Based on removal of the RCS dynamic pipe break loads, the shim plate pack (stop) on the reactor side of the keyway (underneath the RCS hot leg) was permanently removed from the SG-1 and SG-2 sliding base. Additionally, the SG-1 and SG-2 sliding base supports were modified to remove the shim plate from the perimeter of the SG support skirt flange. These modifications were performed as part of the changeout of the steam generators.

🡐(EC-8438, R307)

The reactor coolant pump (RCP) support is basically a braced structure supported by four columns (see Figure 3.8-37). As in the support for the steam generator, in the event of an accident or seismic condition, the snubber loads are taken by upper steel framing and the loads are then transferred to the concrete through embedded plates welded to the structural framing. The Reactor Coolant System is designed for accident conditions by means of pipe stops (restraints) and as for all major equipment supports, the load is then transferred to the concrete by means of embedded plates (see Figures 3.8-38 and 3.8-39). The pressurizer support is a skirt supported on a concrete base, and has a steel framed service platform with a grating deck at the operating floor.

The safety injection tanks are supported by the major platform at the operating floor. This platform as well as the platform at elevation +21.0 ft. MSL is supported by 23 columns on a 136 ft. diameter ring with vertical bracing between the columns to take all horizontal loads down to the concrete support mat. The platform framing at the lower elevation of -4.0 ft. MSL is supported at the ends directly by the concrete. As for all other equipment supports, all pipe, tray and duct restraint steel loads are transferred to the concrete internal structures by means of attaching these structures to embedded support plates.

The polar crane is used for erecting the major nuclear supply system equipment and for servicing and refueling when the plant is in operation. The crane loads and its built up ring girder support are carried by the steel containment vessel cylindrical walls. A detailed description of the polar crane is given in Subsection 9.1.4.1.3.

Restraint framing is provided for all pipes, equipment, electrical trays and heating and ventilating ducts where failure of any of these items could effect the safe shutdown of the reactor.

3.8.3.2. <u>Applicable Codes Standards and Specifications</u>

3.8.3.2.1 General Codes and Standards

a) Concrete Internal Structures

All concrete internal structures are designed in accordance with applicable portion of ACI 318-63, Ultimate Strength Design Part IV B, with the exception that ACI 318-71 is used for design of reinforcing steel splices. A listing of other standard codes or standards is as follows:

1) ACI-214-65 - Recommended Practice for Evaluation of Compression Test Results of Field Concrete

2) ACI-301-66 - ACI-301-72 Specification for Structural Concrete for Buildings (Exceptions noted in Subsection 3.8.3.6-1-2)

3) ACI-315-65 - Manual of Standard Practice for Detailing Reinforced Concrete

4) ACI-347-68 - Recommended Practice for Concrete Formwork

5) ACI-211.1-70 ACI-211.1-74 (Formerly ACI-613-54) - Recommended Practice for Selecting Proportions for Normal Weight Concrete

6) ACI-304-73 (Formerly ACI-614-59) - Recommended Practice for Measuring, Mixing, Transporting and Placing Concrete

7) ACI (1967) - Manual of Concrete Inspection

8) CRSI-65 - Recommended Practice for Placing Reinforcing Bars (20th Edition)

9) ANSI N45.2.5 - 1974 - Supplementary Quality Assurance Requirements for Installation, Inspection and Testing of Structural Concrete and Structural Steel During the Construction Phase of Nuclear Power Plants.

ACI-349, "Proposed Code Requirements for Nuclear Safety Related Concrete Structures" was not used in Waterford 3 design.

b) Steel Internal Structures

The design, fabrication, erection and inspection and testing of the steel internal structures comply with the applicable requirements of the documents listed below. Specific sections of these documents which have been followed are indicated in the following:

1) American Society of Mechanical Engineers (ASME) "Boiler and Pressure Vessel Code," 1971,

   (a) Section III -Nuclear Power Plant Components Since the component designs were in advanced state and the design was presented and approved in the PSAR prior to the issuance of Subsection NF, Subsection NF was not used in the design of steel internal structures Applicable subarticles in Subsection NF were followed for welding and repairs, inspection, testing and heat treatment requirements.

   (b) Section IX - Welding Qualifications

2) American Institute of Steel Construction (AISC) "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings - 1970, 7th Edition."

3) AWS Structural Welding Code D1.1-72. Subsequent to February 1984 the visual inspection acceptance criteria applied to Category I steel structural welds was in accordance with AWS D1.1 Section 8 except as modified by the following criteria:

   (a) Oversize Fillet Welds:

   Either or both fillet weld legs may exceed design size. Welds may be longer than specified. Continuous welds may be used in lieu of intermittent welds.

(b) Undersize Fillet Welds:

The leg of 5/16 inch and larger fillet welds may be 1/16 inch less than required for a continuous run of 2 inches in length, provided there is no less than a 6 inch separation between each undersize increment. For weld runs less than 8 inches long, an undersize span of 25% of the total weld run length will also apply.

(c) Porosity:

The weld may contain a maximum of 5 percent by surface area of unaligned, unclustered porosity.

(d) Weld Profile:

Convexity height and butt weld reinforcement may be accepted without limit, however, a minimum of 1200 included angle must be present between weld runs in multipass welds for acceptable profile.

(e) Craters:

Underfilled craters are not acceptable in continuous groove butt and grooved full or partial penetration Tee welds. Fillet welds may have underfilled craters provided underfill depth does not exceed 1/16 inch, the crater has a smooth contour, blending gradually with the adjacent weld and base metal and the underfilled crater extends beyond the specified weld length.

(f) Undercut:

Undercut up to 1/32 inch depth is acceptable in all steels. Undercut in 5/16 inch or thicker steels produced to a maximum specified tensile strength of 60,000 psi is acceptable to a depth of 1/16 inch, for a continuous run of 2 inches, provided there is no less than a 6 inch separation between each undercut increment and its surface width is greater than its depth. The summation of shorter than 2 inch increments shall not be considered for acceptance/rejection. For weld runs less than 8 inches long, an undercut span of 25%, of the total weld run length will also apply.

(g) Fusion:

Incomplete fusion between weld metal and base metal is not acceptable when a condition such as shown by Example A on Figure 3.8-47 is visible. Incomplete fusion is acceptable provided a weld profile such as shown by Example B on Figure 3.8-47 is visible.

(h) Arc Strike:

Arc strikes in high-strength low-alloy steels (minimum specified tensile strength greater than 60,000 psi), shall be removed by grinding. The ground area shall be visually inspected to assure complete removal of the arc strike.

For other, maximum specified 60,000 psi tensile steels, arc strikes shall be visually examined and accepted if no cracking is evident. If cracking is evident, the cracked region shall be ground to smooth contour and checked to ensure soundness. Arc strike regions which are not cracked shall not require brushing or grinding before visual examination.

4) ANSI N45.2.5-1974 - Supplementary Quality Assurance Requirements for Installation Inspection and Testing of Structural Steel During the Construction Phase of Nuclear Power Plants.

3.8.3.2.2 NRC Regulatory Guides

The following Regulatory Guides were used:

a)	Regulatory Guide 1.10 Revision 1, 1/2/73	Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures
b)	Regulatory Guide 1.15 Revision 1, 12/28/72	Testing of Reinforcing Bars for Category I Concrete Structures
c)	Regulatory Guide 1.31 (Interim: Position-undated)	Control of Stainless Steel Welding
d)	Regulatory Guide 1.38 (May, 1977)	Packing, Shipping, Receiving Storage and Handling of Items for Water Cooled Nuclear Power Plants.
e)	Regulatory Guide 1.39 (March, 1973)	Housekeeping Requirements for Water Cooled Nuclear Power Plants.
f)	Regulatory Guide 1.55 (June, 1973)	Concrete Placement in Category I Structures (Exceptions noted in Subsection 3.8.3.6.1.3).

3.8.3.2.3 Specifications

a) Material Specifications

1) ASTM and ACI Specifications as specified in Subsection 3.8.3.6

b) Coatings

Steel Structure Painting Council (SSPC) (1963)

1) SSPC-SP-6 Commercial Blast Cleaning

2) SSPC-SP-3 Clean-N-Strip and Needle Gun

c) Purchase Specifications

The purchase specifications were prepared by Ebasco Services, Incorporated and included the requirements for materials, design criteria fabrication, erection, inspection and quality compliance. These specifications emphasize important points of the industry standards and reduce options that might otherwise be permitted by the industry.

The following is a listing of the principal purchase specifications:

1) Drilled in Expansion Type Anchors in Concrete - LOU 1564.468

2) Concrete-masonry (concrete materials and mixes, mixing and transportation; concrete placement, curing and finishing, concrete aggregate processing and concrete mixing plants) - LOU 1564.472

3) Concrete reinforcing steel - LOU 1564.473

4) Mechanical splicing of reinforcing steel - LOU 1564.479

5) Filter and backfill - LOU 1564.482

6) Grouting - LOU 1564.483

7) Structural Steel - LOU 1564.723

3.8.3.3 Loads and Loading Combinations

3.8.3.3.1 Loads

All the major loads to be encountered or to be postulated are listed below.

a) Normal Loads

Normal loads are those loads to be encountered during normal plant operation and shutdown.

They include the following:

Dead Load, D = Dead load consist of dead weight of the concrete and steel internal structures and miscellaneous building item within the containment vessel.

The specific weights used to establish the dead loads are as follows:

1) Concrete: 140 lb./cu.ft.

2) Steel reinforcing: 489 lb./cu.ft.

3) Structural steel: 489 lb./cu.ft.

Live Load, L = Live load is set on the various floors and slabs to assure a structure sufficiently strong to support a random temporary load condition during reactor shutdown. These loads are as follows:

1) Operating floor: 1000 psf or equipment load in designated laydown area whichever is greater

2) Other areas: 100 psf or equipment load in designated area, whichever is greater

Equipment Load, L' = Dead weight of the various pieces of equipment, including water, steam or the other enclosed fluids, piping, cables and trays supported by the containment internal structure. Major equipment loads listed in Table 3.8-20.

Water Load, F = These loads are exerted by the water in the refueling canal which is filled only during reactor shutdown. The water elevation is assumed to be at elevation +46.0 ft. MSL and the specific weight is assumed to be 62.4 lb./cu.ft.

Thermal Load, T = These loads are caused by the expansion of the containment internal structure due to increased internal ambient temperature during normal operation. The temperature of all components of the internal structure is assumed to uniformly stabilize at the same temperature as the internal ambient. This is 120 F; the as constructed temperature is assumed to be 70 F. The thermal load due to neutron radiation within the primary shield wall is also considered.

→(DRN 05-1534, R14-A)

The thermal load for the reactor cavity was analyzed with an ambient steady state air temperature of 145°F for the upper cavity and 135°F for the lower cavity.

←(DRN 05-1534, R14-A)

Pipe or Equipment Anchor Loads, A = The pipe or equipment anchor loads are the loads exerted upon the various structural elements in the containment internal structure by the pipe or equipment restraints for normal thermal expansion of the various piping systems.

b) Severe Environmental Loads

Severe environmental loads are those loads that could infrequently be encountered during the plant life. Included in the category is:

E = Loads generated by the operating basis earthquake (OBE) having a horizontal ground acceleration of 0.05g (refer to Section 3.7)

c) Extreme Environmental Loads

Extreme environmental loads are those loads which are credible but are highly improbable. They include:

E’ = Loads generated by the safe shutdown earthquake (SSE) having a horizontal ground acceleration of 0.10g (refer to Section 3.7)

d) Abnormal Loads

Abnormal loads are those loads generated by a postulated high energy pipe break accident within the containment and/or compartment thereof. Included in this category are the following:

Loss of Coolant Accident Load, P = Pressure equivalent static load within or across a compartment generated by the postulated break and determined by analysis of pressure transients inside the primary and secondary shield wall (refer to Subsection 6.2.1).

Equipment or Pipe Accident Load, Q = These are the static and dynamic loads exerted upon the containment internal structure by a pipe or piece of equipment as a result of a postulated LOCA

Thermal Accident Load, T’,T",T"’ = Thermal loads under thermal conditions generated by postulated LOCA, T’ is associated with 1.0 P, whereas T" is associated with 1.25 P, and T"’ is associated with 1.5 P

Internal Missiles Loads or Jet Forces, M = These loads are due to internal missiles associated with the postulated LOCA, like pipe whipping. The effect of jet impingement forces is also included (refer to Sections 3.5 and 3.6).

3.8.3.3.2 Load Combinations

The design of an internal concrete structure is based upon limiting load factors which are used as the ratio by which loads are multiplied for design purposes to assure that the load/deformation behavior of the structure is one of elastic, small strain behavior at the design load condition. The load factor approach is used in this design as **a** means of making a rational evaluation of the isolated factors which must be considered in assuring an adequate safety margin for the structure. This approach permits the designer to place the greatest conservatism on those loads most subject to variation and which most directly control the overall safety of the structure. In the case of the containment internal structure, therefore, this approach places minimum emphasis on the gravity loads and maximum emphasis on accident and earthquake loads. The loads, hereafter referred to as factored loads, utilized to determine the required limiting capacity of any concrete structural element of the containment structure are computed as shown on Table 3.8-21. The load factors chosen are explained in Subsection 3.8.3.3.3.

The design of steel internal structures is based on loading combinations shown on Table 3.8-22.

3.8.3.3.3 Load Factors

The load factors represent the conservatism desired in the design of concrete structure to guard against an error in judgment in assigning a numerical value to a particular load either by itself or in combination with other loads. The load factors for these structures are variable and depend upon (1) the combination of loads acting as dictated by their effects on the overall safety of the structure, (2) the probability of miscalculation of magnitude in two or more loads, and (3) the probability of simultaneous application of the maximum values of these loads.

The rationale used in obtaining the load factors is explained as follows:

The dead load, equipment load, thermal load, and water load are not subject to great variation. The dead load is quite readily computable to within small limits and a change in it in the future is not likely since the structure is part of a closely controlled facility where alterations require much investigation. The equipment load likewise is obtainable within close limits and major components of equipment will not be changed without a major structural investigation. The thermal loads assumed are extreme and the water load is computable to within very small limits and cannot be exceeded due to the geometry of the refueling canal. For the major portion, if not all of the plant life, these loads will be the major loads exerted. Therefore the ACI-318-63 Code load factors are applied to them for normal operating and shutdown conditions and for the OBE. When placed in combination with other than normal design loads such as a LOCA and a SSE, a load factor of one plus or minus 10 percent is chosen to reflect the accuracy of the calculated dead and thermal loads. A factor of 1.25 is placed on the equipment load to reflect the fact that at time of design all equipment weights may not be finalized. The factor for equipment load is reduced to one plus or minus 10 percent in combination with a SSE to reflect the remoteness of the possibility of the loads being severely incorrect. The factor is greater for these loads in this structure than for the Shield Building to reflect the fact that they more closely govern the overall safety of this structure (refer to Subsection 3.8.4).

Live loads are conservatively computed with an intended purpose of assuring ample strength in the structure to allow for anticipated construction and maintenance conditions. Normal ACI-318-63 factors are utilized to assure that the structure remains significantly below yield for these conditions.

The equipment or pipe anchor (operating) load, while its computation is considered quite valid, may be controlling load for at least a portion of the structure for the life of the structure. It is therefore treated in the same manner as a live load for the normal operating condition to assure a significant margin on elastic action. The load factor is reduced to 1.25 when it is combined with other transient loads, i.e., LOCA and/or OBE, to reflect the lower acceptable margins on elasticity for these conditions, with the exception of the stability check equation (4b) of Table 3.8-21. The load factor is reduced to one plus or minus 10 percent for a SSE and/or LOCA transient loads.

The loss of coolant accident pressure and temperature loads are significant loads and do control the overall safety of the structure. The computation of the loads is considered conservative and the type of accident considered is of an outside limit type. Therefore, the load factors of 1.5 and 1.0 for pressure and temperature respectively reflect the desired margin on elasticity for the structure when the LOCA alone is considered. When considered in combination with other loads of a transient nature (except SSE), the probability of simultaneous application and miscalculation of the loads is considered to be low enough to justify factors of 1.25 and 1.0 for the pressure and temperature load respectively.

The missile, equipment or pipe accident loads are conservatively computed so as to represent what is probably the greatest value they could have. The load factor in combination with other transient loads (except SSE) represents the low probability of their occurring simultaneously, especially as they each affect generally only a portion of the structure. The load factor of 1.25 also represents the desired margin on elasticity when combined with the other loads.

The earthquake load can represent a significant portion of the total load which a portion of the structure must sustain. The load factors for an OBE correspond with the values selected in ACI-318-63 when considered in a normal operating or shutdown condition. In combination with other transient loads, the load factor of 1.25 reflects the desired margin on elasticity.

Since the OBE represents the largest earthquake which would normally be expected at the plant site, the use of load factors greater than one are reasonable. The SSE represents the most intense earthquake which could ever occur at the plant site. It is not expected to occur within the life of the plant and is used to demonstrate the capability of the plant to be safely shutdown should it occur, or be maintained in a safe condition should it occur in addition to a loss of coolant accident. A load factor of one is therefore dictated for the SSE both by itself under normal operating conditions and in conjunction with other loads. Those other loads which are permanent have a factor of one plus or minus 10 percent to reflect the margin of safety desired on the transient loads should the permanent loads be subtractive in nature. The transient loads have a load factor of one applied to them when in conjunction with the SSE to reflect the extremely low probability of simultaneous occurrence of loads and miscalculation of magnitude of loads.

3.8.3.4 Design and Analysis Procedures

3.8.3.4.1 General Considerations

The internal structures are designed to provide structural support and radiation shielding for the Reactor Coolant System, auxiliary systems, and engineered safety features. The internal structures are designed as an integral unit with all structural components interconnected to allow transmittal of all vertical and horizontal design loads to the containment slab.

The structural components are designed using both reinforced concrete and structural steel as appropriate. All design aspects are integrated with the design criteria of the Nuclear Steam Supply System supplier and included particular attention to the combined thermal and dynamic effects. Design loads and loading combinations for the internal structures are listed and described in Subsection 3.8-3.3.

The basic techniques of analyzing the internal structures can be classified into two groups: (1) conventional methods involving simplifying assumptions such as those found in beam theory, and (2) plate theories using various degrees of approximation.

Design considerations implemented in the design of the internal structures are presented in the following discussion.

Loads and deformations resulting from a LOCA and the associated effects on any one of the basic systems are restricted so that propagation of the failure to any other system is prevented. In addition, a failure in one loop of the Nuclear Steam Supply System is restricted so that propagation of the failure to the other loop is prevented. All components of the engineered safety features are protected by barriers from missiles which might be generated from the primary system.

The structures are in general proportioned to maintain elastic behavior when subject to various combinations of dead loads, thermal loads, seismic and accident loads. The upper limit of elastic behavior is considered to be the yield strength of the effective load-carrying structural material. The yield strength, F_y, for steel (including reinforcing steel) is considered to be the guaranteed minimum in appropriate ASTM specifications. The yield strength for reinforced concrete structures is considered to be the ultimate resisting capacity as calculated from the "Ultimate Strength Design" portion of the ACI-318-63 Code.

Reinforced concrete structures are designed for ductile behavior, that is with reinforcing steel stresses controlling. The design is in accordance with the "Ultimate Strength Design" portion of the ACI-318-63 Code, except that design of reinforcing steel splices is in accordance with the ACI 318-71 Code.

Under seismic loading, no plastic analysis is considered. Local yielding or erosion of barriers due to pipe rupture loading or missile impact is considered permissible, provided there is no general failure.

Structural steel is designed in accordance with basic working stress design methods. Increased allowable stresses are used for the accident condition.

The final designs of the interior structures and equipment supports are reviewed to assure that they can withstand applicable design pressure loads, jet forces, pipe reactions, and earthquake loads without loss of function. The deflections or deformations of the structures and supports are checked to ensure that the functions of the containment and safety feature systems are not impaired.

3.8.3.4.1.1 Computer Programs Utilized for Structural and Seismic Analyses

The following computer programs have been used in structural and seismic analyses to determine stress and deformation responses of seismic Category I structures. A brief description of each program and the extent of its use are given below:

<u>FIXMAT 2037</u>

FIXMAT 2037 is an Ebasco in-house computer program which operates on BURROUGHS 6700 and handles the dynamic analysis of lump-mass-spring type models. It provides results of natural periods of vibration, mode shapes participation factors and structural responses. Both methods of time history and response spectrum can be specified. The program also generates floor response spectra.

This program was used for all seismic analysis of seismic Category I structures and to calculate all floor responses and their spectra curves.

STARDYNE 2 AND NASTRAN

STARDYNE 2 AND NASTRAN are public domain computer programs designed to analyze static and dynamic problems of linear elastic structural systems using finite element techniques.

The programs are capable of a) computing structural deformations and member loads and stresses caused by an arbitrary set of thermal and mechanical applied loads' and/or prescribed displacements, and b) dynamic response analyses for transient, steady state, harmonic, random and shock spectra excitation type loading conditions. The results are presented as displacements, accelerations or velocities and/or as internal member loads/stresses.

EAC/EASE

The EAC/EASE (Elastic Analysis for Structural Engineering) is a public domain computer program developed by Engineering/Analysis Corporation (Redondo Beach, California) which provides static structural analyses of linear, three-dimensional systems, subjected to sets of arbitrarily prescribed mechanical and thermal loads and displacement boundary conditions. The program is capable of modeling with three distinct types of structural elements, beams, membranes, and plates, which can be used separately or together in assembling a three-dimensional array. The program computes joint displacements, reactive forces, beam forces moments and stresses.

Rigid Frame 2117

Rigid Frame 2117 is an Ebasco in-house computer program which analyzes a two dimensional single or multi-story rigid frame under vertical or horizontal loads. This is accomplished by using a stiffness matrix approach with a Gaussian elimination method. This program was used for frame analysis of all seismic Category I structures.

FIXMAT 2037 program was developed by Ebasco. Since this program is not a recognized program in public domain, a comparison with STARDYNE (version 4/1/72) and NASTRAN, both proven programs in public domain, is made in Tables 3.8-23 to 3.8-30 to demonstrate its validity and applicability.

Rigid Frame 2117 is also an Ebasco program and operates on a Burroughs 6600 machine. Due to the relatively simple nature of the program, comparison of results were made by solving several sample problems with known solutions to demonstrate its validity and applicability.

As discussed above, CDC/STARDYNE and EAC/EASE programs are proven programs existing in the public domain and therefore no comparison of results with other programs is presented.

3.8.3.4.1.2 Analysis and Design Procedures

a) Dynamic Analysis

Analytical techniques for the seismic dynamic analysis are described in Section 3.7.

Analytical techniques for the protection against dynamic effects associated with the postulated pipe rupture are described in Section 3.6.

Analytical technique for the protection against missiles is described in Section 3.5.

b) Design Procedures

All the structural elements of the internal structures are analyzed statically based on a LOCA loading combination described in Subsection 3.8.3.3. The equivalent static load resulting from the application of the accelerations at various levels obtained from the above mentioned dynamic analysis are included.

1) Primary Shield Wall

The MRI/STARDYNE 2 computer program was used to analyze the thick wall behavior using cubic elements "CUBEG" in the model. The model is fixed at the base at elevation -11.00 ft. MSL.

The results of the analyses indicated that the critical section of the reactor cavity was governed by a postulated high energy pipe break accident loading condition (Equation 3 of Table 3.8-21. The loads generated by the postulated break are discussed in Subsection 6.2.1. Provisions are incorporated in the reactor cavity design to safely withstand the calculated differential pressure so as to maintain the integrity of the reactor cavity.

The reinforcing arrangement of the primary shield wall and anchorage into the concrete floor fill at elevation -11.0 ft. MSL is shown on Figure 3.8-33.

2) Secondary Shield Wall and Refueling Canal Wall

The analysis of the combined secondary shield wall and refueling canal wall is done by MRI/STARDYNE 2. Due to symmetry only the half model is employed which is made of 948 finite triangular bending plates. The output includes moments, in-plane shears and out-of-plane shears.

The boundary condition is such that the model is fixed only at the bottom and free at the top.

The secondary shield wall is designed to withstand differential pressure across the cavity walls resulting from a postulated pipe break.

3) Steam Generator Support

For the upper support the finite element method is used. The model consists of steel beam elements acting together with concrete plate elements forming a composite section. The entire structure is assumed fixed at elevation -11.00 ft. MSL. The analysis is carried out using the STARDYNE program. Forces that occur during a LOCA are transferred to steel girders and columns from the steam generator at snubber and key locations and in turn are transferred to the concrete.

For the lower support the finite element method is also used. The steel plate assembly and concrete pier were modeled together as one unit. The entire structure is fixed at elevation -11.0 ft. MSL. The top plate of the steel assembly is allowed to expand freely laterally by providing a gap between the top steel plate of the steel assembly and the top six or 12 in. of the concrete all around. The analysis as carried out using the STARDYNE program. The horizontal forces are transferred through bearing between the key and stiffener plate to the bottom of the steel plate assembly and in turn to the concrete pier.

Vertical loads (compression) are transferred in bearing between the concrete and the steel. Vertical loads (tension or uplift) are transferred through anchor bolts as for any conventional structure.

4) Reactor Vessel Support

All design and analysis was based on the elastic theory. The built up inner ring girder was designed using the NASTRAN finite element program with a spring support. The lateral loads were transferred to the embedded ring girder by the use of shear keys. The embedded ring girder was designed by the STARDYNE rigid frame program using a hinged support. The forces or stresses obtained from the above computer outputs have been increased by a contingency factor for the design, 15 percent for the steel design and by 25 percent for the loads to be transferred to the concrete. The compressive loads were transferred to the concrete through a continuous ring base plate and the tension or uplift was transferred to the continuous concrete base support by means of anchor bolts.

5) Reactor Coolant Pump Supports

The analysis of the reactor coolant pump supports is done by MRI/STARDYNE 2 using the same model for the analysis of the secondary shield wall and refueling canal wall.

The vertical column supports are arranged as a braced space frame positioned between the bottom of the pump and the top of the concrete mat and is designed to take the tension and compressive loads with a minimum misalignment.

The horizontal stops are rolled sections placed adjacent to the sides of the pump at the designated elevations and transmit the compressive forces to the concrete wall. The horizontal column supports are welded to embedded plates for the compressive load and to anchor plates (with anchor bolts) for the tensile loads.

The snubber support (the uppermost support) is designed for both tension and compression, and transmits the load to the concrete wall.

A contingency factor of 15 percent is added to the combined loads to obtain a design load. A load factor of 1.25 is added to the design loads for all structural members in contact with the concrete. All the appropriate design loads are transferred to the internal concrete wall or slabs by means of embedded plates, with shear bars and anchor bolts where required.

6) Reactor Coolant Pipe Stops

The design of the reactor coolant pipe stops is first performed by hand calculations. A model is then set up and checked by the MRI/STARDYNE program to ascertain that the stiffness of the model is within the range required by Combustion Engineering.

The vertical column support which is braced with horizontal members is the main structure carrying the entire vertical load to the concrete mat below. A high-hardness forged steel saddle located at the top of the column, will be positioned in the field by means of shim plates to a level so that a prescribed hot gap can be secured accurately during the hot functional test.

To obtain the required stiffness for a stop column of limited space, a flexible plate with a two or four sided support, is introduced.

The horizontal members connected to the column are designed mainly to take horizontal forces due to the wedging or friction effect.

A load factor of 1.25 is added to the design loads for structural members in contact with the concrete.

Table 3.8-31 shows the actual and allowable stresses for reactor coolant pipe stop A.

7) Pressurizer and Regenerative Heat Exchanger Enclosure

Ebasco Computer Program-2117 "RIGID FRAME ANALYSIS" is used to analyze the frame analysis. For a description, refer to Subsection 3.8.3.4.1.1.

8) Main Steam and Feedwater Pipe Restraints

The majority of the restraints are soft restraints (U-bolts) because of their energy absorbing capability in view of the large numerical value of these loads. The U-bolts in turn are supported by a braced space frame or tower. The STARDYNE program is used for the analysis of the braced space frames or towers. The frame column loads are transferred to an embedded semicircular ring girder and in turn to the concrete walls or transferred directly to the concrete mat.

9) Cable tray and duct restraints are designed as nonrigid structures with a natural frequency of $\geq$ 16 cycles per second.

10) Steel framed platforms are designed for 100 psf generally with heavier loads for laydown areas and for the various equipment loads. The loads for the operating floor platform at elevation +46 ft. MSL and the intermediate platform at elevation +21 ft. MSL are taken down to the concrete mat at elevation -4 ft. MSL by means of steel columns. The lowest platform at elevation -4 ft. MSL is supported directly by the concrete wall by means of embedded plates.

11) The polar crane bridge girders are supported by a ring girder, which in turn is supported by 72 brackets directly to the containment vessel shell plate.

3.8.3.4.2 Mechanism for Load Transfer

The seismic shears and moments are transferred from the internal structure and containment vessel into the base slab through bearing and bond. The movement of the concrete inside or outside of the containment shell relative to the shell or base is precluded by limiting the concrete bearing and bond stresses to below concrete code allowable stresses. The bearing is effected by the geometry of the bottom head of the containment vessel.

3.8.3.5 Structural Acceptance Criteria

3.8.3.5.1 Concrete Structures

Using the factored loads, the various components have the required load capacity if the stresses do not exceed the yield strengths of the materials used.

The capacity reduction factors (ϕ) are provided to allow for variations in material strength, workmanship and dimensions resulting in a structural element having less than its theoretical capacity. In the ACI Code 318-63, ϕ varies with the type of stress or member considered; that is, with flexure, bond shear, or compression.

The ϕ factor is multiplied into the basic strength equation or, for shear, into the basic permissible unit shear to obtain the dependable strength. The basic strength equation gives the "ideal" strength assuming materials are as strong as specified, sizes are as shown on the drawings, the workmanship is excellent, and the strength equation itself is theoretically correct. The practical, dependable strength may be something less since all these factors vary.

The ACI 318-63 Code provides for these variables by using the following ϕ factors which will be used for the design of the Shield Building and the containment internal structure. Additional ϕ values are assigned to each material and condition not covered directly by the ACI 318-63 Code. The additional ϕ factors have been selected based on material quality in relation to the existing ϕ factors.

ϕ = 0.90 for concrete in flexure

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

ϕ = 0.75 for spirally reinforced compression members

ϕ = 0.70 for tied compression members

ϕ = 0.90 for fabricated structural steel

ϕ = 0.90 for reinforcing steel in axial tension without splices

ϕ = 0.90 for reinforcing steel in axial tension with mechanical splices

ϕ = 0.85 for reinforcing steel in axial tension with lapped splices

Mechanical splices will develop at least 125 percent of the yield strength of the reinforcing steel. Therefore, ϕ = 0.90 is used for this type of splice. Reinforcing steel in axial tension without splices is not as sensitive to variations in placing and workmanship as flexural reinforcing, however, the same ϕ factors will be applied in both cases.

In all cases, the loads, moments and stresses calculated for each of the design loading combination given in Subsection 3.8.3.3 for the Reactor Building internal structures are within the ultimate capacity of the structural elements. Table 3.8-32 gives a comparison of the calculated values of shear, moment and stress with the ultimate capacity for the principal structure members of the structural elements, and indicates the margin of safety provided in the design.

3 8.3.5.2 Steel Structures

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For all normal operating loading combinations, such as dead load plus thermal load for equipment, or live load plus dead load for duct and tray restraints, or live load plus dead load plus equipment load for platforms or live load plus dead load plus impact load for cranes, the allowable stresses are as permitted by the AISC code, 1970, 7th Edition. For the addition of the operating basis earthquake (E) to the normal operating loading combination, no increase in code allowable stresses is allowed. For all loading combinations that include abnormal loads or a SSE, the allowable stress is increased to 90 percent of the code yield stress for flexure and to $0.9Fy/\sqrt{3}$ for shear.
←

The limiting values of stress, strain, and gross deformations were established as follows:

a) to maintain the structural integrity when subjected to the most severe load combinations, and

b) to prevent structural deformations from displacing the equipment to the extent that the equipment suffers a loss of function.

The calculated results for representative structural members of the Nuclear Steam Supply System steel internal structures, shown in Tables 3.8-33 through 36 and indicate the margins of safety provided in the design.

All computer output forces and stresses for the Nuclear Steam Supply System equipment supports have been increased by 15 percent for the design of the steel members and by 25 percent for the loads transferred to the concrete. For actual final loads for the reactor vessel support framing see Table 3.8-36.

3.8.3.6 <u>Materials, Quality Control and Special Construction Technique</u>

The basic materials used for the construction of internal structures are concrete, reinforcing steel and structural steel (refer to Table 3.8-37). The material specifications, testing steel requirements, and quality control measures specified in this section form a part of the overall quality assurance program described in the QA Program Manual.

3.8.3.6.1 Concrete Construction

3.8.3.6.1.1 Codes and Standards

The requirements for the concrete construction conform to purchase specifications compiled for the project and to the appropriate ASTM, ACI and ANSI Standards or portions thereof as required by purchase specifications and discussed below.

3.8.3.6.1.2 Concrete Materials

a) Cement

Type I and Type II low alkali cement, conforming to ASTM C150-1972 was used for the project. The cement contained no more than 0.60 percent by weight of alkalis calculated as $Na_2O + 0.658K_2O$. The cement was manufactured by Texas Industries at Midlothian and Artesia Plants.

Cement which had a temperature in excess of 170°F was not used in the work.

b) Aggregate

Fine and coarse aggregates were furnished by Louisiana Industries from Price Pit Quarries in Louisiana, and conformed to ASTM C33-1971a with the exception that:

1) use of blast furnace slag for coarse aggregate was not permitted,

2) organic material was more strictly controlled in fine aggregate, and

3) grading requirements of coarse aggregate were more stringent.

The coarse aggregate was supplied in two size fractions with the overall size being No. 4 up to one in.

c) Admixtures

Air entraining admixture and chemical admixtures conform to ASTM C260-1969 and ASTM C494-1971, respectively.

1) Air Entraining Admixture

The air entraining admixture, manufactured by Protex Industries, was used in concrete within the dosage limits recommended by the manufacturer. Total air content including that due to use of chemical admixtures generally ranged between
3.5 and 6.5 percent by volume.

2) Chemical Admixtures

Two types of chemical admixtures, i.e., water reducing and water reducing set retarding, Type A and Type D respectively, manufactured by Protex Industries and Master Builders, were used in accordance with the Ebasco specification. Type D admixture was used whenever the dry bulb temperature equaled or exceeded 85 F or the temperature of the fresh concrete equaled or exceeded 75 F. Type A admixture was used in all other concrete.

d) Water

Water used for mixing or curing of concrete was free from any injurious amounts of acids, alkali, salts, oil, sediment or organic matter. The chlorides and dissolved solids were within the following limits:

Chloride ion content (mixing water)	100 ppm
Total soluble chloride ion content (concrete mixture)	250 ppm
Total dissolved solids	1000 ppm

e) Concrete

The concrete ingredients consist of fine and coarse aggregates, Type I or II Portland cement, water and admixtures, added to entrain air and increase workability and retard the initial set time when necessary.

The concrete mixes were designed and tested by Barrow Agee/Peabody Laboratory, to produce concrete of required consistency, workability and a minimum strength 15 percent in excess of the required design strength at 28 days. The required 28 day design strength for the entire concrete internal structure is 5000 psi with the exception of concrete floor fill which is 4000 psi. Slumps for various types of placements were four in. or less.

The concrete was produced from an automatic batching and mixing plant of 200 cu yd per hour capacity installed in the vicinity of the project. Concrete was generally shrink mixed in a central mixer at the plant and transferred to nine cu yd trucks for final mixing and delivery.

3.8.3.6.1.3 Quality Control

The quality control requirements for concrete construction are in accordance with ANSI N45.2.5 (1974) and Regulatory Guide 1.55 (June 1973) except as noted herein. Quality control procedures are established and implemented at the site for installation, inspection and testing of concrete construction to verify conformance to specified requirements. These are discussed in the following paragraphs.

a) Cement

Sampling and testing of cement was performed on each 1200 tons delivered to the site in accordance with ASTM C183-1971 and ASTM C150-1972. A certified mill test report, attesting to the conformance of cement to the specification by the manufacturer's chemist was furnished with each shipment and reviewed. The testing program was supplemented by inspection at the batch plant and measurement of cement temperature.

b) Aggregate

Sampling and testing of aggregates were performed during production in accordance with ASTM C33-1971a and the test frequency specified below:

Gradation (ASTM C136-1971)	- once per shift
Organic impurities (ASTM C40-1966)	- weekly
Material finer than No. 200 sieve (ASTM C117-1969)	- daily
Clay lumps and friable particles (ASTM C142-1971)	- monthly
Specific gravity and absorption (ASTM C127-1968 or C128-1968)	- weekly
Percentage voids (ASTM C30-1970)	- weekly
Lightweight pieces (ASTM C123-1969)	- monthly
Soft fragments (ASTM C235-1968)	- monthly
Los Angeles abrasion (ASTM C131-1969 or C535-1969)	- 6 months
Flat and elongated particles (CRD C119-1953)	- 6 months
Potential reactivity (ASTM C289-1971)	- 6 months
Soundness (ASTM C88-1971a)	- 6 months
Moisture content (ASTM C566-1967)	- daily

The above program was supplemented by visual inspection of aggregate stockpiles, daily during concrete production or weekly during off-production periods, to verify conformance to the specifications.

c) Admixtures

Inspection was performed for each batch of admixture delivered to the site to assure that certified infrared spectrophotometry analyses reports were supplied by the manufacturer, attesting to the conformance ot the admixture supplied to admixture used in trial mixes. Inspection was also performed to verify the type of admixture, Type A or Type D, used in concrete in accordance with the ambient temperature limitations.

d) Water

Sampling and testing of water was performed every month for chloride content and every three months for dissolved solids. In addition, tests for effect on compressive strength setting time and soundness were also performed to verify compliance with American Association of State Highway Officials (AASHO) T-26 on a continuous basis.

e) Concrete

Inspections were performed to verify that the concrete batch plant and the transporting equipment complied in all respects, including provisions of storage and precision of measurements, with ASTM C94-1972 and National Ready Mixed Concrete Association (NRMCA) - Certification of Ready Mixed Concrete Production Facilities and Measuring the Uniformity of Concrete Produced in Truck Mixer (1972).

Daily inspections were performed of all truck mixers and agitator units to verify: satisfactory interior condition, no appreciable accumulation of hardened concrete, blades free of excessive wear, charging and discharging chute in good condition, equipped with counter in working condition to indicate the number of total revolutions and mixing revolutions of drum.

Six month inspections were performed on truck mixers for the following:

a) to check blade wear in the drum,

b) to check each truck is equipped with a plate showing manufacturer, mixer capacity and mixing speed, and

c) to verify uniformity of concrete produced.

During concrete production, periodic inspections were also made for the following:

a) to check scale remote readout for zero reading prior to the weighing of a batch,

b) to check batch selector to assure that proper mix is being batched,

c) to check moisture meter reading to verify moisture and aggregate weight compensation,

d) to witness the weighing of cement, sand, coarse aggregate, water and measurement of admixtures to assure that weights are within the specified limits,

e) to check recording ticket for confirmation of batch weights, and

f) to verify that the truck ticket has batch time stamped on it.

Inspections were also made prior to, during and after placement of concrete to ensure conformance to Ebasco Specification LOU 1564.472 and ANSI N45.2.5-74.

In-process tests were performed on concrete sampled at the truck mixer discharge, in accordance with ASTM C172-1971, to control the consistency and other structural properties of concrete. For pumped concrete, sampling was initially performed at both the truck discharge and pump line discharge to first establish a correlation in concrete properties (slump, air content, temperature and strength); thereafter sampling and testing was permitted at the truck discharge. Additional correlation tests were performed whenever the equivalent pumping lengths exceeded the original length. No concrete was placed into the forms prior to testing and acceptance of concrete. The test requirements, test method and test frequency for concrete conformed to the following:

Requirement	Test Method	Test Frequency
Slump	ASTM C143-1971	First batch produced and every 50 cu yd
Air Content	ASTM C173-1971 or C231-1971T	First batch produced and every 50 cu yd
Temperature		First batch produced and every 50 cu yd
Unit Weight	ASTM C138-1971T	Each 150 cu yd or daily during production whichever is the greater frequency
Compressive Strength	ASTM C39-1971	Each 150* cu yd (1 cylinder @ 7 day, 2 @ 28 days and 1 spare)

Seven day strength results were used to establish a correlation between seven day and 28 day strengths. After a correlation was established, the seven-day tests were used as an indicator of the compressive strengths which were to be expected at 28 days. If seven day tests indicated low strengths, corrective measures were taken immediately without waiting for the results of the 28 day tests.

The strength tests is statistically evaluated in accordance with ACI-301 with a coefficient of variation for the tests not exceeding 10 percent (ACI-214-1965). The actual coefficient of variation for concrete is within 10 percent.

The fabrication and placement of concrete in the containment internal structure did not utilize any special construction techniques.

3.8.3.6.2 Reinforcing Steel

3.8.3.6.2.1 Codes and Standards

The following codes and standards are used to establish the purchase specification governing reinforcing steel.

ASTM A 615-68 - Standard Specification for Deformed and Plain Billet Steel Bars for Concrete Reinforcement

ASTM A 370-68 - Standards Methods and Definitions for Mechanical Testing of Steel Products

ANSI N45.2.5-74 - Supplementary Quality Assurance Requirements for Installation, Inspection and Testing of Structural Concrete and Structural Steel During the Construction of Nuclear Power Plants.

* NRC Regulatory Guide 1.55 lists ACI and ANSI Standards as references in Appendix A, which call for the following:

ACI-301-1972 requires one strength test for every 100 cu yd or fraction thereof, of each concrete mix placed in any one day (one cylinder at seven days, two at 28 days). Similarly ANSI N45.2.5-74 requires strength test (two cylinders at 28 days) from each 100 cu yd concrete placed or a minimum of one set per day for each class of concrete. However, ACI-318-71 requires one strength test (two cylinders at 28 days) for each 150 cu yd of concrete placed.

3.8.3.6.2.2 ‘ Materials

All reinforcing bars, used, are new billet steel in accordance with ASTM A615-1968 Grade 60 (60,000 psi minimum yield strength).

3.8.3.6.2.3 Quality Control

The quality control requirements for reinforcing steel were in conformance with ANSI N45-2.5-74.

Certified mill test reports are furnished by the reinforcing steel supplier for each heat of steel, showing that the reinforcing steel met the specified composition, strength and ductility requirements. The testing frequency is in accordance with ANSI N45.2.5-74 and Regulatory Guide 1.15 (Rev 1, 12/28/72), and includes bend tests on full sections of bars No. 14 and 18 in accordance with ASTM A-615-72. In addition, users tests are performed on representative specimens to supplement the standard mill test.

All reinforcing steel is shipped to the site in bundles bearing a tag identifying its size, grade and code number keyed to heat numbers. This information is verified by certified mill test report, which accompanies each shipment of reinforcing steel. High strength steel bars are further identified with the minimum yield strength rolled into the surface of each bar. Visual inspection of fabricated reinforcement is performed to ascertain dimensional conformance with specifications and drawings. The placing inspector performs visual inspection of in-place reinforcement to assure dimensional and locational conformance with drawings and specifications.

Placing and splicing of bars No. 11 and smaller meet the requirements of ACI 318-71 Code. Larger bars (No. 14 and 18) are spliced with cadweld splices in accordance with the Ebasco specification and ANSI N45.2-5-74.

All cadweld sleeves and cartridges are furnished along with certified mill reports showing conformance to requirements and traceability. This is supplemented by visual inspection on delivery. The qualification splices are made prior to production splicing and are verified by visual inspection and tensile strength requirements. Visual inspections are also made for each production splicing. Each cadweld splice is marked with a sequential number and an operator identification number and located on a map.

The testing frequency and acceptance criteria for cadweld splices conforms to ANSI N45.2.5-74 and Regulatory Guide 1.10 (Rev 1, 1/2/73).

3.8.3.6.2.4 Special Construction Techniques

The fabrication and placement of the reinforcement did not utilize any special construction techniques.

3.8.3.6.3 Structural and Miscellaneous Steel

Structural and miscellaneous steel generally conform to the following ASTM specifications:

Rolled Shapes, Bars and Plates	A36-69 or A441-70, A533 Grade B, Class 2, A588
High Strength Bolts	A325-70a or A490-67
Other Bolts	A307-68
Stainless Steel	A167, Type 304L

Other types of steel were used in small quantities in the internal structures as required.

Mill test reports were obtained for structural steel materials used.

3.8.3.7 <u>Testing and Inservice Inspection Requirements</u>

There are no testing and inservice inspection program for the internal structures.

→(EC-26170, R305)

3.8.3.8 <u>Masonry Wall Design</u>

There are no Masonry Walls in Containment.

←(EC-26170, R305)

→(EC-26170, R305)

←(EC-26170, R305)

3.8.4 OTHER SEISMIC CATEGORY I STRUCTURES

3.8.4.1 Description of the Structures

3.8.4.1.1 Shield Building

The Shield Building, as shown in Figures 3.8-31, 40 and 44, is a reinforced concrete structure constructed as a right cylinder with a shallow dome roof. It has an outside diameter of 154 ft. and a height from base slab to the top of the dome of 249.5 ft. The thickness of the wall is three ft. except at the base (below elevation -18.17 ft. MSL) where it is 10.0 ft. to provide support for the construction of the containment vessel. A nominal four ft. annular space is provided between the interior face of the concrete shield structure walls and the outside face of the steel containment. This space provides the means of collecting and diluting any leakage from the containment vessel following a LOCA. A 4.0 ft. nominal clearance is provided between the bottom face of the concrete shield structure dome and the top of the steel containment dome to allow for access for construction and inspection and to assure freedom of movement of the steel containment. The volume contained within the annulus is 550,000 cu ft.

The Shield Building is a free standing structure without any structural ties between it and the containment vessel above the foundation level. A concrete fill is placed in the bottom of the structure to support the steel containment. The Shield Building is designed to serve the following functions:

a) as a biological shield during normal operation and after any accident within the steel containment up to and including the postulated loss of coolant accident,

b) as a low leakage structure following any accident within the steel containment tip to and including postulated LOCA (refer to Table 3.8-38), and

c) as a shield for the primary steel containment for adverse external environmental conditions due to low temperatures, winds, tornadoes, and external missiles.

The Shield Building is designed to seismic Category I requirements considering the loads and loading combinations as specified in Subsection 3.8.4.3 and Table 3.8-39. The Shield Building is protected against exterior flooding up to elevation +30.0 ft. MSL (See Section 3.4). During normal operation, the Shield Building is maintained at a negative pressure by the Annulus Negative Pressure System, described in Subsection 9.4.5. After a LOCA, the pressure in the annular space will increase due to thermal energy transfer from the containment vessel. This pressure increase will be vented by the Shield Building Ventilation System (Subsection 6.2-3).

→(EC-8427, R307)
The Shield Building concrete construction hatch opening was used to support the replacement of the steam generators and reactor vessel closure head. To restore the opening, new reinforcing bars were used which conform to ASTM A615 Grade 60. Mechanical property tests for yield strength, tensile strength, and percentage elongation as well as bend tests required by ASTM A615 were performed per ASTM A370. Placement of rebar was in accordance with ACI 301. Replacement of rebar in the SG/RVCH construction opening either employed lapped splices, mechanical splices, or limited fusion welding of rebar. Mechanical splices were used to splice replaced rebar to the existing rebar projection. Fusion welding was only used where physical limitations prohibited mechanical splices. Mechanical splices were provided and tested per Specification SPEC-10-00001-W, which demonstrated compliance with ACI 349. As an acceptable design equivalent to mechanical cadweld splicing of rebar, these mechanical splice devices complied with ACI 318-71 to be capable of developing not less than 125% of the specified yield strength of the bars in question. Welded splices were welded and inspected in accordance with AWS D1.4-1998, Structural Welding Code – Reinforcing Steel.
←(EC-8427, R307)

3.8.4.1.2 Reactor Auxiliary Building

→(DRN 00-1121)
The Reactor Auxiliary Building is a multistory reinforced concrete structure located immediately south of the Reactor Building, as shown on Figure 3.8-42. The interior floor construction is a beam and girder construction supported by reinforced concrete columns. The building occupies an area approximately 260 fl. by 219 ft. and extends from top of the common mat at elevation -35 ft. MSL up to roof levels varying from elevation + 46 ft. MSL to elevation + 106.5 ft. MSL. Above the common mat, the building is structurally separated from the centrally located Reactor Building at all levels.
←(DRN 00-1121)
The Reactor Auxiliary Building houses the waste treatment facilities, engineered safeguards systems, switchgear, laboratories, diesel generators and main control room. It further provides protection to the cable and piping penetration areas of the Reactor Building. The building exterior walls, floors, and interior partitions are designed to provide plant personnel with the necessary biological radiation shielding, and to protect the equipment inside from the effects of adverse atmospheric conditions, including winds, temperature, missiles, flooding and corrosive environment. The condensate and refueling water storage pools are contained as an integral part of the building.

The Reactor Auxiliary Building is designed to seismic Category I requirements considering the loads and loading combinations for abnormal/extreme environmental conditions as specified in Subsection 3.8.4.3. The building is protected against exterior flooding up to elevation +30.0 ft. MSL (see Section 3.4).

3.8.4.1.3 Fuel Handling Building

The Fuel Handling Building is a reinforced concrete structure located immediately north of the Reactor Building shown on Figure 3.8-43. It occupies the area approximately 73 ft. by 117 ft. and it extends from top decontamination area for the spent fuel cask, and miscellaneous equipment. The spent fuel pool is a stainless steel-lined reinforced concrete tank structure that provides space for storage of spent fuel, spent fuel cask and miscellaneous items.

→(EC14275, R306)
The spent fuel cask is handled by a 125 ton bridge crane which runs on rails, supported on a reinforced concrete frame above the operating floor (elevation +77.0 ft. MSL). The equipment and new fuel storage area are serviced by monorail hoists and a 15 ton crane. The handling of the spent fuel in the spent fuel pool is accomplished with the spent fuel machine which runs on rails along the top of the spent fuel pool structure (see Subsection 9.1.4.1.3). The spent fuel is transferred from the Reactor Building via the fuel transfer tube into the refueling canal. Direct railroad Access is provided into the building at the grade level. The building, other than the rail bay, is protected against exterior flooding up to elevation +30 ft MSL (see Section 3.4).
←(EC14275, R306)

The Fuel Handling Building exterior walls, floors, and interior partitions are designed to provide plant personnel with the necessary biological radiation shielding and to protect equipment from the effects of adverse atmospheric conditions such as winds, temperature, missiles, flooding and corrosive environment.

The Fuel Handling Building is designed to seismic Category I requirements considering the loads and loading combinations for abnormal/extreme environmental conditions as specified in Subsection 3.8.4.3.

3.8.4.1.4 Component Cooling Water System (CCWS) Structure

Component Cooling Water System (CCWS) structure comprises two independent sets of dry and wet cooling towers located on the east and west side of the Reactor Building. Each set of dry and wet cooling towers consists of reinforced concrete box structure with overall dimension 37 ft. by 103 ft. and 26 ft. by 57 ft., respectively. An access to the equipment hatch of the Reactor Building is provided in the west CCWS structure.

The cooling towers are supported on the common mat at elevation -35 ft. MSL and extend in height to elevation +30 ft. MSL.

Each dry cooling tower is further subdivided into five reinforced concrete chambers and is equipped with three fans supported on the walls at different levels in each chamber (totaling 15 fans). Each wet cooling tower has two reinforced concrete chambers. The minimum thickness of the walls is two ft. with 3.5 ft. thick walls supporting the fans.

The CCWS structure is designed to seismic Category I requirements considering the loads and loading combination for abnormal/extreme environmental conditions as specified in Subsection 3.8.4.3.

3.8.4.2 Applicable Codes, Standards and Specifications

The applicable codes, standards, and specifications are given in Subsection 3.8.3.2.

The only exception for this is that flexure analysis for beams and slabs in the other Category I structures may be performed using the load factors for live and dead loads from ACI 318-71.

See subsection 3.8.4.3.3 for a more detailed discussion of this. The load factors from ACI 318-63 must be used for all new concrete designs. The load factors from ACI 318-71 may only be used for reviewing new or revised loads resulting in flexure on beams and slabs.

3.8.4.3 Loads and Loading Combinations

3.8.4.3.1 Loads

All major loads to be encountered or to be postulated are listed below.

a) Normal Loads

Normal loads are those loads to be encountered during normal plant operation and shutdown. They include the following:

Dead Load, D = Dead load consists of dead weight of the concrete structure, structural steel and miscellaneous building items within the building.

The specific weights used to establish the dead loads are given in Subsection 3.8.3.3.1. In addition, the specific weight of water was taken as 62.4 lb/cu.ft.

For Fuel Handling Building, the water level in the spent fuel pool was assumed to be at operating floors level, elevation +46 ft. MSL.

Live Load, L = Live load is set on the various floors and slabs to assure a structure sufficiently strong under normal operation to support a random temporary load condition during reactor shutdown. These loads are as follows:

Shield Building Live load consists of snow or construction loads on the dome which are uniformly applied to the roof at a value of 30 lb per horizontal plant projection square ft.

Reactor Auxiliary and Fuel Handling Buildings Roof: 30 psf

Floors: 100 psf or equipment load in designated area, whichever is greater.

Equipment Load, L'= Equipment load consists of dead weight of the various pieces of equipment including water, steam or other inclosed fluid within the equipment.

Buoyancy, B = Uplift load exerted by the displacement of ground-water, assumed to be at elevation +8.0 ft. MSL.

Thermal Load,T = Thermal load is that load induced by thermal gradient existing across the walls between the building interior and the ambient external environment during normal operating or shutdown conditions. Both winter and summer operating conditions are considered as follows:

1) Summer (Operating and Shutdown)

(a) Interior sustained air temperature (Shield Building) 120°F

(b) Interior sustained air temperature (Reactor Auxiliary and Fuel-Handling Buildings) 104°F

(c) Exterior sustained concrete temp (air) 95°F

(d) Exterior sustained concrete temp (all soils) 70°F

(e) Spent fuel pool water 150°F

2) Winter (operating)

(a) Interior sustained air temperature

i - Shield Structure 70°F

ii - Fuel Handling and Reactor Auxiliary Bldgs 80°F

(b) Exterior sustained concrete temp (air) 30°F

(c) Exterior sustained concrete temp (recent soils) 46°F

(d) Exterior sustained concrete temp (Pleistocene soils) 70°F

(e) Spent fuel pool water 120°F

3) Winter (Shutdown)

(a) Operating Temperature Inside Buildings 50°F

(b) Exterior sustained concrete temperature (air) 30°F

(c) Exterior sustained concrete temperature (Recent soil) 46°F

(d) Exterior sustained concrete temperature (Pleistocene soil) 70°F

For all cases, an "as constructed" concrete temperature is assumed at 70°F.

Earth Loads, S, S' = These are the loads caused by the pressure of the earth resting against the structure both in an "at rest" condition and during an earthquake, respectively. The loads are based on the following soil properties.

1) Natural soil (recent: Grade to elevation -40.0 ft MSL)

(a) Dry unit weight: γd = 80 pcf

(b) Saturated unit weight: γs = 115 pcf

(c) Submerged unit weight: γb = 52.5 pcf

(d) Angle of internal friction: $\phi = 0°$

(e) Cohesion: C = 500 psf

2) Natural soil (Pleistocene: below elevation -40.0 ft MSL.)

(a) Dry unit weight γd = 90 pcf

(b) Saturated unit weight: γs = 120 pcf

(c) Submerged unit weight: γ b = 57.5 pct

(d) Angle of internal friction: $\phi = 0°$

(e) Cohesion: C = 1,650 psf

3) Backfill (sand)

(a) Dry unit weight γ d = 105 pcf

(b) Saturated unit weight: γ s = 128 pcf

(c) Submerged unit weight: γ b = 65.5 pcf

(d) Angle of internal friction: $\phi = 35°$

(e) Cohesion: C = 0

4) Under earthquake, earth load (S') is considered as passive earth pressure grading to minimum active pressure on the opposite side of the structure. Normal soil pressure(s) shall be "at rest" pressure. The computation and distribution of passive pressure with depth is done in accordance with Subsection 2.5.4.10.3.

Dynamic Load of Machinery (R) = <u>Reactor Auxiliary Building Only</u> This dynamic load is caused by an assumed or actual imbalance in the machine. It is computed, based upon the natural frequency of the machine support structure, and spinning weight of the machine, and converted to an equivalent static load.

Pipe Anchor Load, A = The pipe anchor loads are the loads exerted upon the various structural elements by the pipe or equipment restraints for normal thermal expansion of the piping system.

b) Severe Environmental Loads

Severe environmental loads are those loads that could infrequently be encountered during the plant life. Included in this category are:

E = Loads generated by the operating basic earthquake (OBE) having a horizontal acceleration of 0.05g, (refer to Section 3.7).

Hu = Loads generated by the design wind specified for the plant. These are based on a 200 mph wind with gust factors included at 30 ft. above ground level (refer to Section 3.3).

Buoyancy, B' = Uplift load based on river flooding caused by accidental breaching of the levee with the water level rising to elevation +30 ft. MSL.

Buoyancy, B" = Uplift load, based on minimum groundwater level, which is assumed to be at normal low water level in the river, i.e., elevation +5 ft MSL.

River Transportation Accident Overpressures, AP = Pressure load exerted due to detonation of a gasoline tanker in the Mississippi River channel (refer to Section 2.2).

c) Extreme Environmental Load

Extreme environmental loads are those loads which are credible but are highly improbable. They include:

E'= Loads generated by the safe shutdown earthquake (SSE) having a horizontal ground acceleration of 0.10g (refer to Section 3.7).

W = Loads generated by the design tornado specified for the plant.

These include loads resulting from a tornado funnel with a horizontal rotational velocity of 300 mph and a horizontal translational velocity of 60 mph and from tornado-created differential pressures. The tornado is conservatively taken as 360 mph wind applied uniformly with height and width, with simultaneous decrease in atmospheric pressure of three psi in three seconds (refer to Section 3.3).

M = Loads generated by impact of high velocity external missiles associated with design tornado are considered. See Table 3.5-10.

d) Abnormal Loads

Abnormal loads are those loads generated by a postulated high-energy pipe break accident within a building and/or compartment. Included in this category are the following:

1) Shield Building

The loss of coolant accident load is determined by analysis of the pressure and temperature transients in the annulus during a loss of coolant accident. The shield structure is designed for the pressure and temperature loads given in Subsection 6.2.3. In addition, a bearing pressure of 44 psig for the top four ft of concrete floor fill exerted by the containment vessel has been considered internal pressure.

IN = Internal negative pressure (other than LOCA) of three psig.

IP = Internal positive pressure (other than LOCA) of two psig.

2) Reactor Auxiliary Building

Pressure Load, P_a = Pressure equivalent static load within or across a compartment and/or building, generated by the postulated break, and including an appropriate dynamic factor to account for the dynamic nature of the load.

Thermal Load, Ta = These are thermal loads under thermal conditions generated by the postulated break and including the normal thermal load, T.

Pipe Reaction, Ra = These are pipe reactions under thermal conditions generated by the postulated break and including the normal pipe anchor load, A.

Pipe Loads, Yr = These are the equivalent static loads on the structure generated by the reaction on the broken high energy pipe during the postulated break, and including an appropriate dynamic factor to account for the dynamic nature of the load.

Jet Impingement, Yj = These are the jet impingement equivalent static loads on the structure generated by the postulated break, and including an appropriate dynamic factor to account for the dynamic nature of the load.

Missile Load, Ym = This load is the missile impact equivalent static load on the structure generated by or during the postulated break, like pipe whipping, and including an appropriate dynamic factor to account for the dynamic nature of the load.

3) Fuel Handling Building

The spent fuel pool is designed for the following accident cases:

(a) Equivalent static load on the structure due to accidental dropping of spent fuel cask or the spent fuel cask hitting the pool side walls during the refueling operation. The impact loads are based on a height of drop of 30 ft. above the pool bottom slab.

(b) Load due to loss of water in the fuel cask loading area

(c) Thermal load due to rise of pool water temperature to 200 F during the summer due to loss of heat exchanger.

3.8.4.3.2 Load Combinations

The design of the Shield Building and other seismic Category I structures outside the containment is based upon limiting load factors to assure the structural integrity and adequate margin of safety of these structures (refer to Subsection 3.8.3.3.2). The load combinations are presented in Table 3.8-39. The load factors are explained in Subsection 3.8.4.3.3.

In load combinations 9a), b) and c) of Table 3.8-39, the maximum values of Pa, Ta, Ra, Yj, Yr and Ym are used unless a time history analysis justifies otherwise. The failure capacity of concrete structures is checked by using the "Yield Line Theory," so that the combined loads do not exceed 90 percent of the calculated failure capacity. However, it is also verified that neither excessive deflections nor excessive cracking, will result in the loss of function of any safety related system.

3.8.4.3.3 Load Factors

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Dead loads, equipment loads, and thermal loads are not subject to great variation. The dead load is quite readily computable to within small limits and a change in it in the future is not likely since the structure is part of a closely controlled facility where alterations require much investigation. The equipment load likewise is obtainable within close limits and major components of equipment will not be changed without a major structural investigation. The thermal loads assumed are extreme and the water load is computable to within very small limits and cannot be exceeded due to the geometry of the refueling canal. For the major portion, if not all of the plant life, these loads will be the major loads exerted. Therefore the ACI 318-63 code load factors are applied to them for normal operating and shutdown conditions and for the OBE. The only exception to this is for reviewing new or revised loads on existing concrete structures resulting in flexure on beams and slabs. For these conditions, the load factor for dead load of 1.4 from the ACI 318-71 code may be used only for the flexure analysis on beams and slabs. The load factor was lowered by ACI due to greater research and experience, and due to better quality control for construction. The only change in the analysis for the flexure of the beams and slabs will be the load factor, since the capacity reduction factors as discussed in subsection 3.8.3.5.1, and the equations used to determine the capacity of the concrete structures remain the same between the 1963 and 1971 versions of ACI 318. The load factor of 1.5 must be used for all new concrete design for Category I structures. When placed in combination with other than normal design loads such as a LOCA and a SSE, a load factor of one plus or minus 10 percent is chosen to reflect the accuracy of the calculated dead and thermal loads. A factor of 1.25 is placed on the equipment load to reflect the fact that at the time of design all equipment weights may

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not be finalized. The factor for equipment load is reduced to one plus or minus 10 percent in combination with a SSE to reflect the remoteness of the possibility of the loads being severely incorrect. A load factor of one plus or minus five percent is chosen for Shield Building other than normal operating conditions because these loads generally do not directly govern the overall design of the structure.
←
Force due to buoyancy loads are readily and accurately computable and a conservative assumption has been made for groundwater level both for normal operating and flooding conditions. A load factor of one is justifiable, particularly since the load does not govern the overall safety of the structure.

Live loads are conservatively computed for a condition not likely to occur. It is believed that an extreme value has been chosen but the nature of the load is such that it is subject to variation. The load does have some bearing on the overall safety of the structure; therefore the load factor of 1.8 is chosen for the operating condition. This is chosen to meet the criteria of ACI-318-63.
→
The load factor of 1.8 for live load is the preferred value to be used when checking existing reinforced concrete structures for new or revised loads. However, for flexure analysis of beams and slabs, the load factor of 1.7 for live load may be used if necessary. This is the value given in ACI 318-71. The load factor was lowered by ACI due to greater research and experience, and due to better quality control for construction. The only change in the analysis for the flexure of the beams and slabs will be the load factor, since the capacity reduction factors as discussed in sub section 3.8.3.5.1, and the equations used to determine the capacity of the concrete structures remain the same between the 1963 and the 1971 versions of ACI 318. The load factor of 1.8 must be used for all new concrete design for Category I structures.
←
Loss of coolant accident loads do not directly control the overall safety of the structure. It is however, desirable to assure that a substantial minimum margin of safety, comparable to recent industry practice on concrete containment structures, will exist on any portion of the structure which may be, at this point in time, exposed to such a load which will be controlling the overall safety of that portion. A load factor of 1.5 is consistent with this philosophy.

Earth loads are not subject to great variation above the values chosen which are based upon laboratory testing of the soil. Conservative values are chosen for these values under earthquake conditions. Since the values chosen are considered as extremes, a load factor of one is justified.

Operating basis earthquake loads are subject to variation both above and below the chosen values. These values have been chosen as being conservative for this particular site based upon present day knowledge and the "state-of-the-art" for seismic design and analysis. The effect on the overall safety of the structure is minimal with moderate increases in the load. Carefully weighing the probable accuracy of the predicted earthquake loads along with the probability of such an occurrence and the effect of a possible judgment on the low side, a load factor of 1.25 is chosen both for combination with normal operating loads and in combination with accident loads. The effect of earthquake on the structure is much greater than the effect of accident loads.

Safe shutdown earthquake loads are chosen to be representative of the most severe ground motion which could be postulated for this particular site based upon a rational study and analysis of the region. The intention of utilizing such a load is to demonstrate the functional capability of the structure and therefore a load factor of one is dictated to meet this criteria.

Tornado loads are subject to some speculation as to variation. They do affect the overall integrity of the structure. Little measured data is available pertaining to tornadoes; hence a large load factor would be justified. However, it is felt that the tornado properties chosen for design are reasonable and the probability of a tornado funnel striking the Shield Building is such that the combined probability of a tornado associated load and a direct hit of a tornado funnel justifies a load factor of one.

Hurricane loads are based upon a study showing them to be an extreme value with a very large recurrence interval; however, it is considered probable that the structure will be exposed to hurricane loads in its lifetime. A load factor of 1.25 is chosen for these loads in combination with normal operating loads or in combination with accident associated loads. This factor also reflects the ACI-318-63 value for a similar condition.

Internal negative and internal positive pressure loads are utilized to represent a desired capability for the structure even though this type of loading cannot reasonably occur in the annular space between the steel containment structure and the Shield Building. These loads do not, at their computed values, affect the overall safety of the structure. The 1.25 load factor represents a desired margin on elasticity for this load condition.

3.8.4.4 Design and Analysis Procedures

3.8.4.4.1 Assumptions and Boundary Conditions

The basic assumptions and boundary conditions in concrete design are as follows:

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a) The reinforced concrete Shield Building, Reactor Auxiliary Building and Fuel Handling Building are analyzed in accordance with the ultimate strength design methods of ACI 318-63 Part IV B. The only exception to this is that the load factors from the ACI 318-71 code may be used for flexure analysis of beams and slabs for existing structures.

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b) A fixed boundary is assigned to the Shield Building cylindrical wall and other structural walls and columns that are doweled to the common foundation mat. Other boundary conditions for the buildings are determined in accordance with the framing arrangement.

   Boundary conditions for the analysis of local areas are obtained and verified from the results of the analysis described in Subsection 3.8.4.4.1.2.

c) The Reactor Auxiliary and Fuel Handling Buildings are designed as reinforced concrete structures with beam and girder floors spanning between interior columns and shear bearing exterior walls.

d) The portion of the Fuel Handling Building floor slab which is under the spent fuel cask storage area is structurally independent of the spent fuel storage pool slab. This is necessary in order to insure that the cask storage area slab can react under the impact of a dropped cask without having damaging structural effects on the independent fuel storage pool slab.

3.8.4.4.2 Computer Programs

Computer programs used in the analysis are described in Subsection 3.8.3.4.1.1.

3.8.4.4.3 Design and Analysis Procedures

a) Dynamic Analysis

Analytical techniques for the seismic dynamic analysis are described in Section 3.7.

Analytic techniques for the missile protection are described in Section 3.5.

Analytic techniques for the protection against dynamic effects associated with the postulated pipe rupture are described in Section 3.6.

Analytical techniques for the wind and tornado loadings are described in Section 3.3. The analysis is based upon elastic impact between the missile and the cylinder wall. Local penetration due to the high velocity missile is determined using equations suggested in Section 3.5.

b) Design Procedures

All the buildings are analyzed statically, based on loading combinations described in Subsection 3.8.4.3. The equivalent static loads resulting from the application of the accelerations or displacements at various levels obtained from the above mentioned dynamic analysis are utilized. Design procedures generally conform to the requirement of Subsection 3.8.3.4.1.

1 - Shield Building

Since the thickness of the shallow dome and right cylinder are small in comparison with the radii of principal curvatures, the dome and cylinder are considered as a thin-walled shell in the form of surface of revolution.

The static analysis of the Shield Building is performed by applying the CDC/STARDYNE 2 computer program and by a half-model made of finite triangular bending plates. The properties of this half-model is that the structure is symmetric with respect to the plane of symmetry but the loads and deflections need not be symmetric. No opening is considered in this model.

The critical sections for shear and tension stresses are at (1) the juncture of shield structure cylinder and the mat, (2) the juncture of the shield structure cylinder and the dome, (3) around penetrations and (4) those regions of contraflexure and maximum curvature caused by seismic and wind loadings. The finite element modeling of the shield structure into many triangular elements provides the requisite definition for items (1) and (2). Another model, with finer mesh, is prepared for the areas around major penetrations and using boundary loading conditions determined by the coarser element model to provide the requisite definition for item (3). Major penetrations were defined as those having an inside diameter equal to or greater than 2.5 times the Shield Building wall and included the construction hatch, equipment and maintenance hatch, and personnel locks. Item (4) is determined by an inspection of the load combination output of the program to determine critical design areas in addition to items (1), (2) and (3).

The controlling shear forces at critical locations are generally designed to be resisted by the shear strength of concrete. In localized areas, stirrups are used to distribute shear stresses to the degree compatible with the concrete shear strength of the surrounding shell. The margin that will be present with respect to failure is inherent in the shield structure load combinations and code allowable stresses as shown in Subsection 3.8.4.5.

2 - Reactor Auxiliary Building

Reactor Auxiliary Building is analyzed by CDC/EASE computer program for loading combinations given in Subsection 3.8.4.3 for various two dimensional frames.

3 - Fuel Handling Building

→ (DRN 99-1095)

Portions of the Fuel Handling Building are analyzed by conventional methods. Other portions of the building are analyzed using the NASTRAN computer program described in Subsection 3.8.3.4.1.1.

← (DRN 99-1095)

The resultant internal loads and stresses from the computer program of each individual loading condition are combined in accordance with the equations of Subsection 3.8.4.3 and used for proportioning of all components of structures. Concrete structures are designed for ductile behavior, that is, reinforcing steel stresses controlling. The design is in accordance with the Ultimate Strength Design portion of ACI-318-63 code with the exception that ACI-318-71 code was used for design of reinforcing steel splices.

Under seismic loading, no plastic analysis is considered. Local yielding of structures is considered permissible due to LOCA or missile forces provided there was no general failure.

3.8.4.4.4 Mechanism for Load Transfer

Load transfer from seismic Category I structures to the foundation mat is discussed in Subsection 3.8.5.4.3.

3.8.4.5 Structural Acceptance Criteria

The structural acceptance criteria for the seismic Category I structures is given in Subsection 3.8.3.5.1.

In all load conditions, the calculated design loads are within the ultimate capacity of the structural members.

Tables 3.8-40, 41, and 42 give a comparison of the calculated values of shear, moment and stress with ultimate capacity for the principal structural members of the Shield Building, Reactor Auxiliary Building and Fuel Handling Building, respectively. The calculated values are given for the most severe loading conditions for the particular member and indicate the margin of safety provided in the design.

3.8.4.6 Materials, Quality Control and Specific Construction

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The basic materials used for the construction of seismic Category I structures, described in Subsection 3.8.4.1, are concrete, reinforcing steel and structural steel. The material specifications, testing requirements and quality control measures specified in this section form a part of the overall Quality Assurance Program described in the QA Program Manual.
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3.8.4.6.1 Concrete Construction

3.8.4.6.1.1 Codes and Standards

See Subsection 3.8.3.6.1.1

3.8.4.6.1.2 Concrete Materials

Concrete materials conforms to requirements discussed in Subsection 3.8.3.6.1.2 except that the required 28 day design strength for the Shield Building, RAB Fuel Handling Building and the CCWS structure concrete is 4000 psi.

3.8.4.6.1.3 Quality Control

See Subsection 3.8.3.6.1.3

3.8.4.6.1.4 Shield Building Tolerances

The finished concrete tolerances for the Shield Building are as follows:

a) Cylindrical Wall

1) Variation from plumb: not more than ± four in. for the total structure height, taken at vertical axis of vessel, or more than ± one in. in any 20 ft. of wall height

2) Variation from true circular section: not more than ± three in. in radius from vertical axis of vessel

3) Variation of wall thickness: not more than - 1/4 in. or more than + one in.

b) Dome

1) Variation from true spherical section: not more than ± three in. in radius

2) Variation in dome thickness: not more than + one in. or more than -1/2 in.

3.8.4.6.1.5 Special Construction Techniques

a) Shield Building

The three ft. cylindrical portion of the Shield Building was constructed by using the slip form construction technique. The dome was constructed in two stages by first forming a thin layer of structural concrete supported by a form, which was in turn supported by the dome of the steel containment vessel. The second stage of the dome was supported on the thin dome, which was designed for its dead load, construction load and dead load of the second stage. A typical reinforcement of the Shield Building dome is shown in Figure 3.8-41.

3.8.4.6.2 Reinforcing Steel

See Subsection 3.8.3.6.2

3.8.4.6.3 Structural Steel

See Subsection 3.8.3.6.3

3.8.4.7 <u>Testing and Inservice Surveillance Requirements</u>

Testing of structural materials during the preoperational and construction period is discussed in Subsection 3.8.4.6.

Various concrete walls were subject to the crack surveillance program described in Subsection 2.5.4.13.4. There are no additional planned systematic testing or surveillance programs for the seismic Category I structures after the plant has been placed in operation. The structural steel framing and connections will be generally accessible to visual inspection.

3.8.4.8 <u>Masonry Wall Design</u>

→(EC-26170, R305)

All concrete block masonry walls in the proximity of safety related piping or equipment and those enclosing stairwells and elevators are not classified seismic Category I but have been designed to withstand safe shutdown earthquake.

The primary function of these concrete block masonry walls, which vary in thickness from 8 to 12 inches, is to act as a barrier or as a fire wall with a minimum fire rating of 2 hours. The hollow cores are filled with mortar every 4 feet on center and are reinforced with two #6 vertical bars on each face. Every second course (16 inches) is reinforced horizontally with extra heavyweight “Dur-O-Wal” trusses.

The hollow concrete block load units conform to ASTM C-90, Grade N-1 and are composed of normal weight aggregate conforming to ASTM C-33 with cinders unacceptable. The vertical reinforcing steel is deformed, intermediate grade, new billet steel conforming to ASTM A-615, grade 60. The horizontal steel reinforcement conforms to ASTM A-116, Class 3. The mortar is in compliance with ASTM C-270 having a minimum compressive strength of 2500 psi.

The laying of these blocks is in accordance with the recommended practices for laying concrete block by the Portland Cement Association. The follow cells that are filled with mortar are thoroughly rodded every two courses to eliminate entrapped air voids. All concrete block masonry walls receive a full mortar bedding prior to embedding the horizontal reinforcement so as to obtain proper bonding and covered with an additional spread of mortar if necessary to insure full embeddment.

←(EC-26170, R305)

→(EC-26170, R305)
One other function of the concrete block masonry walls is to provide shielding. These are mainly multiple wythe walls with each individual wall horizontally reinforced with extra heavyweight “Dur-O-Wal” trusses at every course. The composite behavior of the walls is assured by connecting the horizontal reinforcing between the individual walls with 16 gage bent straps at 6 inches on center horizontally in each course and 16 inches on center vertically in alternate courses. At the end supports the walls are interconnected with a mortar fill and the horizontal reinforcement is interconnected with a continuous #4 reinforcing bar.

The shield walls resisting pressurization loads have a maximum span of 4 feet and those without pressure loading have a maximum span of 10 feet.

The shielding concrete blocks are the solid type with a maximum uniform density of 138 pounds per cubic foot and a minimum compressive strength of 4000 psi.

Although the concrete block masonry walls are not considered to be seismic Category I, these walls are designed so that in the event of a seismic occurrence or an accident condition, all concrete block masonry walls in the proximity of safety related equipment, components or structures will remain intact.

There are no safety related piping systems or equipment being supported directly or indirectly on the Waterford 3 concrete block masonry walls.
←(EC-26170, R305)

3.8.5 FOUNDATIONS

3.8.5.1 Description of the Foundations

The Nuclear Plant Island Structure (NPIS) housing all the seismic Category I structures, is supported on a continuous reinforced concrete foundation mat 270 ft. wide, 380 ft. long and 12 ft. thick. The foundation rests on a one ft thick compacted shell filter blanket, which is supported by the Pleistocene sediments at elevation -48 ft. MSL. The arrangement of the NPIS and the common foundation mat is shown in Figures 3.8-1 and 31. Figure 3.8-45 shows a typical masonry plan of the common foundation mat.

The cylindrical wall of the Shield Building is directly founded on the common mat. The steel containment vessel is supported on the concrete fill, which transfers the loads by bearing to the foundation mat below. To assure proper contact between the containment and the concrete fill, the interface is grouted with epoxy between the concrete fill and the bottom of the vessel through channels left by pulled tubing.

The primary shield wall, secondary shield wall and refueling canal of the internal structure, as described in Subsection 3.8.3 are also founded on concrete fill which transfers the loads by bearing to the foundation mat. For the Reactor Auxiliary, Fuel Handling Building and Component Cooling Water System Structures, as described in Subsection 3.8.4, load transfer to the common foundation mat is achieved through the walls and columns.

Waterproofing membranes are provided around the exterior surfaces of the walls and mat, from elevation -37.0 ft. MSL to the finished plant grade (varying from elevation +14.5 ft. MSL to +17.5 ft. MSL). Double water stops are provided at the construction joints in the foundation mat, while single water stops for vertical construction joints in the exterior walls are provided up to elevation +30 ft MSL (see Section 3.4). In addition, stainless steel plates are provided at the bottom of the mat, where sumps and manholes are located. For further details refer to Subsection 3.4.1.

Piling or similar foundation concepts are not used.

3.8.5.2 Applicable Codes, Standards and Specifications

The applicable codes, standards and specifications are as given in Subsection 3.8.3.2.

3.8.5.3 Loads and Loading Combinations

The common foundation mat is subjected to loads transferred from the steel containment vessel, containment internal structure and other seismic Category I structures founded on it. The loads and loading combinations considered in the design of the foundation mat are as described in Subsection 3.8.3.3 and 3.8.4.3 recognizing the simultaneous application of loads from the superstructures in each loading combination.

3.8.5.4 Design and Analysis Procedures

The base slab was analyzed as a rectangular flat slab resting on an elastic foundation.

3.8.5.4.1 Analytical Techniques

3.8.5.4.1.1 Static Analysis

The static analysis of the common mat was performed using MRI/STARDYNE 2 computer program. The structure was represented by an assembly of 643 beams, 2393 plates and 1087 nodes. The foundation soil was represented by linear springs at every node in the mat. The finite element model was designed to closely represent each part of structure rigidity together with load distribution, in order that the stress and deformation of the mat could be analyzed more accurately. Model simplification was made where minor carry-over effects existed. Structure walls which are directly supported by the mat, and floor slab systems which are supported by the column and beam frame systems on the mat were modeled in detail with little or no simplification.

The technique of utilizing the effective foundation springs, rather than the actual soil modulus of elasticity, was used to represent the structural foundation support since the long term effects of consolidation and settlement were considered. The initial subgrade modulus was calculated utilizing the elastic stress-strain characteristics from laboratory tests of the various soils as well as the geometry of the structure. The modulus was then adjusted to lower values in an iterative process based upon the results of bearing pressures and foundation settlement characteristics, as discussed in Subsection 2.5.4.

3.8.5.4.1.2 Dynamic Analysis

The dynamic analysis of the nuclear plant island structure including the foundation mat has been described in Section 3.7.

By varying the magnitude of soil shear modulus in the dynamic analysis, the maximum structure loads were established and used in the mat design. The maximum structure and soil displacements resulting from the dynamic analysis were used to calculate the earthquake soil pressures used in the mat stress analysis.

3.8.5.4.2 Design Procedures

The mat foundation has been designed such that the resulting soil bearing pressure are within the allowable limits as discussed in Subsection 2.5.4.

In order to insure uniform settlement and uniform pore pressure distribution under the base of the foundation structure, selected filter material (clam shell) is placed under the mat with a compacted thickness of 12 in. The degree of compaction was determined and verified by means of field tests.

The results of the analyses indicate that the normal operating and SSE govern the mat design which was determined after checking all other loading conditions as listed in Subsections 3.8.3.3 and 3.8.4.3. A typical reinforcement of the mat foundation is shown in Figure 3.8-46.

3.8.5.4.3 Design and Construction Coordination

The implementation of the design-construction condition was studied carefully to eliminate any overstress of the subsoil and to maintain mat stability from differential settlement and tilting. Each construction stage was established to meet the requirements of the net and the allowable differential soil bearing pressures. The excavation, concrete and backfill sequencing as well as the effects of dewatering and recharging of groundwater, were planned. In addition, subsurface and structure instrumentation were installed to monitor the subsoils behavior during construction as described in Subsection 3.8.5.7.

3.8.5.4.4 Mechanism for Load Transfer

Load transfer from seismic Category I structures to the foundation mat is achieved through conventional wall-to-mat connection. Load transfer from seismic Category I equipment to the foundation mat is achieved through supporting structural media such as concrete pedestals, floor slabs and concrete walls.

The effects of loads and soil reaction on foundation mat produced by the loading combinations referenced in Subsection 3.8.5.3 are obtained by following the design and analysis procedures described in Subsections 3.8.3.4 and 3.8.4.4. The mechanism for load transfer to the soil materials is discussed in Subsections 2.5.4.7, 2.5.4.10 and 2.5-4.11.

3.8.5.5 Structural Acceptance Criteria

The structural acceptance criteria for the seismic Category I foundation mat is discussed in Subsection 3.8.3.5.1.

In addition adequate margins of safety are provided for the foundation materials against shear failure, overturning and other parameters that identify the foundation behaviors under loads as discussed in Subsection 2.5.4. The comparison of calculated stresses, strains and deformations with the allowable limits specified in Subsection 2.5.4, is provided in Table 3.8-43.

3.8.5.6 Materials, Quality Control And Special Construction Techniques

For details of applicable material specifications, quality control provisions and any special construction techniques for the seismic Category I concrete foundation refer to Subsection 3.8.3.6 and 3.8.4.6. The required 28 day design strength for the common foundation mat is 4000 psi.

3.8.5.7 Testing And Inservice Surveillance Requirements

In order to monitor the heave and recompression settlement of the mat foundation during construction, detailed instrumentation was installed prior to start of excavation. The results and analyses are presented in Subsection 2.5.4.13. Additionally, Subsection 2.5.4.13.4 contains a description of the post-construction crack surveillance program.

SECTION 3.8: REFERENCES

1) "Theory of Turbulent Jets", G.N. Abramovich, MIT Press. 1963.

2) Local Stresses in Spherical and Cylindrical Shells Due to Exterior Loading, Welding Research Council 107, December 1968.

SECTION 3.8: REFERENCES (Cont'd)

3) Analysis of Shells of Revolution Subjected to Symmetrical and NonSymmetrical Loads, Kalnins, Arthur; ASME Journal of Applied Mechanics, Volume 31, Series E, No. 3 pages 467-476.

4) Timoshenko, Woinowsky and Krieger, Theory of Plates and Shells, 2nd Edition, McGraw Hill.

5) P. P. Bijlaard, Stresses from Radial Loads and External Moments in Cylindrical Pressure Vessels, Pressure Vessel and Piping Design Collected Papers 1927-1959, The American Society of Mechanical Engineers, 1960, page 581, Figures 3 and 7.

6) Grinter, Theory of Modern Steel Structures, Volume II, page 259, Revised Edition, 1949, The Macmillan Company.

7) E. Reissner, "A New Derivation of the Equations of the Deformation of Elastic Shells," American Journal of Mathematics, 63, 1941, pp. 177-184.

8) Biaxial Stress Criteria for Low Pressure Tanks, Welding Research Council 69, June 1961.

9) E. 0. Bergman, The New Type Code Chart for the Design of Vessels Under External Pressure, Pressure Vessel and Piping Design Collected Papers 1927-1959, The American Society of Mechanical Engineers, 1960, page 647.

TABLE 3.8-1 (Sheet 1 of 2)

CONTAINMENT VESSEL PENETRATIONS - LOAD COMBINATIONS AND STRESS LIMITS

Load Combination	Temperature Condition	Stress Limits as Per ASME Code (Section III)
	Type I Penetration	
Pc + Pa + EPT + W	Tc, Ta	Fig. NE-3221-1
Pcl + Pal +EPT + W	Tcl, Tal	Fig. NE-3221-1
PC + PA + TT + 2EPT + W	Te, Ta	NB-3222.2 NE (for Sm)
Pcl + Pal + TT + 2EPT + W	Tcl, Tal	NB-3222.2, NE (for SM)
Pcl + Pal + EEPT + W	Tcl, Tal	NE-3131(c)
Pc + Pa + PRT + EEPT + W	TC, Ta	NE-3131.2
	Type II Penetration	
Pd + Pc + Pa + EPA + EPM +W	Td, Tc, Ta	Fig. NE-3221-1
Pd + PC + Pa + EPT + EPTo + W	Td, Tc, Ta	Fig. NE-3221-1
Po + Pcl + Pal + EpA + EPM + W	To, Tcl, Tal	Fig. NE-3221-1
Po + Pcl + Pal + EPT + EPTo + W	To, Tcl, Tal	Fig. NE-3221-1
Po + Pc + Pa + TA + TM + 2EPA + 2 EPM + W	To, Tc, Ta	NB-3222.2, NE (for Sm)
Po + Pc + Pa + TT + TTo + 2 EPT + 2EPTo + W	To, Tc, Ta	NB-3222.2, NE (for Sm)
Po + Pcl + Pal + TA + TM +2EPA + 2EPM + W	To, Tcl, Tal	NB-3222.2 NE (for Sm)
Po + Pcl + Pal + TT + TTo + 2EPT + 2EPTo + W	Toc, Tcl, Tal	NB-3222.2, NE (for Sm)
Po + Pcl + Pal + EEPA + EEPM + W	To, Tcl, Tal	NE-3131(c)
Po + Pcl + Pal + EEPTo + W	To, Tcl, Tal	NE-3131(c)
	Type III Penetration	
Pg + Pc + Pa + EPA + EPM + W	Tg, Tc, Ta	Fig. NE-3221-1
Pg + Pc + Pa + EPT + EPTo + (120 in.) (EPT) + W	Tg, Tc, Ta	Fig. NE-3221-1
Pgl + Pcl + Pal + EPA + EPM + W	Tgl, Tcl, Tal	Fig. NE-3221-1
Pgl + Pcl + Pal + EPT + EPto + (120 in.) (EPT) + W	Tgl, Tcl, Tal	Fig. NE-3221-1
Pg + Pc + Pa + TA + TM + 2EPA + 2EPM + W	Tg, Tc, Ta	NB-3222.2, NE (for Sm)
Pg + Pc + Pa + TT + TTo + 2EPT +2EPTo + (120 in.)x(TT + 2EPT) + W	Tg, Tc, Ta	NB-3222.2, NE (for Sm)
Pgl + Pcl + Pal + TA + TM + 2EPA + 2EPM + W	Tgl, Tcl, Tal	NB-3222.2, NE (for Sm)

TABLE 3.8-1 (Sheet 2 of 2)

CONTAINMENT VESSEL PENETRATIONS - LOAD COMBINATIONS AND STRESS LIMITS

Load Combination	Temperature Condition	Stress Limits as Per ASME Code (Section III)
	Type III Penetration	
Pgl + Pcl + Pal + TT + TTo + 2EPT + 2EPTo + (120 in.)x(TT + 2EPT + W	Tgl, Tcl, Tal	NB-3222.2, NE (for Sm)
Pgl + Pcl + Pal + EEPA + EEPM + W	Tgl, Tcl, Tal	NE-3131(c)
Pgl + Pcl + Pal + EEPA + EEPM + EEPTo + (120 in.)x(EEPT) + W	Tgl, Tcl, Tal	NE-3131(c)
Pg + Pc + Pa + PRA + PRM + W + EEPA + EEPM	Tg, Tc, Ta	NE-3131.2
Pg + Pc + Pa + PRT + PRTo + (120 in.) x(PRT + EEPT) + EEPT + EEPTo + W	Tg, Tc, Ta	NE-3131.2
Pgr + Rc + Pa + PRT + EEPA + EEPM + W	Tgr, Tc, Ta	NE-3131.2
Pgr + Pc + Pa + PRT + EEPT + EEPTo + (120 in.) x (EEPT) + W	Tgr, Tc, Ta	NE-3131.2
Pgr + Pc + Pa + PRT + EEPT + EEPM + W	Tgr, Tc, Ta	NE-3131.2
Pg + Pc + Pa + PRT + PRM + EEPT + EEPM + W	Tg, Tc, Ta	NE-3131.2
Pg + Pc + Pa + EPT + EPM + W	Tg, Tc, Ta	Fig. NE-3221-1
Pgl + Pcl + Pal + EPT + EPM + W	Tgl, Tcl, Tal	Fig. NE-3221-1
Pgl + Pcl + Pal + EEPT + EEPM + W	Tgl, Tcl, Tal	NE-3131(c)

TABLE 3.8-2

CONTAINMENT VESSEL LOAD COMBINATIONS

LOAD COMBINATIONS		CASE 1	CASE 2	CASE 3	CASE 4	CASE 5	CASE 6	CASE 7	CASE 8	CASE 9	CASE 10
PRESSURE	INTERNAL	-	49.5	44	-	-	39.6	39.6*	39.6	39.6	-
	EXTERNAL	-	-	-	.65	.65	-	-	-	-	
DEAD LOAD OF VESSEL & APPURTENANCES		X	X	X	X	X	X	X	X	X	X
CONTAINED AIR @ TEST			X	X							
DEAD LOAD OF ATTACHMENTS		X	X	X	X	X	X	X	X	X	X
DEAD LOAD OF INSULATION		X									
CRANE	LIVE					X					
	DEAD			X	X	X	X	X	X	X	X
LATERAL LOAD DUE TO WIND		X	X								
LATERAL DUE TO EARTHQUAKE	OPERATING	X	X	X	X		X				
	SAFE SHUTDOWN							X		X	X
VERTICAL LOAD DUE TO EARTHQUAKE	OPERATING			X	X	X	X		X		
	SAFE SHUTDOWN							X		X	X
LIVE LOAD ON PERSONNEL LOCK					X	X					
LIVE LOAD ON ESCAPE LOCK					X	X					
LIVE LOAD ON CONSTRUCTION & MAINTENANCE HATCH COMBINATION						X					
LIVE LOAD ON MAINTENANCE HATCH						X					
LIVE LOAD ON PLATFORMS		X									
PIPE THERMAL LOADS					X				X	X	
PIPE SEISMIC LOADS					X	X			X	X	
PIPE RUPTURE LOADS									X	X	
TEMPORARY ROOF LOADS											X

* Design internal pressure as defined by 1971 Edition of ASME III Subsection NE

TABLE 3.8-3

CONTAINMENT SHELL STRESSES AT JUNCTION

OF COLUMN AND KNUCKLE

CONDITIONS	LOAD COMBINATIONS
I. P.W.H.T.	1) Vertical Bending Due to Column Loads 2) Horizontal Bending Due to Wind Shear and Column Eccentricity. 3) Torsion a) Due To A-Frame Eccentricity b) Due to A-Frame Pin Connections
II. Construction State (with Concrete in Bottom Head & Wind Load)	1) Vertical Bending Due to Column Loads 2) Horizontal Bending Due to Wind Shear and Column Eccentricity 3) Torsion Due To A-Frame Eccentricity
III. Construction State (with Concrete in Bottom Head & Hard Point Loads)	1) Vertical Bending Due To Column Loads 2) Horizontal Bending Due To Column Eccentricity 3) Torsion Due To A-Frame Eccentricity
IV. During Final Test	1) Vertical Bending Due To Column Loads 2) Horizontal Bending Due To Wind Shear and Column Eccentricity 3) Torsion Due To A-Frame Eccentricity 4) Internal Pressure

TABLE 3.8-4

CONTAINMENT VESSEL TEMPERATURE GRADIENT

Time after Loca	Distance (ft) Below Top of Containment Floor (Elevation -1.5 ft. MSL) Measured Along Shell							
	0.0	0.5	1.0	1.5	2.0	2.5	3.0	4.0
5 min	180	123	120	120	120	120	120	120
10 min	195	131	120	120	120	120	120	120
20 min	207	147	124	120	120	120	120	120
30 min	211	158	133	122	120	120	120	120
40 min	215	166	136	124	121	120	120	120
50 min	218	172	146	127	122	120	120	120
1 hr	219	177	147	130	123	121	120	120
2 hr	223	192	166	146	133	126	123	120
3 hr	221	198	176	153	143	135	127	122
5 hr	211	197	184	168	155	145	137	127
10 hr	197	189	183	173	165	157	150	138
15 hr	187	183	179	172	167	161	155	145
24 hr	173	171	169	166	163	160	156	146

TABLE 3.8-5

PENETRATION ANALYSIS

Style or Reinforcement	Location of Analysis	Reference for Curves	Radial M Parameter (1) stress	Radial M % Increment	Tangential M Parameters stress	Tangential M % Increment	Radial N Parameter stress	Radial N % Increment	Tangential N Parameter stress	Tangential N % Increment	Comments
Insert	Insert to Neck	WRC 107	Tins/Tins	20	Tins/Tins	20	Ts/Tins	-	Ts/Tins	-	
(width < 1.65 $\sqrt{RTins}$	1/2 $\sqrt{RTins}$ from local stress	WRC 107	Tins/Tins	20	Tins/Tins	20	Ts/Tins	-	Ts/Tins	-	See Note 3
	Insert to Shell	WRC 107	Ts/Tins	-	Ts/Tins	-	Ts/Tins	-	Ts/Tins	-	See Note 4
Pad	Pad to Neck	WRC 107	Tp+s/Tp+s	20	Tp+s/Tp+s	20	Ts/Tp+s	-	Ts/Tp+s	-	
(width <165 $\sqrt{RTeq}$	1/2 $\sqrt{RTs}$ from local stress	WRC 107	Tp+s/Tp+s	20	Tp+s/Tp+s	20	Ts/Tp+s	-	Ts/Tp+s	-	See Note 3
	Edge of Pad	WRC 107	Ts/Ts	-	Ts/Ts	-	Ts/Ts	-	Ts/Ts	-	See Note 4
Shell	Neck to Shell	WRC 107	Ts/Ts	-	Ts/Ts	-	Ts/Ts	-	Ts/Ts	-	
(No reinforcement width ≥ 1.65 $\sqrt{RT\ reinf}$	1/2 $\sqrt{RTs}$ from Neck	WRC 107	Ts/Ts	-	Ts/Ts	-	Ts/Ts	-	Ts/Ts	-	See Note 3

Notes:
(1) Indicates thickness for calculation of parameter and stresses.
(2) Any alternate method may be used for a location of analysis in combination with the above method for other locaitons.
(3) Check only if stresses at other location > 1.1 Sm. Use Ts and no increase in stresses if outside reinforcing.
(4) Stresses due to loads may be reduced in accordance with WRC 95.

Nomenclature:

Tins = thickness of insert
Teq = equivalent thickness
Tp+s = thickness of pad plus shell
Ts = thickness of shell

TABLE 3.8-6 (Sheet 1 of 3)

ALLOWABLE STRESS

CASE 1 [(a)]-CONSTRUCTION AT PWHT

No ASME design (Shell is analyzed using methods consistent with the ASME Code)

AISC design-AISC allowables

CASE 2-ACCEPTANCE TEST AT AMBIENT TEMPERATURE

ASME design

$PM \leq 1.25\ (1.0\ Sm)$
$PL + PB \leq 1.25\ (1.5\ Sm)$
$PL + PB + Q \leq 3.0\ Sm$

A 25 percent increase is also taken for buckling

AISC design-AISC allowables

CASE 3-PRE-OPERATION TEST AT AMBIENT TEMPERATURE

ASME Design

$PM \leq 1.1\ (1.0\ Sm)$
$PL + PB \leq 1.1\ (1.5\ Sm)$
$PL + PB + Q \leq 3.0\ Sm$

A 10 percent increase is also taken for buckling

AISC design-AISC allowables

CASE 4 - NORMAL OPERATING CONDITION AT TEMPERATURE RANGE OF 30 F TO 150 F

ASME design

$PM \leq 1.0\ Sm$
$PL + PB \leq 1.5\ Sm$
$PL + PB + Q \leq 3.0\ Sm$

AISC design-AISC allowables

CASE 5-COLD SHUTDOWN AT TEMPERATURE RANGE OF 30 F TO 120 F

ASME design with operating basis earthquake

$PM \leq 1.0\ Sm$ (Includes seismic stress)
$PL + PB \leq 1.5\ Sm$
$PL + PB + Q \leq 3.0\ Sm$

AISC design with operating basis earthquake-AISC allowables with normal increase

TABLE 3.8-6 (Sheet 2 of 3)

ALLOWABLE STRESS

CASE 6-ACCIDENT CONDITION WITH OPERATING BASIS EARTHQUAKE

ASME DESIGN WITH OPERATING BASIS EARTHQUAKE

$PM \leq 1.0\ Sm$ (Includes seismic stress)
$PL + PB \leq 1.5\ Sm$
$PL + PB + Q \leq 3.0\ Sm$

AISC design with operating basis earthquake-AISC allowables with normal increase

CASE 7-ACCIDENT CONDITION WITH SAFE SHUTDOWN EARTHQUAKE

ASME design with safe shutdown earthquake

$PM \leq 0.9\ Sy$ (Includes seismic stress)
$PL + PB \leq 0.9\ Su$

AISC design with safe shutdown earthquake-AISC allowables with normal increase

CASE 8-ACCIDENT CONDITION WITH OPERATING BASIS EARTHQUAKE PLUS PIPE LOADS

$PM \leq 1.0\ Sm$ (Includes seismic stress)
$PL + PB \leq 1.5\ Sm$
$PL + PB + Q \leq 3.0\ Sm$

AISC design with operating basis earthquake-AISC allowables with normal increase

Note: Pipe rupture loads are investigated as a local effect separately:

$PM \leq 0.9\ Sy$
$PL + PB \leq 1.2\ Sy$

TABLE 3.8-6 (Sheet 3 of 3)

ALLOWABLE STRESS

CASE 9-ACCIDENT CONDITION WITH SAFE SHUTDOWN EARTHQUAKE PLUS PIPE LOADS

ASME design with safe shutdown earthquake

$PM \leq 0.9\ Sy$ (Includes seismic stress)
$PL + PB \leq 0.9\ Su$

AISC design with safe shutdown earthquake-AISC allowables with normal increase

Note: Pipe rupture loads are investigated as a local effect separately:

$PM \leq 0.9\ Sy$
$PL + PB \leq 1.2\ Sy$

CASE 10-TEMPORARY ROOF CONSTRUCTION LOADS WITH SAFE SHUTDOWN EARTHQUAKE

ASME design with safe shutdown earthquake

$PM \leq 0.9\ Sy$ (Includes seismic stress)
$PL + PB \leq 0.9\ Su$

AISC design with safe shutdown earthquake-AISC allowables with normal increase

(a) Refer to Subsection 3.8.2.3 for a description of each load case or combination.

General Note:

ASME Code is used in the design of the steel shell and its penetrations. AISC refers to all other steel structures interacting with the containment vessel, such as crane girders, platforms and temporary supports. Seismic effects on the personnel locks and equipment hatches are investigated separately using ASME Code allowables. AISC allowables with normal increase is permitted by Section 1.5.6 of the AISC Code (33.3 percent increase in allowable for seismic stresses.)

TABLE 3.8-7

SUMMARY OF CONTAINMENT VESSEL STRESSES FOR LOAD CASE 1

MERIDIONAL STRESS σϕ (psi)

	POINT										
LOAD	1	2	3	4	5	6	7	8	9	10[a]	11[a]
SHELL WT	-558	-520	-468	-422	-379	-303	-244	-187	-237	-176	-139
ATTACHMENTS	-215	-204	-190	-170	-166	-134	-69	-24	-37	-42	-53
WIND	-366	-328	-276	-234	-107	-138	-99	-66	-77	-30	-10
PRESSURE											
INSULATION	-23	-22	-20	-19	-17	-15	-13	-11	-17	-12	-10
CRANE L.L.[b]											
PERSONNEL LOCK L.L.											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC											
VERT. SEISMIC											
WALKWAY L.L.	-5	-5	-5	-5	-5	-5	-5	-5	-9	-11	-19
CONST. HATCH L.L.											
MAIN HATCH L.L.											
σϕ TOTAL	-1167	-1079	-959	-850	-764	-595	-430	-292	-377	-271	-231

CIRCUMFERENTIAL STRESS σϕ(psi)

	POINT										
LOAD	1	2	3	4	5	6	7	8	9	10[a]	11[a]
SHELL WT	0	0	0	0	0	0	0	0	237	91	-32
ATTACHMENTS	0	0	0	0	0	0	0	0	37	42	53
WIND	0	92	92	123	123	154	153	184	272	314	248
PRESSURE											
INSULATION	0	0	0	0	0	0	0	0	17	12	10
CRANE L.L.											
PERSONNEL LOCK L.L.											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC											
VERT. SEISMIC											
WALKWAY L.L.	0	0	0	0	0	0	0	0	9	11	19
CONST. HATCH L.L.											
MAIN. HATCH L.L.											
σϕ TOTAL	0	92	92	123	123	154	153	184	572	470	298

Buckling Allowables Cylinder σϕ = 3280 psi σϕ = 300 psi
Head σϕ (compressive) = 820 psi

(a) These points were added into the stress calculations even though the dome was not included in the PWHT. By adding these two points, the stresses calculated for the other locations are conservative.

(b) L.L. = live load

TABLE 3.8-8

SUMMARY OF CONTAINMENT VESSEL STRESSES FOR LOAD CASE 2

MERIDIONAL STRESS σϕ (psi)

LOAD	POINT 1	2	3	4	5	6	7	8	9	10	11
SHELL WT	-558	-520	-468	-422	-379	-303	-244	-187	-237	-176	-139
ATTACHMENTS	-215	-204	-190	-170	-166	-134	-69	-24	-37	-42	-53
WIND	-366	-328	-276	-234	-197	-138	-99	-66	-77	-30	-10
PRESSURE	10925	10925	10925	10925	10925	10925	10925	10925	21880	21880	21880
INSULATION											
CRANE L.L.[(a)]											
PERSONNEL LOCK L.L.											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC											
VERT. SEISMIC											
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN HATCH L.L.											
σϕ TOTAL	9786	9873	9991	10099	10183	10350	10513	10648	21529	21632	21678

CIRCUMFERENTIAL STRESS σϕ(psi)

LOAD	POINT 1	2	3	4	5	6	7	8	9	10	11
SHELL WT	0	0	0	0	0	0	0	0	237	91	-32
ATTACHMENTS	0	0	0	0	0	0	0	0	37	42	53
WIND	0	92	92	123	124	154	153	184	272	314	248
PRESSURE	21850	21850	21850	21850	21850	21850	21850	21850	21880	21880	21880
INSULATION											
CRANE L.L.											
PERSONNEL LOCK L.L.											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC											
VERT. SEISMIC											
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN. HATCH L.L.											
σϕ TOTAL	21850	21942	21942	21973	21974	22004	22003	22034	22426	22327	22149

Allowable Stress intensity = 1.25 x S_m = 21875 psi

(a) L.L. = live load

TABLE 3.8-9

SUMMARY OF CONTAINMENT VESSEL STRESSES FOR LOAD CASE 3

MERIDIONAL STRESS σφ (psi)

LOAD	POINT 1	2	3	4	5	6	7	8	9	10	11
SHELL WT	-558	-520	-468	-422	-379	-303	-244	-187	-237	-176	-139
ATTACHMENTS	-215	-204	-190	-170	-166	-134	-69	-24	-37	-42	-53
WIND											
PRESSURE (44 psi)	9710	9710	9710	9710	9710	9710	9710	9710	19452	19452	19452
INSULATION											
CRANE											
PERSONNEL LOCK L.L.[a]											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC	-465	-411	-340	-283	-232	-153	-101	-68	-83	-39	0
VERT. SEISMIC	-59	-53	-49	-44	-41	-34	-28	-17	-24	-18	-20
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN HATCH L.L.											
σφ TOTAL	8413	8522	8663	8791	8892	9086	9268	9414	19071	19177	19240

CIRCUMFERENTIAL STRESS σφ(psi)

LOAD	POINT 1	2	3	4	5	6	7	8	9	10	11
SHELL WT	0	0	0	0	0	0	0	0	237	91	-32
ATTACHMENTS	0	0	0	0	0	0	0	0	37	42	53
WIND											
PRESSURE (44 psi)	19420	19420	19420	19420	19420	19420	19420	19420	19452	19452	19452
INSULATION											
CRANE											
PERSONNEL LOCK L.L.											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC	0	0	0	0	0	0	0	0	82	39	0
VERT. SEISMIC	0	0	0	0	0	0	0	0	24	18	20
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN. HATCH L.L.											
σφ TOTAL	19420	19420	19420	19420	19420	19420	19420	19420	19832	19642	19493

Allowable stress intensity = 1.1 x S_m = 19250 psi

(a) L.L. = live load

TABLE 3.8-10

SUMMARY OF CONTAINMENT VESSEL STRESSES FOR LOAD CASE 4

MERIDIONAL STRESS σϕ (psi)

	POINT										
LOAD	1	2	3	4	5	6	7	8	9	10	11
SHELL WT	-558	-520	-468	-422	-379	-303	-244	-187	-237	-176	-139
ATTACHMENTS	-215	-204	-190	-170	-166	-134	-69	-24	-37	-42	-53
WIND											
PRESSURE (.65 psi)	-144	-144	-144	-144	-144	-144	-144	-144	-288	-288	-288
INSULATION											
CRANE L.L.											
PERSONNEL LOCK L.L.[a]	-1	0	0	0	0	0	0	0	0	0	0
ESCAPE LOCK L.L.	-1	-1	0	0	0	0	0	0	0	0	0
HORIZ. SEISMIC	-465	-411	-340	-283	-232	-153	-101	-68	-83	-39	0
VERT. SEISMIC	-59	-53	-49	-44	-41	-39	-28	-17	-24	-18	-20
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN HATCH L.L.											
σϕ TOTAL	-1443	-1333	-1191	-1063	-962	-773	-586	-440	-669	-563	-500

CIRCUMFERENTIAL STRESS σϕ(psi)

	POINT										
LOAD	1	2	3	4	5	6	7	8	9	10	11
SHELL WT	0	0	0	0	0	0	0	0	237	91	-32
ATTACHMENTS	0	0	0	0	0	0	0	0	37	42	53
WIND											
PRESSURE (.65 psi)	-288	-288	-288	-288	-288	-288	-288	-288	-288	-288	-288
INSULATION											
CRANE L.L.											
PERSONNEL LOCK L.L.	0	0	0	0	0	0	0	0	0	0	0
ESCAPE LOCK L.L.	0	0	0	0	0	0	0	0	0	0	0
HORIZ. SEISMIC	0	0	0	0	0	0	0	0	83	39	0
VERT. SEISMIC	0	0	0	0	0	0	0	0	24	18	20
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN. HATCH L.L.											
σϕ TOTAL	-288	-288	-288	-288	-288	-288	-288	-288	93	-98	-247

Bucking Allowables Cylinder	σϕ =	4200 psi	σθ = 570 psi
Head	σϕ + σθ =	2036 psi	

(a) L.L. = live load

TABLE 3.8-11

SUMMARY OF CONTAINMENT VESSEL STRESSES FOR LOAD CASE 5

MERIDIONAL STRESS σϕ (psi)

	POINT										
LOAD	1	2	3	4	5	6	7	8	9	10	11
SHELL WT	-558	-520	-468	-422	-379	-303	-244	-187	-237	-176	-139
ATTACHMENTS	-215	-204	-190	-170	-166	-134	-69	-24	-37	-42	-53
WIND											
PRESSURE (.65 psi)	-144	-144	-144	-144	-144	-144	-144	-144	-288	-288	-288
INSULATION											
CRANE L.L.[(a)]	-53	-53	-53	-53	-53	-53	-27	0	0	0	0
PERSONNEL LOCK L.L.	-1	0	0	0	0	0	0	0	0	0	0
ESCAPE LOCK L.L.	-1	-1	0	0	0	0	0	0	0	0	0
HORIZ. SEISMIC	-465	-411	-340	-283	-232	-153	-101	-68	-83	-39	0
VERT. SEISMIC	-59	-53	-49	-44	-41	-39	-28	-17	-24	-18	-20
WALKWAY L.L.											
CONST. HATCH L.L.	-2	-2	-2	-2	-2	0	0	0	0	0	0
MAIN HATCH L.L.	-1	-1	0	0	0	0	0	0	0	0	0
σϕ TOTAL	-1499	-1389	-1246	-1118	-1017	-826	-613	-440	-669	-563	-500

CIRCUMFERENTIAL STRESS σϕ(psi)

	POINT										
LOAD	1	2	3	4	5	6	7	8	9	10	11
SHELL WT	0	0	0	0	0	0	0	0	237	91	-32
ATTACHMENTS	0	0	0	0	0	0	0	0	37	42	53
WIND											
PRESSURE (.65 psi)	-288	-288	-288	-288	-288	-288	-288	-288	-288	-288	-288
INSULATION											
CRANE L.L.	0	0	0	0	0	0	0	0	0	0	0
PERSONNEL LOCK L.L.	0	0	0	0	0	0	0	0	0	0	0
ESCAPE LOCK L.L.	0	0	0	0	0	0	0	0	0	0	0
HORIZ. SEISMIC	0	0	0	0	0	0	0	0	83	39	0
VERT. SEISMIC	0	0	0	0	0	0	0	0	24	18	20
WALKWAY L.L.											
CONST. HATCH L.L.	0	0	0	0	0	0	0	0	0	0	0
MAIN. HATCH L.L.	0	0	0	0	0	0	0	0	0	0	0
σϕ TOTAL	-288	-288	-288	-288	-288	-288	-288	-288	93	-98	-247

Bucking Allowables Cylinder	σϕ =	4200 psi	σθ = 570 psi
Head	σϕ + σθ =	2036 psi	

(a) L.L. = live load

TABLE 3.8-12

SUMMARY OF CONTAINMENT VESSEL STRESSES FOR LOAD CASE 6

MERIDIONAL STRESS σϕ (psi)

	POINT										
LOAD	1	2	3	4	5	6	7	8	9	10	11
SHELL WT	-558	-520	-468	-422	-379	-303	-244	-187	-237	-176	-139
ATTACHMENTS	-215	-204	-190	-170	-166	-134	-69	-24	-37	-42	-53
WIND											
PRESSURE (39.6 psi)	8740	8740	8740	8740	8740	8740	8740	8740	17510	17510	17510
INSULATION											
CRANE L.L.[(a)]											
PERSONNEL LOCK L.L.											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC	-465	-411	-340	-283	-232	-153	-101	-68	-83	-39	0
VERT. SEISMIC	-59	-53	-49	-44	-41	-39	-28	-17	-24	-18	-20
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN HATCH L.L.											
σϕ TOTAL	7433	7552	7693	7821	7922	8111	8298	8450	17129	17235	17298

CIRCUMFERENTIAL STRESS σϕ(psi)

	POINT										
LOAD	1	2	3	4	5	6	7	8	9	10	11
SHELL WT	0	0	0	0	0	0	0	0	237	91	-32
ATTACHMENTS	0	0	0	0	0	0	0	0	37	42	53
WIND											
PRESSURE (39.6 psi)	17480	17480	17480	17480	17480	17480	17480	17480	17510	17510	17510
INSULATION											
CRANE L.L.											
PERSONNEL LOCK L.L.											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC	0	0	0	0	0	0	0	0	83	39	0
VERT. SEISMIC	0	0	0	0	0	0	0	0	24	18	20
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN. HATCH L.L.											
σθ TOTAL	17480	17480	17480	17480	17480	17480	17480	17480	17891	17700	17551

Allowable stress instensity = Sm = 17500 psi

(a) L.L. = live load

TABLE 3.8-13

SUMMARY OF CONTAINMENT VESSEL STRESSES FOR LOAD CASE 7

MERIDIONAL STRESS $\sigma\phi$ (psi)

LOAD	POINT 1	2	3	4	5	6	7	8	9	10	11
SHELL WT	-558	-520	-468	-422	-379	-303	-244	-187	-237	-176	-139
ATTACHMENTS	-215	-204	-190	-170	-166	-134	-69	-24	-37	-42	-53
WIND											
PRESSURE (39.6 psi)	8740	8740	8740	8740	8740	8740	8740	8740	17510	17510	17510
INSULATION											
CRANE L.L.[(a)]											
PERSONNEL LOCK L.L.											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC	-929	-822	-565	-463	-305	-201	-135	-165	-77	-39	0
VERT. SEISMIC	-118	-106	-97	-88	-81	-68	-56	-33	-48	-36	-39
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN HATCH L.L.											
$\sigma\phi$ TOTAL	6920	7088	7305	7495	7651	7930	8170	8361	17023	17179	17279

CIRCUMFERENTIAL STRESS $\sigma\theta$(psi)

LOAD	POINT 1	2	3	4	5	6	7	8	9	10	11
SHELL WT	0	0	0	0	0	0	0	0	237	91	-32
ATTACHMENTS	0	0	0	0	0	0	0	0	37	42	53
WIND											
PRESSURE (39.6 psi)	17480	17480	17480	17480	17480	17480	17480	17480	17510	17510	17510
INSULATION											
CRANE L.L.											
PERSONNEL LOCK L.L.											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC	0	0	0	0	0	0	0	0	167	77	0
VERT. SEISMIC	0	0	0	0	0	0	0	0	48	36	39
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN. HATCH L.L.											
$\sigma\theta$ TOTAL	17480	17480	17480	17480	17480	17480	17480	17480	17999	17756	17570

Allowable stress instensity = 0.9 Sy = 30300 psi

(a) L.L. = live load

TABLE 3.8-14

SUMMARY OF CONTAINMENT VESSEL STRESSES FOR LOAD CASE 10

MERIDIONAL STRESS σϕ (psi)

	POINT										
LOAD	1	2	3	4	5	6	7	8	9	10	11
SHELL WT	-558	-520	-468	-422	-379	-303	-244	-187	-237	-176	-139
ATTACHMENTS	-215	-204	-190	-170	-166	-134	-69	-24	-37	-42	-53
WIND	-366	-328	-276	-234	-197	-138	-99	-66	-77	-30	-10
PRESSURE											
INSULATION											
CRANE L.L.[(a)]											
PERSONNEL LOCK L.L.											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC	-929	-822	-680	-565	-463	-305	-201	-135	-165	-77	0
VERT. SEISMIC	-118	-106	-97	-88	-81	-68	-56	-33	-48	-36	-39
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN HATCH L.L.											
TEMP ROOF LOADS	-231	-231	-231	-231	-231	-231	-231	-231	-462	-530	-358
σϕ TOTAL	-2417	-2211	-1942	-1710	-1517	-1179	-900	-676	-1026	-891	-599

CIRCUMFERENTIAL STRESS σθ(psi)

	POINT										
LOAD	1	2	3	4	5	6	7	8	9	10	11
SHELL WT	0	0	0	0	0	0	0	0	237	91	-32
ATTACHMENTS	0	0	0	0	0	0	0	0	37	42	53
WIND											
PRESSURE											
INSULATION											
CRANE L.L.											
PERSONNEL LOCK L.L.											
ESCAPE LOCK L.L.											
HORIZ. SEISMIC	0	0	0	0	0	0	0	0	165	77	0
VERT. SEISMIC	0	0	0	0	0	0	0	0	48	36	39
WALKWAY L.L.											
CONST. HATCH L.L.											
MAIN. HATCH L.L.											
TEMP ROOF LOADS	0	0	0	0	0	0	0	0	462	530	358
σθ TOTAL	0	0	0	0	0	0	0	0	949	776	418

Buckling Allowables Cylinder σϕ = 4200 psiσθ = 570 psi
Head σϕ(compression only) = 2036 psi

(a) L.L. = live load

TABLE 3.8-15

ALLOWABLE BUCKLING STRESSES FOR CONTAINMENT VESSEL ELLIPSOIDAL HEAD

φ Normal Degrees	Plate Thickness (in.)	$^T/_{R_2}$	$^T/_{R_1}$	External Pressure σφ (PSI)	Internal Pressure σ (PSI)
90	2.1875	.00260	.01040	4,680	14,800
95	2.1875	.00259	.01032	4,662	14,700
100	2.1875	.00257	.01005	4,626	14,600
105	2.1875	.00254	.00963	4,572	14,400
110	2.1875	.00248	.00907	4,464	14,200
115	2.1875	.00242	.00839	4,356	13,800
120	2.1875	.00234	.00762	4,212	13,000
125	2.1875	.00226	.00680	4,068	12,200
130	2.1875	.00216	.00597	3,888	10,746
135	2.1875	.00206	.00514	3,708	9,252
140	2.1875	.00195	.00436	3,510	7,848
145	1.903	.001594	.003169	2,869	5,704
150	1.903	.001496	.002619	2,693	4,714
155	1.903	.001402	.002153	2,524	3,875
160	1.903	.001315	.001776	2,367	3,197
165	1.903	.001239	.001489	2,230	2,680
170	1.903	.001181	.001288	2,126	2,318
175	1.903	.001114	.001170	2,059	2,106
180	1.903	.001131	.001131	2,036	2,036

TABLE 3.8-16

ALLOWABLE BUCKLING STRESSES FOR CONTAINMENT VESSEL ELLIPSOIDAL HEAD
AT POST WELD HEAT TREATMENT

ϕ Normal Degrees	Plate Thickness (in.)	$\frac{T}{R_2}$	$\frac{T}{R_1}$	External Pressure $\sigma\phi$ (PSI)	Internal Pressure σ (PSI)
90	2.1875	.00260	.01040	1,880	4,600
95	2.1875	.00259	.01032	1,870	4,600
100	2.1875	.00260	.01005	1,860	4,600
105	2.1875	.00259	.00963	1,840	4,600
110	2.1875	.00257	.00907	1,790	4,600
115	2.1875	.00254	.00839	1,750	4,600
120	2.1875	.00248	.00762	1,690	4,600
125	2.1875	.00242	.00680	1,630	4,600
130	2.1875	.00234	.00597	1,560	4,320
135	2.1875	.00226	.00514	1,490	3,720
140	2.1875	.00195	.00436	1,410	3,160
145	1.903	.001594	.003169	1,150	2,300
150	1.903	.001496	.002619	1,080	1,900
155	1.903	.001402	.002153	1,020	1,560
160	1.903	.001315	.001776	953	1,290
165	1.903	.001239	.001489	900	1,080
170	1.903	.001181	.001288	860	930
175	1.903	.001114	.001170	830	850
180	1.903	.001131	.001131	820	820

TABLE 3.8-17

SUMMARY OF SHELL STRESSES IN KNUCKLE

BETWEEN COLUMNS

	σθ (PSI)						
	Location Degrees	Vertical Bending	Horizontal Bending	Torsion	Pressure	Total PSI	Allowable PSI
CONDITION I	95	-78	-55	-323	0	-456	-4600
	100	+34	-30	+140	0	+144	+5000[(a)]
	105	+148	-120	+613	0	+641	+5000[(a)]
	110	+267	-215	+1102	0	+1154	+5000[(a)]
	115	+389	-315	+1608	0	+1682	+5000[(a)]
	120	+515	-415	+2129	0	+2229	+5000[(a)]
CONDITION II	95	-94	-625	-74	0	-793	-14700
	100	+41	-500	+32	0	-427	-14600
	105	+179	-470	+140	0	-151	-14400
	110	+321	-515	+252	0	+58	+17500[(b)]
	115	+469	-560	+368	0	+277	+17500[(b)]
	120	+621	-605	+487	0	+503	+17500[(b)]
CONDITION III	95	-86	-455	-67	0	-608	-14700
	100	+37	-365	+29	0	-299	-14600
	105	+162	-355	+127	0	-66	-14000
	110	+292	-385	+229	0	+906	+17500[(b)]
	115	+426	+420	+334	0	+340	+17500[(b)]
	120	+564	-455	+442	0	+551	+17500[(b)]
CONDITION IV	95	-100	-652	-79	-10190	-11021	-18375[(c)]
	100	+43	-515	+34	-15260	-15698	-18250[(c)]
	105	+189	-492	+149	-18020	-18174	-18000[(c)]
	110	+340	-536	+268	-17590	-17518	-17750[(c)]
	115	+496	-581	+391	-16140	-15834	-17250[(c)]
	120	+656	-642	+517	-13640	-13109	-16250[(c)]

Notes:

(a) Allowable tension as per ASME Section VIII, Appendix P, Paragraph UA-500
(b) As per ASME: Sm allowable = 17,500 psi
(c) As per ASME: Allowable stress = 1.25 Sm

TABLE 3.8-18

SUMMARY OF SHELL STRESSES IN KNUCKLE

AT COLUMNS

	σθ (PSI)						
	Location Degrees	Vertical Bending	Horizontal Bending	Torsion	Pressure	Total PSI	Allowable PSI
CONDITION I	95	+156	-55	-323	0	-222	-4600
	100	-68	-30	+140	0	+42	+5000[a]
	105	-297	-120	+613	0	+196	+5000[a]
	110	-533	-215	+1102	0	+354	+5000[a]
	115	-777	-315	+1608	0	+516	+5000[a]
	120	-1029	-415	+2129	0	+685	+5000[a]
CONDITION II	95	+188	-625	-74	0	-511	-14700
	100	-82	-500	+32	0	-550	-14600
	105	-357	-470	+140	0	-687	-14400
	110	-642	-515	+252	0	-905	-14200
	115	-937	-560	+368	0	-1129	-13800
	120	-1241	-605	+487	0	-1359	-13000
CONDITION III	95	+171	-455	-67	0	-351	-14700
	100	-74	-365	+29	0	-410	-14600
	105	-325	-355	+127	0	-553	-14000
	110	-583	-385	+229	0	-739	-14200
	115	-851	-420	+334	0	-937	-13800
	120	-1127	-455	+442	0	-1140	-13000
CONDITION IV	95	+199	-652	-79	-10190	-10722	-18375[b]
	100	-86	-515	+34	-15260	-15827	-18250[b]
	105	-378	-492	+149	-18020	-18741	-18000[b]
	110	-679	-536	+268	-17590	-18531	-17750[b]
	115	-991	-581	+391	-16140	-17321	-17250[b]
	120	-1312	-642	+517	-13640	-15077	-16250[b]

Notes:

(a) Allowable tension as per ASME Section VIII, Appendix P, Paragraph UA-500

(b) As per ASME: Allowable stress intensity = 1.25 Sm = 21,875 psi

Table 3.8-19 (1 of 2)

PHYSICAL PROPERTIES FOR MATERIALS

TO BE USED FOR PRESSURE PARTS OR ATTACHMENT TO PRESSSURE PARTS

MATERIAL SPECIFICATION	Su MINIMUM ULTIMATE TENSILE (KSI)	Sy MINIMUM YIELD AT AMBIENT (KSI)	Sy MINIMUM YIELD AT 263 F (KSI)	Sm ASME CODE ALLOWABLE STRESS INTENSITY (KSI)
PLATE				
SA516 GR 70	70	38	33.7	17.5
SA240 Type 304	75	30	23.2	16.9
SB 168	80	35	31.5	18.97
PIPE				
SA333 GR 6	60	35	31.32	15
SB167	80	35	31.50	18.94
FORGINGS				
SA350 LF-2	70	36	———	17.5
SA182 F304	75	30	23.3	17
BOLTING				
SA193 B7 < 2-1/2"	125	105	———	25
SA194 G4 7	———	———		

Table 3.8-19 (2 of 2)

PHYSICAL PROPERTIES FOR MATERIALS

PHYSICAL PROPERTIES FOR MATERIALS TO BE USED FOR NON-PRESSURE PARTS

MATERIAL SPECIFICATION	MINIMUM ULTIMATE TENSILE (KSI)	MINIMUM YIELD AT AMBIENT (KSI)	MINIMUM YIELD AT 263 F (KSI)
A36	58	36	———
SA516 GR 70	70	38	33.7
A53 GR B	60	35	———
A106 GR B	60	35	31.2

TABLE 3.8-20 Revision 8 (5/96)

MAJOR EQUIPMENT LOADS FOR CONTAINMENT INTERNAL STRUCTURE

Equipment	No. of Pieces	Individual Weight (lbm)*	
Reactor Vessel	1	882,000	(Dry)
		2,152,000	(Normal Operation)
Steam Generator	2	1,246,000	(Dry)
		1,506,000	(Normal Operation)
Reactor Drain Tank	1	17,000	
Reactor Coolant Pump	4	226,260	
Regenerative Heat Exchanger	1	4,000	
Pressurizer	1	203,000	(Dry)
		233,000	(Normal Operation)
Safety Injection Tank	4	99,000	(Dry)
		239,000	(Full)
Quench Tank	1	27,000	
Fuel Transfer Unit	1	10,000	

*rounded off to the nearest thousand

TABLE 3.8-21 Revision 7 (10/94)

CONTAINMENT INTERNAL CONCRETE STRUCTURE LOAD COMBINATIONS

1. Normal Operating

   $C^* = 1.5\,(D+L'+T) + 1.8\,A$

2. Normal Shutdown

   $C = 1.5\,(D+L'+T+F) + 1.8\,L$

3. Abnormal

   $C = (1.0 \pm 0.1)\,(D+T) + 1.5\,P + + 1.25\,(L'+M+A+Q) + T'''$

4. Severe Environmental/Normal Operating

   a) $C = 1.25\,(D+L'+T+A+E)$

   b) $C = 0.9\,(D+L') + 1.1\,E + (1.0 \pm 0.1)\,T$

5. Severe Environmental/Normal Shutdown

   a) $C = 1.25\,(D+L'+T+E+F)$

   b) $C = 0.9\,(D+L'+F) + 1.1\,E + (1.0 \pm 0.1)\,T$

6. Abnormal/Severe Environmental

   $C = (1.0 \pm 0.1)\,(D+T) + 1.25\,(P+M+Q+A+L'+E) + T''$

7. Extreme Environmental/Normal Operating

   $C = (1.0 \pm 0.1)\,(D+T+L'+F) + 1.0\,E'$

8. Extreme Environmental/Normal Shutdown

   $C = (1.0 \pm 0.1)\,(D+T+L'+F) + 1.0\,E'$

9. Abnormal/Extreme Environmental

   $C = (1.0 \pm 0.1)\,(D+T+L'+A) + 1.0\,(P+E'+M+Q) + T'$

*C = Ultimate Design Load

TABLE 3.8-22

CONTAINMENT INTERNAL STEEL STRUCTURE LOAD COMBINATIONS

1A) Steam Generator Upper Support

a) D+T+E'+Q
b) D+T+E'+Q+P

1B) Steam Generator Lower Support (Sliding Base)

a) D+T+E'+Q

2) Reactor Coolant Pump Support

a) D+T+E'+Q

3) Reactor Vessel Support

a) D+T+E'+Q

4) React. Coolant Pump Stops

a) D+R+E'+Q

5) Main Steam and Feedwater Pipe Restraints

Q+T

6) Duct And Tray Restraints

a) D+E
b) D+E'

7) Platform Framing

a) L+D+L'+E
b) L+D+L'+Q+E'

8) Polar Crane

a) L+D+Impact+Lateral or Longitudinal Thrust
b) D+E
c) D+E'

TABLE 3.8-23

COMPARISON OF MAX. DISPLACEMENT

FIXMAT 2037 AND STARDYNE PROGRAMS

MASS POINT	Fixmat 2037		Stardyne (Version 4/1/72)	
	Time Secs	Max. Disp. Meters	Time Secs	Max. Disp. Meters
1	2.48	0.0565	2.48	0.0567
10	2.64	0.0179	2.69	0.0179
11	2.48	0.0419	2.43	0.0420
29	2.69	0.0172	2.69	0.0172
30	2.48	0.0436	2.48	0.0437
42	2.69	0.0156	2.69	0.0155

TABLE 3.8-24

COMPARISON OF MAX. FORCE

FIXMAT 2037 AND STARDYNE PROGRAMS

MASS POINT	Fixmat 2037		Stardyne (Version 4/1/72)	
	Time Secs	Max. Disp. Tons	Time Secs	Max. Disp. Tons
1	2.48	3,180	2.48	3,150
10	2.68	14,200	2.69	14,100
11	2.48	20	2.48	20
29	2.68	2,050	2.69	2,040
30	2.49	124	2.49	123
42	2.68	332	2.69	329

TABLE 3.8-25

COMPARISON OF MAX. SHEAR

FIXMAT 2037 AND STARDYNE PROGRAMS

MASS POINT	Fixmat 2037		Stardyne (Version 4/1/72)	
	Time Secs	Max. Shear Tons	Time Secs	Max. Shear Tons
1	2.48	3,170	2.48	3,150
10	2.68	85,600	2.69	85,600
11	2.48	20	2.48	20
29	2.68	3,670	2.69	3,690
30	2.49	124	2.49	123
42	2.68	2,840	2.69	2,810

TABLE 3.8-26

COMPARISON OF MAX. MOMENT

FIXMAT 2037 AND STARDYNE PROGRAMS

MASS POINT	Fixmat 2037		Stardyne (Version 4/1/72)	
	Time Secs	Max. Moment Ton-Meters	Time Secs	Max. Moment Ton-Meters
1	2.48	30,315	2.48	29,900
10	2.48	2,890,000	2.48	2,890,000
11	2.48	54	2.48	54
29	2.48	62,000	2.48	62,300
30	2.49	525	2.48	520
42	2.67	39,600	2.69	38,600

TABLE 3.8-27

COMPARISON OF NATURAL FREQUENCIES

FIXMAT 2037 AND NASTRAN PROGRAMS

	Ebasco FIXMAT 2037		NASTRAN		
Mode No.	Frequency(cps)	Periods(sec)	Frequency(cps)	Periods(sec)	Difference
1	6.7125	0.148976	6.7126	0.148978	.001%
2	10.3649	0.96479	10.3655	0.096474	.005%
3	17.5966	0.056829	17.5982	0.056824	.009%
4	19.0368	0.052530	19.0372	0.052529	.002%
5	24.4162	0.040956	24.4148	0.040959	.006%
6	26.8882	0.037191	26.8870	0.037193	.004%
7	40.1862	0.024884	40.1882	0.024883	.005%
8	55.3813	0.018056	55.3746	0.018059	.012%
9	56.4565	0.017713	56.4578	0.017712	.002%
10	77.6412	0.012880	77.6431	0.012879	.002%
11	102.0160	0.009402	102.0294	0.009801	.013%
12	140.2630	0.007129	140.2915	0.007128	.020%

Notes: Each mass point has two degrees of freedom, i.e., translation and rotation (plane motion).

TABLE 3.8-28

COMPARISON OF ACCELERATIONS

FIXMAT 2037 AND NASTRAN PROGRAMS

Load Case No.	Mass Point No.	Ebasco FIXMAT 2037 Maximum Acceleration(g)	Ebasco FIXMAT 2037 Time(sec)	NASTRAN Maximum Acceleration(g)	NASTRAN Time(sec)	Difference
I	1	0.647337	6.12	0.647330	6.12	0.0%
	2	0.560239	6.05	0.557619	6.12	0.5%
Trans-	3	0.462239	6.05	0.456960	6.04	1.6%
lation	4	0.416418	6.05	0.411061	6.04	1.3%
at	5	0.284222	6.05	0.283965	6.04	0.1%
Base	6	0.149526	5.11	0.149074	5.10	0.3%
II	1	0.049395	2.58	0.048804	2.59	1.2%
	2	0.042394	2.58	0.041944	2.59	1.0%
Rocking	3	0.034682	2.58	0.034359	2.59	0.9%
at Base	4	0.031209	2.58	0.030930	2.59	0.9%
	5	0.020966	2.58	0.020771	2.59	0.9%
	6	0.010505	2.58	0.010399	2.59	1.0%
III	1	0.643573	6.05	0.632863	6.05	1.7%
Trans-	2	0.558311	6.05	0.547478	6.05	1.9%
lation	3	0.460407	6.05	0.451547	6.04	1.9%
and	4	0.414717	6.05	0.407912	6.04	1.8%
Rocking	5	0.282840	6.05	0.281619	6.04	0.4%
at Base	6	0.154736	5.11	0.153424	5.10	0.8%

Notes:

a) Load Case I: Base subjected to translational acceleration time history only.

b) Load Case II: Base subjected to rocking acceleration tme history only.

c) Load Case III: Base subjected to translational and rocking time histories simultaneously.

TABLE 3.8-29

COMPARISON OF SHEAR FORCES

FIXMAT 2037 AND NASTRAN PROGRAMS

		Ebasco FIXMAT 2037		NASTRAN		
Load Case No.	Mass Point No	Shear Force(kips)	Time(sec)	Shear Force(kips)	Time(sec)	Difference
I	1	3006	6.12	3009	6.12	0.1%
	2	3774	6.12	3777	6.12	0.1%
Trans-	3	4607	6.05	4545	6.05	1.4%
lation	4	6703	6.05	6601	6.05	1.5%
at Base	5	29317	6.05	28928	6.04	1.3%
	6	42402	6.04	42094	6.04	0.7%
II	1	229	2.58	228	2.59	0.0%
	2	228	2.58	286	2.59	0.0%
Rocking	3	351	2.58	348	2.59	0.9%
at Base	4	508	2.58	505	2.59	0.6%
	5	2178	2.58	2165	2.59	0.5%
	6	3149	2.58	3132	2.59	0.5%
III	1	2989	6.05	2963	6.05	0.9%
Trans-	2	3758	6.05	3723	6.05	0.9%
lation	3	4592	6.05	4546	6.05	1.0%
and	4	6680	6.05	6600	6.05	1.2%
Rocking	5	29184	6.05	28673	6.04	1.7%
at Base	6	42088	6.05	41744	6.04	0.8%

Notes:

a) Load Case I: Base subjected to translational acceleration time history only.

b) Load Case II: Base subjected to rocking acceleration time history only.

c) Load Case III: Base subjected to translational and rocking time histories simultaneously.

TABLE 3.8-30

COMPARISON OF BENDING MOMENTS

FIXMAT 2037 AND NASTRAN PROGRAMS

		Ebasco FIXMAT 2037		NASTRAN		
Load Case No.	Mass Point No.	Bending Moments (ft-kips)	Time(sec)	Bending Moments (ft-kips)	Time(sec)	Difference
I	1	83608	6.12	83663	6.12	0.1%
	2	184357	6.12	184490	6.12	0.1%
Trans-	3	238633	6.12	238810	6.12	0.1%
lation	4	518412	6.12	518874	6.12	0.1%
at	5	1500830	6.05	1471245	6.05	2.0%
Base	6	3058273	6.05	3015678	6.04	1.4%
II	1	6894	2.58	6815	2.59	1.1%
	2	14935	2.58	14781	2.59	1.0%
Rocking	3	19518	2.58	19313	2.59	1.1%
at Base	4	45808	2.58	45229	2.59	1.3%
	5	154328	2.58	152410	2.59	1.3%
	6	318637	2.58	308803	2.59	3.0%
III	1	81128	6.12	83046	6.12	2.3%
Trans-	2	183242	6.05	181843	6.05	0.8%
lation	3	237446	6.05	235591	6.05	0.8%
and	4	518961	6.05	514657	6.05	0.4%
Rocking	5	1518430	6.05	1494436	6.05	1.6%
at Base	6	3176557	6.05	3032765	6.04	4.0%

Notes:
a) Load Case I: Base subjected to translational acceleration time history only.

b) Load Case II: Base subjected to rocking acceleration tme history only.

c) Load Case III: Base subjected to translational and rocking time histories simultaneously.

TABLE 3.8-31

REACTOR COOLANT PIPE STOP (STOP A)

MEMBER	MAX. STRESSES (KSI) ACTUAL STRESS	ALLOWABLE	REMARKS
Plate Below Steel Saddle (PT.A on Figure 3.8-39)	89.7 (fb)	90 (Fb)	4-1/2 plate (A-542)
Vertical Column (PT.B on Figure 3.8-39)	82.4 (fb)	90 (Fb)	2 Flanges 4x14 1 web 2x10 (A-542)
Horiz. Member Parallel To Pipe Axis (PT. C on Figure 3.8-38)	34.3 (fa)	79 (Fa)	2 Flanges 2x12 1 Web 2x8 (A-542)
Horiz. Member Perpendicular To Pipe Axis (PT. D on Figure 3.8-38)	3 (fb)	37.8 (Fb)	W10x49 (A-441)

Notes:

f_b = computed bending stress
F_b = allowable bending stress
f_a = computed axial stress
F_a = allowable axial stress

TABLE 3.8-32

COMPARISON OF CALCULATED DESIGN LOADS AND ULTIMATE CAPACITY

OF STRUCTURAL ELEMENTS OF CONCRETE INTERNAL STRUCTURES

Structural Element	Governing Loading Conditions*	Load Description	Calculated Design Value	Ultimate Capacity
Primary Shield Wall				
a) Vertical	3	Axial Load (kips/ft)	128	148
		Moment (ft-kips/ft)	1372	1580
		Shear (KSF)	13.1	17.3
b) Horizontal	3	Axial Load (kips/ft)	666	766
		Moment (ft-kips/ft)	559	643
		Shear (KSF)	12.1	17.3
Secondary Shield Wall				
a) Vertical	3	Axial Load (kips/ft)	56	60
		Moment (ft-kips/ft)	237	260
		Shear (KSF)	39.2	43.3
b) Horizontal	3	Axial Load (kips/ft)	282	290
		Moment (ft-kips/ft)	473	500
		Shear (KSF)	32.9	37
Refueling Canal Wall				
a) Vertical	3	Axial Load (kips/ft)	318	350
		Moment (ft-kips/ft)	704	770
		Shear (KSF)	56	57.4
b) Horizontal	3	Axial Load (kips/ft)	-322	-375
		Moment (ft-kips/ft)	704	1450
		Shear (KSF)	6.8	15.8

Notes

1- *See Subsection 3.8.3.3 for load combination.

2- For axial load + = tension, - = compression.

TABLE 3.8-33

COMPARISON OF MAXIMUM DESIGN AND ALLOWABLE STRESSES FOR

REACTOR COOLANT PUMP SUPPORTS

Member (see Figure 3.8-37)	Stress (Max KSI) fa(KSI) (actual)	Fa(KSI) (allowable)	Remarks
Vertical Col Support R1, R2, R3, R4	49.5	63	Two flange plates 15-1/2" x 1-1/8" with 3/4" web plate (A533)
Snubber S3	57.22	63	(A533)
Top Horiz Stop ST3	16.14	45	W14 x 398 (A 441)
Middle Horiz Stop ST3	26.74	45	W14 x 398 (A 441)
Lower Horiz Stop ST1	30.83	45	W14 x 127 (A 441)

fa = Computed Axial Stress
Fa = Allowable Axial Stress

TABLE 3.8-34

COMPARISON OF MAXIMUM DESIGN AND ALLOWABLE

STRESSES FOR UPPER STEAM GENERATOR SUPPORTS

Structure	Location 13 See Fig. 3.8-35	Max. Design (ksi)	Allowable (ksi)	Remarks
	A	2.1	32.4	
Top	B	0.9	32.4	
Ring	C	0.5	32.4	
Girder	D	4.3	32.4	
	E	5.3	32.4	
	G	2.5	32.4	
Bottom	H	2.7	32.4	
Ring	J	2.3	32.4	
Girder	K	0.5	32.4	
	L	0.4	32.4	
	M	0.5	32.4	
	N	2.9	21.35	Compression Member
Main Column Transferring Snubber Load	P	1.7	32.4	
Other Columns		2.5	32.4	

Notes:

All Material A 36
Maximum design stresses are combined axial and bending stresses.

TABLE 3.8-35

STEAM GENERATOR SLIDING BASE SUPPORT STRESS COMPARISONS

Member	MAXIMUM STRESS (KSI) Design	Allowable	Remarks
Shear Keys	fb = 66.45 fp = 44.58 fv = 35.89	Fb = 72 Fp = 72 Fv = 41.6	12" plate and 18" plate (A-291 Class 3A)
Top plate at elevation + 9.18 ft. MSL	fb = 17.59 fp = 44.58 fv = 9.74	Fb = 63 Fp = 63 Fv = 36.4	12" plate and 6" plates (A-533 Class 2 Gr.B)
Bottom plate at elevation + 7.18 ft. MSL	fb = 2.70 fp = 5.30 fv = 2.0	Fb = 63 Fp = 63 Fv = 36.4	4" plate (A-533 Class 2 Gr. B)
Bottom plate at elevation + 6.10 ft. MSL	fb = 3.00 fp = 20.06 fv = 2.00	Fb = 63 Fp = 63 Fv = 36.4	4" plate (A-533 Class 2 Gr. B)
Vertical plates	fb = 12.17 fp = 1.90 fv = 7.05	Fb = 63 Fp = 63 Fv = 36.4	3", 4" or 6" plates (A-533 Class 2 Gr. B)

NOTES

1. fb = Computed bending stress
   fp = Computed bearing stress
   fv = Computed shear stress

   Fb = Allowable bending stress
   Fp = Allowable bearing stress
   Fv = Allowable shear stress

2. See Figure 3.8-36

TABLE 3.8-36

REACTOR VESSEL SUPPORT

COMPARISON OF MAXIMUM DESIGN

AND ALLOWABLE STRESSES AND LOADS

Inner Ring Girder

Description of Comparison	Actual Stress or Load	Allowable Stress or Load
Maximum Absolute Principal Stress LOCA (Max) + Temp + SSE	49.24 ksi	56.97 ksi
Max Bearing Stress on Base Plate	1.64 ksi	1.9 ksi
Max Uplift Load on Bolt Group (7 Bolts)	2,420 kips	3,780 kips
Outer Embedded Ring Girder		
Max Absolute Bending Stress	38.59 ksi	58.4 ksi
Max Absolute Shear Stress	19.63 ksi	28.6 ksi

NOTES:

1) Steel for ring girders A533 Grade B, Class 2

2) Refer to Figure 3.8-34

TABLE 3.8-37 (Sheet 1 of 2) Revision 307 (07/13)

NSSS SUPPORT STEEL MATERIAL SUMMARY

Reactor Vessel

support structure:	A533 Grade B, Class 2 (25 mils lateral expansion @ 50 F)
studs, nuts and washers:	A540 Grade B24, Quenched and tempered Vacuum degassed, Charpy V-notch (25 mils lateral expansion @ 50 F)
→(EC-41857, R307)	
shims:	A533, Grade B, Class 2 and AISI Type 301 Stainles Steel and AISI Alloy Sheet 4340
←(EC-41857, R307)	

Steam Generator

sliding base	support structure - same as Reactor Vessel. key - A291, Class 3A, Certified Test Report for chemical analysis, tensile, and impact proper-ties. Charpy V-notch 15 mils lateral expansion at + 50 F
studs:	same as Reactor Vessel and A193 Grade B7
nuts:	same as Reactor Vessel and A194 Grade 7
washers:	same as Reactor Vessel and A325 Type 2
→(EC-41857, R307)	
shims:	A588 and A606
←(EC-41857, R307)	
upper support: steel	A36 or A441
bolts	A325 or A193

Reactor Coolant Pump Supports

snubber support:	A533, Grade B, Class 2.
columns and stop supports:	A441 (15 mils lateral expansion @ 50 F)
anchor bolts:	same as reactor vessel
→(EC-41857, R307)	
shims:	A533, Grade B, Class 2 and A606, Type 4
←(EC-41857, R307)	
embedded plates:	A36 or A441.

Reactor Coolant Piping Stops

saddles:	A579 Grade 12. Charpy V-notch 15 mils lateral expansion at + 50 F.
stops:	A542, Class 2 and A553, Grade B, Class 2. Charpy V-notch 15 mils lateral expansion at + 50 F.
studs:	A490 or A540
nuts:	same as reactor vessel
washers:	same as reactor vessel
→(EC-41857, R307)	
shims:	A533, Grade B, Class 2 and A606, Type 4
←(EC-41857, R307)	

TABLE 3.8-37 (Sheet 2 of 2)

NSSS SUPPORT STEEL MATERIAL SUMMARY

Tower Over Steam Generator

steel:	A441.
bolts:	A490.
u-bolts	A276 Type 304

TABLE 3.8-38

SHIELD STRUCTURE LEAKAGE RATES

(Based upon data presented in the Report NAA-SR-10100, Conventional Buildings for Reactor Containment).

Source of Leakage	Leakage Rate(2) (Cubic Ft. in 24 Hours)	Leakage Rate(3) (Percent of Annulus Volume in 24 Hours)
Concrete Surface of(3) Wall and Dome	17.6	0.004
Construction Joints(1)	82.5	0.017
Cracks in Concrete:		
a - Temperature Cracks	NEGLIGIBLE(3)	
b - Shrinkage Crakcs	NEGLIGIBLE	
c - Earthquake Cracks	NEGLIGIBLE	
d - Stress Cracks at Spring Line	1800	
Penetrations (All)	NEGLIGIBLE	
Equipment Hatch	271	0.057
Personnel Door	130	0.027
Total	2381	0.497

(1) Construction joint leakage is based on constructing the wall with built-up forms and allowing cold joints between successive 6 ft. pours with no seals or coatings at joints. Leakage allowance has also been made for construction joints in the roof.

(2) At 1/4 in. WG differential pressure.

(3) Surface leakage is based on one-third of wall thickness as a conservative allowance for cracks.

(4) Stress crack leakage based on a crack width of 10 mils.

TABLE 3.8-39 (Sheet 1 of 2) Revision 8 (5/96)

OTHER SEISMIC CATEGORY I STRUCTURES LOAD COMBINATIONS

1 - NORMAL OPERATING

$C = 1.5\,(D + L' + T) + 1.8\,(L + A + S) + 1.0B + 1.25R$
(Reactor Auxiliary Bldg Only)

$C = 1.5\,(D + L' + T) + 1.8L$ (Shield Bldg Only)

$C = 1.5\,(D + L' + T) + 1.8\,(L + A) + 1.0B$

Note: Load factors of 1.4, instead of 1.5, and 1.7, instead of 1.8, may be used for analysis of existing structures for the flexure analysis of beams and slabs.
No other load factors have been changed.

2 - SEVERE ENVIRONMENTAL (OBE)

i - $C = 1.25\,(D + L' + A + E + T) + 1.0\,B'' + 1.0\,S'$

ii - $C = 0.9\,(D + L') + 1.1\,(E + T) + 1.0\,B'' + 1.0\,S''$

iii - $C = 1.25\,(D + L' + E + T)$ (Shield Building Only)

3 - EXTREME ENVIRONMENTAL (SSE)

$C = (1.0 \pm 0.10)\,(D + L' + T + A) + 1.0\,(E' + B'')$

$C = (1.0 \pm 0.05)\,(D + L' + T) + 1.0\,(E')$ (Shield Bldg Only)

4 - SEVERE ENVIRONMENTAL (WIND)

i - $C = 1.25\,(D + L' + T + Hu + A) + 1.0\,(B'' + S)$

ii - $C = 0.9\,(D + L') + 1.1\,(Hu + T) + 1.0\,(B'' + S)$

iii - $C = (1.0 \pm 0.05)\,(D + T + L') + 1.25\,(Hu)$ (Shield Bldg Only)

5- FLOODING

$C = (1.0 \pm 0.10)\,(D + L' + A + T) + 1.0\,(B' + S)$

NOT APPLICABLE TO SHIELD BLDG.

6 - EXTREME ENVIRONMENTAL (TORNADO)

$C = (1.0 \pm 0.10)\,(D + L' + A + T) + 1.0\,(W + B'' + S) + 1.0M$

$C = (1.0 \pm 0.10)\,(D + L' + T) + 1.0\,(W)$ (Shield Bldg Only)

TABLE 3.8-39 (Sheet 2 of 2) Revision 11-A (02/02)

OTHER SEISMIC CATEGORY I STRUCTURES LOAD COMBINATIONS

7 - EQUIPMENT ACCIDENT

$C = (1.0 \pm 0.10)(D + L' + A + S + T) + 1.25Q + 1.0B$

NOT APPLICABLE TO SHIELD BUILDING

8 - ABNORMAL/SEVERE/EXTREME ENVIRONMENTAL - SHIELD BUILDING

a) $C = (1.0 \pm 0.05)(D + T + L') + 1.5\ LOCA$

b) $C = (1.0 \pm 0.05)(D + T + L') + 1.25\ LOCA + 1.25\ (E)$

c) $C = (1.0 \pm 0.05)(D + T + L') + 1.0\ (LOCA + E')$

d) $C = (1.0 \pm 0.05)(D + T + L') + 1.25\ (LOCA + Hu)$

e) $C = (1.0 \pm 0.05)(D + L' + T) + 1.25\ IN\ (IP)$

9 - ABNORMAL/SEVERE/EXTREME ENVIRONMENTAL - REACTOR AUXILIARY BUILDING

a) $C = D \pm L + Ta + Ra + 1.5\ Pa$

b) $C = D + L + Ta + Ra + 1.25\ Pa + 1.0\ (Yr + Yj + Ym)\ 1.25\ E$

→(DRN 00-1121)

c) $C = D + L + Ta + Ra + 1.0\ Pa + 1.0\ (Yr + Yj + Ym) + 1.0\ E'$

←(DRN 00-1121)

10 - RIVER TRANSPORTATION ACCIDENT

$C = (1.0 \pm 0.05)(D + L' + T) + 1.0\ AP + 1.0\ B$

$C = 1.0\ (D + AP)$ (Shield Bldg Only)

TABLE 3.8-40

COMPARISON OF CALCULATED DESIGN LOADS
AND ULTIMATE CAPACITY OF STRUCTURAL ELEMENTS
OF SHIELD BUILDING

Structural Element	Governing Load Condition*	Load Description	Calculated Design Value	Ultimate Capacity
		Shear (kips/ft)	7	33
Dome		Axial load (kips/ft)	0.7	0.9
a) Meridional	8	Moment (ft-kips/ft)	231	291
b) Circumferential	8	Shear (kips/ft)	7	291
		Axial load (kips/ft)		
		Moment (ft-kips/ft)		
Cylinder Wall				
a) Vertical above elevation +80.0 ft. MSL	8	Shear (kips/ft)	7	41
		Axial load (kips/ft)	6	19
		Moment (ft-kips/ft)	106	150
b) Horizontal above elevation +80.0 ft. MSL	1	Shear (kips/ft)	14	41
		Axial load (kips/ft)	88	140
		Moment (ft-kips/ft)	79	130
c) Vertical below elevation -80.0 ft. MSL	1	Shear (kips/ft)	11.2	41
		Axial load (kips/ft)	-156	-200
		Moment (ft-kips/ft)	80	500
d) Horizontal below elevation 80.0+ft MSL	1	Shear (kips/ft)	11.2	41
		Axial load (kips/ft)	99	115
		Moment (ft-kips/ft)	41	48

NOTES:

1. - * See Section 3.8.4.3 for load combination.

2. - * For axial load += tension, -= Compression.

TABLE 3.8-41

COMPARISON OF CALCULATED DESIGN LOADS
AND ULTIMATE CAPACITY OF STRUCTURAL ELEMENTS
OF SHIELD BUILDING

Structural Element	Governing Load Condition*	Load Description	Calculated Design Value	Ultimate Capacity
Column K9 elevation	1	Axial load (kips)	5032	5500
-35.0 ft MSL	1	Moment (ft-kips)	689	1955
Beam 11 GL elevation				
+69.0 ft MSL				
1) At support	1	Shear (kips)	460	590
	1	Moment (ft-kips)	4427	5563
2) At span centerline	1	Shear (kips)	0	503
	1	Moment (ft-kips)	5089	6832
East wall	3	Shear (kips/ft)	83	101
	3	Moment (ft-kips/ft)	572	583

* See Subsection 3.8.4.3 for load combination.

TABLE 3.8-42 Revision 11 (05/01)

COMPARISON OF CALCULATED DESIGN LOADS AND ULTIMATE CAPACITY OF STRUCTURAL ELEMENTS OF FUEL HANDLING BUILDING

Structural Element	Governing Loading Condition*	Load Description	Calculated Design Value	Ultimate Capacity
→ (DRN 99-1095)				
Slab U 6 elevation +4.5 ft MSL	1	Shear (kips/ft)	60	70
	3	Moment (ft-kips/ft)	619	854
← (DRN 99-1095)				
Beam 2_{FH} elevation +91.08 ft MSL				
1) At support	6	Shear (kips)	380	807
	6	Moment (ft-kips)	4992	5538
2) At span centerline		Shear (kips)	0	335
		Moment (ft-kips))	1110	1950
Wall				
→ (DRN 99-1095)				
1) Vertical 7_{FH} elevation +3.75 ft MSL	1	Shear (kips/ft)	53	88
← (DRN 99-1095)				
7_{FH} elevation +58.4 ft MSL	6	Moment (ft-kips/ft)	326	344
→ (DRN 99-1095)				
2) Horizontal 5_{FH} elevation +35.0 ft MSL	3	Shear (kips/ft)	75	88
7_{FH} elevation +23.5 ft MSL	6	Moment (ft-kips/ft)	684	1336
← (DRN 99-1095)				
Column T 6 elevation +46.0 ft MSL	3	Axial Load (kips)	-243	-311
	3	Moment (ft-kips)	4000	5122

Notes

1- * See section 3.8.4.3 for load combination

2- For axial load + = tension, - = compression.

TABLE 3.8-43 Revision 6 (12/92)

COMPARISON OF CALCULATED DESIGN LOADS AND ULTIMATE CAPACITY OF THE COMMON MAT UNDER THE FOLLOWING STRUCTURAL ELEMENTS OF THE BUILDINGS

Structural Element	Governing Load Condition*	Load Description	Calculated Design Value	Ultimate Capacity
S/B East Wall	3	Shear (kips/ft)	232	274
S/B East Wall	3	Moment (ft-kips/ft)	3321	6979
RAB	1	Shear (kips/ft)	136	176
RAB	1	Moment (ft-kips/ft)	1558	2791
FHB East Wall	3	Shear (kips/ft)	208	274
FHB East Wall	3	Moment (ft-k/ft)	6770	8729

* See Subsection 3.8.5.3 for load combination