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5.0 STRUCTURES

5.1 CONTAINMENT STRUCTURE

5.1.1 DESIGN BASIS

The containment structure completely encloses the reactor coolant system to minimize release of radioactive material to the environment should a failure of the reactor coolant system occur. The structure provides adequate biological shielding for both normal operation and the hypothetical accident condition.

The containment structure is licensed and designed to withstand a pressure of 55 psig and 283°F. The original transient analysis calculated a peak accident pressure of 49.9 psig and a peak accident temperature of 276°F. The higher licensed design pressure and temperature are based on the AEC's guidelines for containment design at the time of the original SER in 1972. Since these AEC guidelines suggested that the design pressure of containment should be at least 10% higher than the calculated peak accident pressure, 55 psig was found to be an acceptable design pressure. The original containment transient analysis yielded the lower analysis pressure and temperature of 49.9 psig and 276°F, respectively; and the higher 55 psig licensed containment design pressure is considered the nominal structural design pressure, thus allowing a margin of 10% over the calculated peak accident analysis pressure. Based on the acceptability of 55 psig as the licensed containment design pressure, the containment preoperational integrity pressure test was performed at a pressure of 63 psig (115% of 55 psig), which was accepted by the AEC (1972 original operating-license stage SER) as sufficient proof of the initial structural integrity of containment.

The containment designs were re-evaluated under extended power uprate conditions at 2644 MWt core power. Peak containment pressure for LOCA and MSLB events was reanalyzed for EPU conditions. The results, presented in Section 14, are within the 55 psig containment design pressure and 283°F design temperature. C26

The principal design basis for the structure is that it should be capable of withstanding, without loss of integrity, the peak pressure resulting from any size pipe break including the maximum hypothetical accident (MHA). The MHA is defined as the release of the water in the system through a double-ended break of a reactor coolant pipe, coincident with a loss of normal power. The subsequent pressure behavior is determined by the engineered safeguards and the combined influence of energy sources and heat sinks as described in Section 14.3.4.

Engineered safeguards systems are provided to limit the consequences of the MHA and are discussed in Sections 6.2, 6.3, and 6.4 of this report. Their energy removal capabilities limit the internal pressure rise so that the containment design limits are not exceeded and the potential for release of the fission products is minimized.

5.1.1.1 Principal Design Criteria

Quality Standards (Category A)

Criterion: Those systems and components of reactor facilities which are essential to the prevention, or the mitigation of the consequences, of nuclear accidents which could cause undue risk to the health and safety of the public shall be identified and then designed, fabricated, and erected to quality standards that reflect the importance of the safety function to be performed. Where generally recognized codes and standards pertaining to design, materials, fabrication, and inspection are used, they shall be identified. Where adherence to such codes or standards does not suffice to assure a quality product in keeping with the safety function, they shall be supplemented or modified as necessary. Quality assurance programs, test procedures, and inspection acceptance criteria to be used shall be identified. An indication of the applicability of codes, standards, quality assurance programs, test procedures, and inspection acceptance criteria used is required. Where such items are not covered by applicable codes and standards, a showing of adequacy is required. (1967 Proposed GDC 1)

Those systems and components which are essential to the prevention, or the mitigation of the consequences, of nuclear accidents which could cause undue risk to the health and safety of the public are identified as Class I systems in Appendix 5A. The applicable codes and standards pertaining to these systems and additional measures taken beyond these codes and standards are discussed in Sections 5.1.2, 5.1.6, 5.2.2, 5.2.3 and 5.3. Quality assurance programs, test procedures and inspection acceptance criteria related to the original design are given in the Section 1.9. Where no applicable codes or standards exist, a discussion of the design is given in the appropriate section. The design criteria for the containment structure is discussed in Appendix 5B.

Performance Standards (Category A)

Criterion: Those systems and components of reactor facilities which are essential to the prevention or to the mitigation of the consequences of nuclear accidents which could cause undue risk to the health and safety of the public shall be designed, fabricated, and erected to performance standards that will enable such systems and components to withstand, without undue risk to the health and safety of the public the forces that might reasonably be imposed by the occurrence of an extraordinary natural phenomenon such as earthquake, tornado, flooding condition, high wind or heavy ice. The design bases so established shall reflect: (a) appropriate consideration of the most severe of these natural phenomena that have been officially recorded for the site and the surrounding area and (b) an appropriate margin for withstanding forces greater than those recorded to reflect uncertainties about the historical data and their suitability as a basis for design. (1967 Proposed GDC 2)

These systems and components are designed, fabricated, and erected to with stand the forces imposed by extraordinary natural phenomena. A discussion of the magnitude of these forces and the design bases derived there from is discussed in Section 5 and Appendices 5A, 5B, and 5G.

Records Requirements (Category A)

Criterion: The reactor licensee shall be responsible for assuring the maintenance throughout the life of the reactor of records of the design, fabrication, and construction of major components of the plant, essential to avoid undue risk to the health and safety of the public. (1967 Proposed GDC 5)

The applicant will maintain through the life of the unit the records of the design, fabrication, and construction of the major components.

Reactor Containment (Category A)

Criterion: Reactor containment shall be provided. The containment structure shall be designed (a) to sustain without undue risk to the health and safety of the public the initial effects of gross equipment failures, such as a large reactor coolant pipe break, without loss of required integrity and (b) together with other engineered safety features as may be necessary, to retain for as long as the situation requires the function capability of the containment to the extent necessary to avoid undue risk to the health and safety of the public. (1967 Proposed GDC 10)

The reactor containment, a continuous, post-tensioned concrete structure, with a welded steel liner to provide leak tightness, completely encloses the entire reactor and reactor coolant system to ensure, with certain engineered safeguards that an acceptable upper limit for leakage of radioactive materials to the environment will not be exceeded, even if Maximum Hypothetical Accident were to occur. The design assures that the integrity of the reactor containment is maintained under normal and accident conditions.

Missile Protection (Category A)

Criterion: Adequate protection for those engineered safety features, the failure of which could cause an undue risk to the health and safety of the public, shall be provided against dynamic effects and missiles that might result from plant equipment. (1967 Proposed GDC 40)

Those engineered safeguards, the failure of which could cause an undue risk to the health and safety of the public, are adequately protected against dynamic effects and missiles that might result from credible unit equipment failures.

Reactor Containment Design Basis (Category A)

Criterion: The reactor containment structure, including access openings and penetrations and any necessary containment heat removal systems shall be designed so that any leakage of radioactive materials from the containment structure under conditions of pressure and temperature resulting from the largest credible energy release following a loss-of-coolant accident, including the calculated energy from metal-water or other chemical reactions that could occur as a consequence of failure of any single active component in the emergency core cooling system, will not result in undue risk to the health and safety of the public. (1967 Proposed GDC 49)

The containment structure including access openings and penetrations, is designed to a maximum allowable leak rate of 0.20 percent by weight of containment air per day at the Containment design pressure of 55 psig under Extended Power Uprate conditions. Under MHA conditions, the site boundary and off-site doses are below the guidelines of 10 CFR 50.67. The highest transient peak pressure, associated with postulated rupture of the piping in the reactor coolant system and the calculated effects of a metal-water reaction, does not exceed these values. C26

<u>NDT Requirement for Containment Material (Category A)</u>

Criterion: The selection and use of containment materials shall be in accordance with applicable engineering codes. (1967 Proposed GDC 50)

The ferritic materials used as load carrying components in the containment structure are selected in accordance with the appropriate codes, regulations, and testing requirements.

5.1.2 GENERAL DESCRIPTION AND DESIGN LOADS

The containment, which is a Class I structure, consists of a post-tensioned reinforced concrete cylinder and a shallow dome, connected to and supported by a massive reinforced concrete foundation slab as shown in Figure 5.1-1.

The inside surface of the structure is lined with a 1/4" thick welded steel plate to insure a high degree of leak tightness. Numerous mechanical and electrical systems penetrate the containment through welded steel penetrations as shown in Figure 5.1-2 and 5.1-3. These penetrations and all other areas of the liner plate not backed by structural concrete are designed, fabricated, inspected, and installed in accordance with Section III, Subsection B, of the ASME Pressure Vessel Code.

Principal dimensions of the containment structure are as follows:

Inside diameter	116 feet
Inside height (including dome)*	170.6 feet
Vertical wall thickness	3 3/4 feet
Dome thickness	3 1/4 feet
Foundation slab thickness	10 1/2 feet
Internal Free volume	
Minimum Estimated	1.45E+06 cu. ft.
Maximum Estimated	1.60E+06 cu. ft

C26

In the concept of a post-tensioned containment, the internal pressure load is balanced by the application of an opposing external pressure type load on the structure. Sufficient post-tensioning is applied on the cylinder and

* The inside height does not include a nominal 1.6 ft. concrete pad on top of the baseplate. Actual inside height including base slab is 169 ft.

dome to more than balance the internal pressure, leaving a margin of external pressure beyond that required to resist the design accident pressure. Nominal bonded reinforcing steel is also provided to distribute strains due to shrinkage and temperature. Additional bonded reinforcing steel is used at penetrations and discontinuities to resist local moments and shears.

The internal pressure loads on the base slab are resisted by both the external soil pressure due to dead load and the strength of the reinforced concrete slab. Thus, post-tensioning is not required to exert an external pressure for this portion of the structure.

The original post-tensioning system nominally consists of:

1. Three groups of 55 dome tendons oriented at 120 degrees to each other for a total of 165 tendons anchored at the vertical face of the dome ring girder.

2. 180 vertical tendons, anchored at the top surface of the ring girder and at the bottom of the base slab.

3. 489 hoop tendons, each enclosing 120 degrees of arc and anchored at the six vertical buttresses.

Each tendon nominally consists of 90 - 1/4" diameter wires with buttonheaded BBRV type anchorages, furnished by the Prescon Corporation. The replacement tendons installed during reactor vessel closure head replacement were furnished by Precision Surveillance Corporation. The replaced components were a 100% compatible replacement for the original Prescon manufactured tendons. The exact number of tendons and number of effective wires per tendon are tracked to ensure that the required prestressing forces are maintained. The tendons are housed in spirally-wrapped, corrugated, thin wall sheathing and capped at each anchorage with a sheathing filler cap. Within the reactor vessel closure head replacement containment opening, tendon sheathing was fabricated from rigid conduits that were joined to the existing sheathing. After fabrication, the tendon is shop dipped in grease, bagged and shipped. After installation, the tendon sheathing and caps are pumped full with corrosion preventing grease.

In addition to this corrosion protection system, that portion of the tendon system in the base slab, and the reinforcing steel, are tied together into an impressed current cathodic protection system.

High strength reinforcing steel, mechanically spliced with T-Series Cadwelds is used throughout the base slab. Intermediate grade steel is used for bonded reinforcing throughout the cylinder and dome as crack control reinforcing. At areas of discontinuities where additional steel is used, such steel is generally A-432 to provide an additional margin of elastic strain capability.

ASTM A615 grade 60 rebar was also used to repair the containment opening following Reactor Vessel Closure Head replacement. Bar Grip Type-XL mechanical splices were used to repair the existing mat of #9 and #11 reinforcing steel. A new mat of #11 rebar was added on the interior face of the containment opening repair patch.

The 1/4-inch thick liner plate is attached to the concrete by means of an angle grid system stitch welded to the liner plate and embedded in the concrete. The details of the anchoring system are shown in Figure 5.1-1. The spacing of anchors is designed to prevent significant distortion of the liner plate during accident conditions and to ensure that the liner maintains its leak tight integrity. The design of the liner anchoring system also considers the various erection tolerances and their effect on its performance. The liner plate, with the exception of the floor liner, is coated on the inside surface with an inorganic zinc primer and finish painted for corrosion protection. For repair of defective coatings or application of new coatings inside the containment, an engineering approved alternate coating system will be used. There is no paint on the outside surface which is in contact with the concrete shell. The floor liner is coated with a bond breaker to allow free thermal expansion of the 18-inch cover concrete.

For repair of the Reactor Vessel Closure Head Replacement containment opening, a section of the existing liner plate was removed and replaced. The angle and channel grid stiffeners were removed and replaced within the vicinity of the liner plate cut/repair seam. Additional channel stiffeners were welded to the angle grid system to provide strength during concrete repair.

The concrete used in the original construction of the structure is made with Maule Pennesuco limestone (Oolite) aggregate. The concrete design strengths for the structure are 5000 psi at 28 days and 4000 psi at 28 days for the shell and the base slab, respectively.

A nominal 5000 psi at 3-day and 6100 psi at 28 day strength mix, made with Maryville Tennessee aggregate, was used for repairing the temporary construction opening following Reactor Vessel Closure Head replacement.

Normal access to the structure is provided by a double door personnel access lock as shown in Figure 5.1-4. A 14'-0" diameter double gasketed equipment hatch is provided as shown in Figure 5.1-5. A double door emergency personnel escape lock is also installed as shown in Figure 5.1-6. The locks and the equipment hatch are designed and fabricated in accordance with Section III, Subsection B, of the ASME Boiler and Pressure Vessel Code using Firebox quality steel made to SA-300 specification.

The structural brackets provided for the containment crane runway and for the dome liner erection trusses are fabricated from structural shapes and reinforced insert plates (Figure 5.1-1). All structural brackets and reinforcing plates are shop fabricated and stress relieved as completed assemblies and then shipped to the jobsite for welding into the 1/4" liner plate in the same manner as the penetration assemblies.

The containment structure is designed and constructed in accordance with the Design Criteria, Appendix 5B. These criteria are based upon the ACI 318-63, ACI 301-66, and ASME Unfired Boiler and Pressure Vessel Code, Sections III, VIII, and IX. It is the intent of the criteria to provide a structure of unquestionable integrity that will meet the postulated design conditions with a low strain elastic response. The design is, in general, based on the proven stress, strain, and minimum proportioning requirements of the ACI and ASME Codes. These codes have been supplemented in the following manner:

1. The environmental and corrosion conditions, load cycling maintenance and inspection requirements for the structure have been evaluated and compared with those anticipated by the Codes, and modifications made to reflect the actual conditions.

2. The consultant firm of T. Y. Lin, Kulka, Yang & Associate was retained to assist in the development of the criteria. They have been involved in development of the criteria and review of the design methods and drawings to assure that the criteria were being implemented as intended.

3. Upon completion of the PSAR review by the AEC Division of Reactor Licensing Staff it was agreed to pursue the most recent of criteria for combined shear, bending, and axial loads on the various structures. Dr. Alan H. Mattock of the University of Washington was retained by Bechtel Corporation to assist in developing the design.

4. All criteria, specifications, and details relating to the liner plate and penetrations, cathodic protection, and corrosion protection have been reviewed by Bechtel's Metallurgy and Quality Control Department. This department maintains a staff to advise the Corporation on welding, quality control, metallurgy, and corrosion protection. The 1994 replacement containment cathodic protection was reviewed by FPL's CSI (Component, Support & Inspections) corrosion specialists.

The primary membrane integrity of the structure is provided by the unbonded post-tensioning tendons, each one of which is initially stressed to 80% of its ultimate strength, and performs at approximately 60% - 65% during the life of the structure. Thus the main strength elements have been individually proof tested prior to operation.

The post-tensioning tendons of the containment structure are divided into dome, vertical and hoop tendon groups.

Any three adjacent tendons in any of these groups can be lost without significantly affecting the strength of the structure due to the load redistribution capabilities of the shell. The bonded reinforcing steel provided for crack control ensures that this redistribution capability exists.

The unbonded tendons are continuous from anchorage to anchorage, being deflected around penetrations, and isolated from secondary strains of the shell. Thus the membrane integrity of the shell can be ensured regardless of conditions of high local strains

The unbonded tendons exist in the structure at a slightly decreasing stress due to relaxation and creep, and even during pressurization in a design accident condition are subject to a stress change of very small magnitude, 2 to 3% of ultimate strength. Thus the main structural system is never subjected to large changes in load, even during hypothetical accident conditions.

The concrete portion of the structure, like the tendons, is subjected to the highest state of stress during the initial post-tensioning. During pressurization it is subjected to a large change in load (or state of stress) but the change is, in general, a decrease in load. The large membrane compressive stresses are diminished, and replaced, by small radial pressures and stresses.

The deformations of the structure during unit operation, or due to hypothetical conditions, are small due to the low strain behavior of the concrete. The largest deformations occur at the time of initial post-tensioning and shortly thereafter, prior to operation. This low strain behavior, and the inherent, strength of the structure, permit the anchoring of all piping penetrating the structure directly to the shell. The design details (see Figure 5.1-2) eliminate the need for expansion bellows and significantly reduce the likelihood of leaks developing at the penetration.

5.1.2.1 Design Pressure and Temperature

As stated in section 5.1.1, the licensing basis pressure and temperature for containment are 55 psig and 283°F.

The external design pressure of the containment shell is 2.5 psig. This value corresponds to the maximum external pressure that could be developed if the containment were sealed during a period of low barometric pressure and high temperature and subsequently the containment atmosphere were cooled with a concurrent rise in barometric pressure. Vacuum breakers are not provided.

5.1.2.2 Other Design Loads

The containment shell is designed for the following loads in accordance with the Appendix 5B:

1. Dead Loads.
2. Prestress Loads.
3. Live Loads, including allowances for equipment piping, ducting and cable trays.
4. Wind Loads, including tornadoes and hurricanes.
5. Earthquake Loads.
6. Thermal loads from pipes attached to the containment wall.
7. Rupture loads of any one pipe.

5.1.3 CONTAINMENT DESIGN ANALYSES

This section discusses analytical techniques, references and design philosophy for the containment building design/analyses. The results of the original analyses and the 1994 re-analysis are provided in Section 5.1.4 and Appendix 5H, respectively. The original design criteria, analyses, and construction drawings have been reviewed by Bechtel's consultants, T. Y. Lin, Kulka, Yang & Associate.

Original Analysis

The original containment structure analyses fall into two parts, axisymmetric and non-axisymetric. The axisymmetric analysis is performed through the use of a finite element computer program for the individual loads and is described in Section 5.1.3.1. The axisymmetric finite element approximation of the containment structure shell does not consider the buttresses, penetrations, brackets and anchors. These items of configuration, and lateral loads due to earthquakes or winds, and any concentrated loads, are considered in the non-axisymmetric analysis described in Section 5.1.3.2.

1994 Re-analysis

During the performance of the 20th year tendon surveillance of the Turkey Point Units 3 and 4 containment structure post-tensioning systems, a number of measured normalized tendon lift-off forces were below the predicted lower limit (PLL). Evaluation of the 20th year surveillance results concluded that the probable cause for the low tendon lift-off forces was due to an increased tendon wire steel relaxation loss caused by average tendon temperatures higher than originally considered. The evaluations also concluded that the containment post-tensioning system will provide sufficient prestress force to maintain Turkey Point licensing basis requirements through the 25th year tendon surveillance. The evaluations recommended that a structural re-analysis of the containment structure be performed to determine the minimum required prestress forces, and to establish that the containment structure will continue to meet the licensing basis requirements through the end of the licensed plant 40-year life (see Appendix 5H for additional detail).

A containment structure re-analysis was completed in 1994 and Safety Evaluation JPN-PTN-SECJ-94-027 (Reference 9) has been performed to document the results of this re-analysis.

The containment re-analysis used a three dimensional (3-D) finite element model of the containment structure. The 3-D model consisted of the cylindrical wall (including buttresses), ring girder, dome, base slab, and the major penetrations (equipment hatch and personnel hatch). The containment re-analysis did not include a new evaluation of the base slab since it was not affected by the post-tensioning system. The base slab was included in the 3-D model to provide a realistic boundary condition for the model.

Appendix 5H provides a summary of the containment re-analysis methodology, analytical techniques, references, and results.

The portions of Sections 5.1.3 and 5.1.4 relative to the original analysis of the containment structure which are affected by the 1994 re-analysis (see Appendix 5H) are annotated in the pertinent sections.

License Renewal Analysis

During the License Renewal process, the Turkey Point Units 3 and 4 containment tendons were analyzed for a 60-year life. The analysis concluded that the containment tendons will continue to meet the licensing basis requirements through the licensed plant 60-year life. (Subsection 16.3.4).

Subsequent License Renewal Analysis C31

During the Subsequent License Renewal (SLR) process, the Turkey Point Units 3 and 4 containment tendons were analyzed for an 80-year life. The analysis concluded that the containment tendons will continue to meet the licensing basis requirements through the licensed plant 80-year life. (Subsection 17.3.6)

5.1.3.1 Axisymmetric Analysis (original analysis)

The finite element technique is a general method of structural analysis in which the continuous structure is replaced by a system of elements (members) connected at a finite number of nodal points (joints). Standard conventional analysis of frames and trusses can be considered to be examples of the finite element method. In the application of the method to an axisymmetric solid (e.g., a concrete containment structure) the continuous structure is replaced by a system of rings of triangular cross-section which are interconnected along circumferential joints. Based on energy principles, work equilibrium equations are formed in which the radial and axial displacements at the circumferential joints are the unknowns. The results of the solution of this set of equations is the deformation of the structure under the given loading

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conditions. For the output, the stresses are computed knowing the strain and stiffness of each element.

The finite element mesh used to describe the structure is shown in Figure 5.1-7. The upper portion and lower portion of the structure are analyzed independently to permit a greater number of elements to be used for those areas of the structure of major interest such as the ring girder area, and the base of the cylinder. The finite element mesh of the structure base slab is extended down into the foundation material to take into consideration the elastic nature of the foundation material and its effect upon the behavior of the base slab.

The use of the finite element computer program permits an accurate estimate of the stress pattern at various location of the structure. The following material properties have been used in the program for the various loading conditions:

Material Property	Load Condition: D, F, T_O, T_A		Load Condition: P
$E_{concrete}$, foundation (psi)	3.0×10^6		3.0×10^6
$E_{concrete}$, shell (psi)	1.5×10^6		3.0×10^6
$\gamma_{concrete}$, (Poisson's ratio)	.17		.17
$\alpha_{concrete}$, (coeff. of expansion per F)	5.0×10^{-6}		--
E_{soil} (psi)	$.2 \times 10^5$	(Backfill)	$.2 \times 10^5$
	1.2×10^6	(Miami Oolite)	1.2×10^6
	4.0×10^6	(Fort Thompson)	4.0×10^6
E_{liner} (psi)	30×10^6		30×10^6
f_y liner (psi)	40,000		40,000

(for definition of Load Conditions, see Appendix 5B)

The major benefit of the program is the capability to predict shear and moments due to internal restraint and the interaction of the foundation slab with the soil. The structure is analyzed assuming an uncracked homogeneous material. This approach is conservative, because the decreased relative stiffness of a cracked section would result in small secondary shears and moments.

In arriving at the above tabulated values of E (Modulus of Elasticity), the effect of creep is included by using the following equation for long term loads such as thermal load, dead load, and prestress:

$$E_{cs} = E_{ci} \frac{\varepsilon_i}{\varepsilon_s + \varepsilon_i}$$

where E_{cs} = Sustained modulus of elasticity of concrete

E_{ci} = Instanteous modulus of elasticity of concrete

ϵ_i = Instanteous strain, in/in per psi

ϵ_s = Creep strain, in/in per psi

Appendix 5D shows the relationship of instantaneous sustained strain which was used to arrive at the appropriate E_c. No modification is made for the instantaneous or sustained loading.

The thermal gradients used for design are shown in Figure 5.1-8. The gradients for both the design accident condition and the factored load condition are based on the temperature associated with the factored pressure. (Factored loads are discussed in Appendix 5B, subsection B.1.6). The design pressure and temperature are 55 psig and 283°F. A maximum calculated liner plate temperature of 283°F was used for both factored and unfactored Load Conditions.

The thermal loads are a result of the temperature differential across the structure. The design temperature stresses for this finite element analysis have been prepared so that when temperatures are given at every nodal point, stresses are calculated at the center of each element.

Thus, the liner plate is handled as an integral part of the structure, but having different material properties, and not as a mechanism which would act as an outside force to produce loading only on the concrete portion of the structure.

Under the design accident condition or factored load condition, cracking of concrete at the outside face is expected. The value of the sustained modulus of elasticity of concrete, E_{cs}, is used in the method described in ACI Code 505-54 to find the stresses in concrete, reinforcing steel and liner plate from the predicted design accident thermal loads and factored accident loads.

The iso-stress plots shown in Figures 5.1-9 and 5.1-10 do not consider the concrete as cracked. The thermal stresses are combined in the iso-stress output for the cases of $D + F + T_A$ and $D + F + 1.5P + T_A$. The first case is critical for concrete stresses and occurs after depressurization of the containment; the second case is critical for the reinforcing stresses and it occurs when pressure and thermal loads are combined and cause cracking at the outside face, as shown in Table 5.1.4-1.

The loading combination D + F + 1.5P + T_A is critical for reinforcing design as mentioned above in so far as the axisymmetric loading combinations are concerned, as isostress plots can be drawn only for such loadings. Table 5.1.4-1 however covers loading combinations including non-axisymmetric loadings (such as seismic loads) and it can be seen that there are conditions other than D + F + 1.5P + T_A which are critical for reinforcing design.

The stresses shown in Table 5.1.4-1 consider cracking. To determine the stresses in the concrete and reinforcement, an evaluation is made of the stress block of the cross section being analyzed. The value of the stresses is taken from the computer output in the case of axisymmetric loading and from analytical solutions in the case of non-axisymmetric loading. Both computations are based on homogeneous materials; therefore, some adjustment is necessary to evaluate the true stress-strain conditions when cracks develop in the tensile zone of the concrete. An equilibrium equation can be written for the tension force in the reinforcement, the compressive force and the axial force acting on the section. In this manner the neutral axis is shifted from the position defined by the computer analyses to a position which is the function of the amount of reinforcement, the modulus ratio and the acting axial forces.

Large axial compressive forces might prevent the existence of any tension stresses, as in the loading condition $D + F + T_A$; therefore no self-relieving action exists; the stresses are taken directly from the computer output.

In the case of $D + F + 1.5P + T_A$, the development of cracks in the concrete decreases the thermal moment, and this effect is considered, but the self-relieving properties of other loadings are not taken into account, even in places where they do exist, such as at discontinuities, e.g., the cylinder-base slab connection. This means that in analyzing the section, a reduced thermal moment is added to the unreduced moment caused by other loadings.

The procedure used to determine the area of conventional reinforcement and stresses in concrete for thermal loads is as follows:

Basic Assumptions: The thermal stresses in the containment are comparable to those developed in a reinforced concrete slab, which is restrained from rotation. The temperature varies linearly across the slab. The concrete will crack in tension and the neutral axis will be shifted toward the compressive extreme fiber. The cracking will reduce the compression at the extreme fiber and increase the tensile stress in reinforcing steel.

The following analysis is based on the equilibrium of normal forces, therefore any normal force acting on the section must be added to the normal forces resulting from the stress diagram. The effects of Poisson's ratio are considered while the reinforcement is considered to be identical in both directions.

Stress - Strain relationship in compressed region of concrete:

$$E_c \epsilon_x = \sigma_x - \nu_c \sigma_y$$

$$E_c \epsilon_y = -\nu_c \sigma_x + \sigma_y$$

$$\sigma_x = E_c \frac{\epsilon_x+\epsilon_y v}{1-v^2_c}$$

$$\sigma_y = E_c \frac{\epsilon_y+\epsilon_x v}{1-v^2_c}$$

Assuming: $\sigma_x = \sigma_y = \sigma_c$ and

$\epsilon_x = \epsilon_y = \epsilon_c$

$$\sigma_c = \frac{1}{E_c\ \epsilon_c}\ 1-v_c = 1.205\ E_c\ \epsilon_c \text{ (If } v_c = .17)$$

The reinforcement is acting in one direction, independently from the reinforcement in the perpendicular direction.

Example: If $E_c = 3 \times 10^6$ and $E_s = 30 \times 10^6$

$$n_r = \frac{30}{1.205x3} = 8.3$$

The liner plate is acting in two directions, similar to the concrete except for the differences caused by the Poissons ratios.

$$\sigma_L = E_s\ \epsilon_s \frac{1}{1-v_L} = 1.35\ E_s\epsilon_s \qquad \text{If } v_L = .25$$

$$v_c = .17$$

$$n_L = \frac{1.35x30}{1.205x3} - 11.2$$

The following is an example of the use of the analytical method derived. (See Figure 5.1-21)

Thermal stress in base slab: $E_c = 3x10^6$ psi
$E_s = 30x10^6$ psi
$v_c = 0.17$
$v_L = 0.26$
$n_R = 8.3$
$n_L = 11.2$

Equilibrium of forces considering crack section:

$4.42\ (293+\Delta\sigma_c)\ 8.3 - (65.0+105.7+24.0)\ 1000 +$
$\Delta\sigma_c\ (12x42+3x11.2) = N = -95{,}000$ lbs.
$\Delta\sigma_c = 156.5$ psi
$\sigma_s = (293+156.5)\ 8.3 = 3{,}731$ psi

The concrete and reinforcement stresses are calculated by conventional methods, from the moment caused by loading other than thermal. The analyses assume homogeneous concrete sections. Those concrete and reinforcing steel stresses are then added to the thermal stresses as obtained by the method described.

Notation:

E_c Modulus of elasticity of concrete.
E_s Modulus of elasticity of steel.
n_L Modular ratio of liner plate-concrete.
n_R Modular ratio of reinforcement-concrete.
$\Delta\sigma_c$ Reduction of concrete compressive stress considering cracking.
ϵ_c Concrete strain

ϵ_s Steel Strain.

ϵ_x Concrete strain in X direction.

ϵ_y Concrete strain in Y direction.

v_c Poisson's ratio of concrete.

v_L Poisson's ratio of liner plate.

σ_c Stress in concrete.

σ_L Stress in liner plate.

σ_R Stress in reinforcement.

σ_X Stress in concrete in direction X.

5.1.3.2 Non-axisymmetric Analysis

The non-axisymmetric aspects of configuration or loading require various methods of analysis. The description of the method used as applied to different parts of the containment is given below:

(a) Buttresses

1994 Re-analysis:

The 1994 re-analysis included the buttresses in the 3-D finite element model. The methodology and the results of the 1994 re-analysis are documented in Appendix 5H. However, the 1994 re-analysis did not evaluate the buttress tendon anchorage zone stress distribution. The information relative to the local stress distribution in the immediate vicinity of the bearing plates as documented in the remainder of this section, including Figures 5.1-11 and 5.1-12 are unaffected by the 1994 re-analysis.

Original Analysis

The buttresses are analyzed for two effects, non-axisymmetry and anchorage zone stresses. Both effects are shown in the results of a two-dimensional plane strain finite element analysis with loads acting in the plane of the coordinate system (Fig. 5.1-11).

At each buttress, the hoop tendons are alternately either continuous or spliced by being mutually anchored on the opposite faces of the buttress. Between the opposite anchorages, the compressive force exerted by the spliced tendon is twice as much as elsewhere. This value combined with the effect of the tendon which is not spliced will be 1.5 times the prestressing force acting outside of the buttresses. The cross-sectional area at the buttress is about 1.5 times that of the wall, so that the hoop stresses as well as the hoop strains and radial displacements can be considered as being nearly constant all around the structure. Iso-stress plots of the plane strain analysis, Figure 5.1-12 confirm this.

The vertical stresses and strains, caused by the vertical post-tensioning becomes constant at a short distance away from the anchorages because of the stiffness of the cylindrical shell. Since the stresses and strains remain nearly axisymmetric despite the presence of the buttresses, their effect on the overall analysis is negligible when the structure is under dead load or prestressing loads.

When an increasing internal pressure acts upon the structure, combined with a thermal gradient such as at the design accident condition, the resultant forces being axisymmetric, the stiffness variation caused by the buttresses decreases as the concrete develops cracks. The structure tends to shape itself to follow the direction of the acting axisymmetric resultant forces even more closely. Thus, the buttress effect is more axisymmetric at yield loads, which include factored pressure, than at design loads including pressure. This fact combined with the design provision that alternate horizontal tendons terminate in a single buttress, indicates that the buttresses will not reduce the margins of safety available in the structure.

The analysis of the anchorage zone stresses at the buttresses has been performed. The local stress distribution in the immediate vicinity of the bearing plates has been derived by the following three analytical procedures:

1. The Guyon equivalent prism method:
   This method is based on both experimental photo-elastic results as well as on equilibrium considerations of homogeneous and continuous media. It should be noted that the relative bearing plate dimensions, ie., ratio of the width of the bearing plate to the width of the concrete under the plate, are considered.

2. In order to include biaxial stress effects, use is made of the experimental test results presented by S. J. Taylor (Reference 8). This paper compares test results with most of the currently used approaches (such as Guyon's equivalent prism method). It also investigates the effect of the rigid trumpet welded to the bearing plate.

3. The finite element method assuming homogeneous and elastic material is used in a plate strain analysis. The mesh and results are shown in Figures 5.1-11 and 5.1-12.

The Guyon method yields the following results:

Maximum Compressive Stress under the bearing plate:

$\sigma c = -2400$ psi

Maximum Tensile Stress in spalling zone:

σ spalling $= +2400$ psi $= -\sigma_c$

Maximum tensile stress in bursting zones:

σ maximum bursting $= 0.04$ P $= + 96$ psi

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

σ maximum bursting $= +192$ psi

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

σ maximum bursting $= +220$ psi

A state of biaxial tension in the concrete appears on the outside face under the loading case $1.05D + 1.5P + 1.0T_A + 1.0F$. The superposition of the corresponding state of stress with the local anchor stresses reduces the load carrying capacity of the anchorage unit and causes a reduction in the maximum tensile strain to cracking.

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

The design of the buttress anchor zones consider such additional vertical stress, leading to a state of pseudo-biaxial stress, the second direction being radial through the thickness.

For the above mentioned case, $1.05D + 1.5P + 1.0T_A + 1.0F$ of the averaged vertical (meridional) stress component is:

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

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

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

(Note: the steel trumpet is assumed to carry 7.2% of the prestress force.)

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

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

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

shows that failure could occur if vertical reinforcing were not provided. In fact, the maximum allowable vertical averaged tensile stress according to Taylor's interaction curve is

$$\frac{f_a}{f'_c} = 0.03$$

therefore, f_a +150 psi. For this reason, special anchorage zone reinforcing is used in addition to that required by the loading cases. Such special reinforcing is based on the following considerations:

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

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

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

For typical detailed Analysis, see Topical Report B-Top-2 dated October 1969, submitted in connection with Docket No. 50-255, a NON-PROPRIETARY report.

(b) Earthquake or Wind Loading

The stresses in the structure for the earthquake loading conditions exceed the stresses for design tornado or wind.
The earthquake analysis is conducted in the following manner:

The loads on the containment structure caused by earthquake
are determined by a dynamic analysis of the structure. The
dynamic analysis is made on an idealized structure of lumped masses and weightless elastic columns acting as springs.

The analysis is performed in two stages; the determination of natural frequencies of the structure and its mode shapes, and the response of these modes to the earthquake by the spectrum response. For the supported equipment, piping, etc. a time history technique is used to develop the floor response spectrum curves, and the supported elements are then analyzed by the response spectrum method as discussed in Appendix 5A, Section 5A-2.0. |

The natural frequencies and mode shapes are computed using the matrix equation of motion shown below for a lumped mass system. Matrix interaction was performed by use of a digital computer program to yield the natural frequencies and mode shapes. The form of the equation is:

$$(K) \cdot (\Delta) = \omega^2 \cdot (M) \cdot (\Delta)$$

K = Matrix of stiffness coefficients including the combined effects of shear, flexure, rotation and horizontal translation.
M = Matrix of lumped masses
Δ = Matrix of mode shapes
ω = Angular natural frequency of vibration

The results of this computation are the several values of ω_n and mode shapes Δn for n = 1, 2, 3, ---m where m is the number of degrees of freedom (i.e., lumped masses) assumed in the idealized structure.

To obtain the loads on the containment structure the response of each mode of vibration to the design earthquake is computed by the response spectrum technique as follows:

(1) The base shear contribution of the n^{th} mode

$V_n = W_n S_{an} (\omega_n \gamma)$ where:

W_n = effective weight of the structure in the n^{th} mode.

$$W_n = \frac{(\Sigma_x \Delta_{xn} W_x)^2}{\Sigma_x (\Delta_{xn})^2 W_x}$$ where the subscript x refers to

levels through the height of the structure and w_x is the weight of the lumped mass at level x.

ω_n = angular frequency of the n^{th} mode.

$S_{an} (\omega_n \gamma)$ = spectral acceleration of a single degree of freedom system with a damping coefficient of γ, obtained from the response spectra.

(2) The horizontal shear load distribution for the n^{th} mode is then computed as:

$$F_{xn} = V_n \frac{(\Delta_{xn} W_x)}{\Sigma_x \Delta_{xn} W_x}$$

where F_{xn} effective force for the n^{th} mode at point x.

The several mode contributions are then combined to give the final response of the structure to the design earth quake.

V_n = base shear for the n^{th} mode.

Δ_{xn} = model displacement for the n^{th} mode at point x.

W_x = lumped weight for the n^{th} mode at point x.

(3) The number of modes considered in the analysis is established so that it adequately represents the structure under analysis. Since the spectral response technique yields the maximum value of response for each mode, and these maximums do not occur at the same time, the response of the modes of vibration is combined by taking the square root of the sum of the squares of the modal values.

For design calculations, the moments and shears have been based on lateral loads from the ground response spectrum curves (Figures 5A-1, 5A-2) , for a modal frequency of 4.8 cps. (This frequency is based on the data presented in the PSAR Supplement No. 11). A mass model has subsequently been developed, as shown in Figure 5.1-13, and analyzed by computer as discussed. The moments and shears based on the response spectrum curves are conservative compared to the computer results and have therefore not been adjusted.

Floor response spectrum curves are obtained by first conducting a time history analysis of the idealized structure. This is achieved by solving the following modal equation of motion for each mode under consideration:

$$\ddot{q}_n + 2\,\gamma_n W_n \dot{q}_n + W_n^2 q_n = \frac{-\,\ddot{z}_{(t)}\,\{\Delta_n\}^T [M]}{M_n}$$

Where q_n , $\dot{q}_n$, & $\ddot{q}_n$ is respectively the modal displacement, velocity, and acceleration of the n^{th} mode.

γ_n is the modal damping (percent critical damping) of the n^{th} mode.

W_n is the n^{th} mode natural frequency

$\{\Delta_n\}$ is the mode shape of the n^{th} mode

$[M]$ is the matrix of the structure masses

M_n is the generalized mass of the n^{th} mode

$\ddot{z}_{(t)}$ is the earthquake motion as a function of time.

The acceleration for each mode is then transferred to the geometrical coordinate system. All the modal accelerations are then summed together and added to the input accelerations at each increment of time considered to provide the absolute acceleration at the various floor levels.

Knowing the floor level acceleration time history, the response spectrum curve is obtained by solving the following equation for maximum acceleration ($\ddot{y}$) as a function of the natural frequency (W_e).

$$\ddot{y} + 2\beta W_e \dot{y} + W_e^2 y = -\ddot{z}_x$$

Where y, $\dot{y}$, $\ddot{y}$ is respectively the displacement, velocity, and acceleration of the equipment.

ß = ratio of equipment critical damping

W_e = natural frequency of the equipment

$\ddot{z}_x$ = floor acceleration obtained from a time history solution of the structure as discussed above.

The solution of these equations is accomplished by using numerical integration techniques on high speed digital computers. Curves relating maximum acceleration ($\ddot{Y}_{max}$) and natural frequency are plotted by computer for the various floor levels and for the required damping rates.

(c) Large Opening (Equipment Hatch & Personnel Lock Opening)

1994 Re-analysis:

The major penetrations (equipment hatch and personnel hatch) have been analyzed as part of the containment structure re-analysis effort. These penetrations were included in the 3-D finite element model to capture the behavior of the shell in the vicinity of these large penetrations. This portion of the containment re-analysis effort (completed in 1994) is considered the updated analysis for these major penetrations. The methodology, analytical techniques, and the summary of the results are included in Appendix 5H. The information included in Section 5.1.3.2 (c) is considered historical.

Original Analysis

The primary loads considered in the design of the equipment hatch and personnel lock openings as for any part of the structure are the dead load, prestress, pressure, earthquake, and thermal loads. The following secondary effects caused by the above primary loads are considered:

1. The deflection of tendons around the opening.

2. The curvature of the shell at the opening.

3. The thickening around the opening.

The primary loads listed are mainly membrane loads, with the exception of the thermal loads. In addition to membrane loads, accident pressure also produces punching shear around the edge of the opening.

The magnitude of these loads at the elevation of the center of the opening is used for design purposes. These are fairly simple to establish, knowing the values of hoop and vertical prestressing, accident pressure, and the geometry and location of the opening.

Secondary loads are predicted by the following methods:

1. The membrane stress concentration factors and effects of the deflection of the tendons around the equipment hatch were analyzed as a flat plate by the finite element method. The stresses, predicted by conventional stress concentration factors are very similar to those values found from the above mentioned finite element computer program. Various conditions checked by this method were as follows:

1. During prestressing with only the hoop tendons stressed.

2. The local effects of hoop tendon curvature under the D + F + 1.5P design load condition.

3. After total prestressing D + F.

The membrane loads were applied at the flat plate boundary and the tendon loads from curvature in the plane of the model were applied at the tendon locations.

The analysis considered the effects of thickening by assigning increased E values for the elements representing the thickened portion of the shell, but it did not consider the shell curvature effects, and the fact that the thickening is not symmetrical about the opening.

Reference (1) was used to determine the effects of shell curvature on the stress concentration around the opening.

However, it gives an assurance of the correctness of the assumed stress pattern caused by the prestressing around the opening. Results of this analysis are shown in Figure 5.1-14.

2. With the help of Reference (1), stress resultants around the large opening are found for various loading cases. Comparison of the results found from this reference with the results of a flat plate of uniform thickness with a circular hole, shows the effect of the cylindrical curvature on stress concentrations around the openings. Normal shear forces (relative to opening) are modified to account for the effect of twisting moments as shown in Reference (1). These modified shear forces are called Kirchoff's shear forces. Horizontal wall ties are provided to resist a portion of these shear forces.

3. The effect of the thickening on the outside face around the large opening is considered using several methods. Reference (2) is used to evaluate the effect of thickening on the stress concentration factors for membrane stress. A separate axisymmetric finite element computer analysis for a flat plate with anticipated thickening on the outside face is prepared to handle both axisymmetric and non-axisymmetric loads and to predict the effect of the concentration of hoop tendons with respect to the containment, at the top and bottom of the opening.

For the analysis of the thermal stresses around the opening the same method is used as for the other loadings. At the edge of the opening a uniformly distributed moment equal but opposite to the thermal moment existing on the rest of the shell is applied and evaluated using the methods of the preceding Reference (1). The effects are then superimposed on the stresses calculated for the other loads and effects.

In the case of accident temperature, after the accident pressure has already been decreased, very small or no tension develops on the outside, so thermal strains will exist without the relieving effect of the cracks. However, the liner plate reaches a high strain level, and so does the concrete at the inside corner of the penetration, thereby relieving the very high stresses, but still carrying a high moment in the state of redistributed stresses.

In the case of $1.5P + 1.0T_A$, the cracked concrete with highly strained tension reinforcement constitutes a shell with stiffness decreased but still essentially constant in all directions. In order to control the increased hoop moment around the opening, the hoop reinforcement is about twice that of the radial reinforcement. See Figure 5.1-15.

The wall at the equipment hatch opening is thickened for the following reasons:

1. To reduce the predicted membrane stresses around the opening.

2. To accommodate tendon placement.

3. To accommodate bonded steel reinforcing placement.

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

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

The working stress method (elastic analysis) is applied to the load combinations for design loads, as well as for yield loads, for the analytical procedures described above. The only difference is that higher allowable stresses are used for the yield conditions. The various factored load combinations and capacity reduction factors are specified in Appendix 5B and are used for the yield load combinations using the working stress design method. The design assumption of straight line variation of stresses is maintained under yield conditions.

The governing design condition for the sides of the equipment hatch opening at its outside edge is the accident condition. Under this condition, approximately 60 percent of the total bonded reinforcing steel needed at the edge of the opening at the outside face, is required for the thermal load.

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

1. Normal stresses resulting from membrane forces, including the effect of thickening, contribute approximately minus 35 percent (minus 14 percent of total), i.e. they result in compressive stresses in the reinforcement.

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

3. Normal and flexural stresses resulting from membrane
T forces and moments caused by the effect of cylindrical
curvature contribute approximately minus 15 percent (minus 6
percent of total).

(d) <u>Penetrations</u>

The containment penetrations analysis is divided into three parts:

1. The concrete shell

2. The liner plate reinforcement and closure to the pipe of

electrical canister, and

3. The thermal gradients and protection requirements at the high temperature penetrations.

The three categories will be discussed separately.

(1) Concrete Shell

In general, special design consideration is given to all openings in the containment structure. Analysis of the various openings has however, indicated that the degree of attention required depends upon the penetration size. Small penetrations are considered to be those with a diameter smaller than 2 ½ times the shell thickness; i.e., approximately 9 feet in diameter or less. Reference (1) indicates that for openings of 9 feet in diameter or less the curvature effect of the shell is negligible. In general, the typical concrete wall thickness has been found to be capable of taking the imposed stresses using bonded reinforcement, and the thickness is increased only as required to provide space requirements for radially deflected tendons. The induced stresses due to normal thermal gradients and postulated rupture conditions distribute rapidly and are of minor nature, compared to the numerous loading conditions for which the shell must be designed. The small penetrations are analyzed as holes in a plane sheet. Applied piping restraint loads due to thermal expansion or accident forces are assumed to distribute in the cylinder as stated in Reference (3). Typical details associated with these openings are indicated in Figure 5.1-2.

(2) Liner Plate Closure

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

In the case of a pipe penetration on which large external operating loads are imposed, the stress level from the external loads is limited to the design stress intensity values, δm, given in the ASME Boiler and Pressure Vessel Code, Section III, Article 4. The stress level in the anchor bolts from external loads is in accordance with the A.I.S.C. Code.

The combining of stresses from all effects is performed using the methods outlined in the ASME Boiler and Pressure Vessel Code, Section III, Article 4, Figure N-414. The maximum stress intensity is the value from Figure N-415 (a) of the previously referenced code. Figure 5.1-16 shows a typical penetration and the applied loads.

The stresses from the effects of pipe loads, pressure loads, dead load and earthquake loads are calculated and the stress intensity kept below δ_m.

The stresses from the remaining effects were combined with the above-calculated stresses and the stress intensity kept below δ_a.

The design of the penetrations precludes failure of the leakage barrier at the penetration due to forces associated with pipe whipping. See Figure 5.1-2.

(3) Thermal Gradient

Forced air cooling has been provided at the penetrations for the main steam lines, feedwater lines, blow-down lines, and the non-regenerative letdown line, to cool the space between the liner plate closures and the penetrations in the concrete wall. Steady state computer analysis, using the generalized heat transfer program for the idealized conditions with no cooling, indicate concrete temperatures will be below 150F, which is the arbitrary design objective. (See Figure 5.1-8). Thermocouples have been embedded in the concrete at the subject penetrations to monitor the actual temperatures development.

(e) Liner Plate

(1) There are no design conditions under which the liner plate

is relied upon to assist the concrete in maintaining the integrity of the structure, even though the liner will, at times, provide assistance in order to maintain deformation compatibility.

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

Possible cracking of concrete is considered and reinforcing steel is provided to control the width and spacing of the cracks. In addition, the design is such that total structural deformation remains small during loading conditions and the order of magnitude of any cracking will be less than that sustained in the repeated attempts to fail the prestressed concrete reactor vessel "Model 1", and even smaller than the concrete strains of over-pressure tests of "Model 2" (Both at General Atomic) (Reference 4) and (Reference 5).

As described, the structural integrity consequences of concrete cracking are limited by the bonded reinforcing and unbonded tendons provided in accordance with the design criteria Appendix 5B. The effect of concrete cracking on the liner plate is also considered. The anchor spacing and other design criteria are such that the liner will sustain orders of magnitude of strain, less than did the liner of Model 1 at General Atomic (Reference 4) without tensile failure.

(f) Liner Plate Anchors

The liner plate anchors are designed to preclude failure when subject to the worst possible loading combinations. The anchors are also designed such, that in the event of a missing or failed anchor, the total integrity of the anchorage system would not be jeopardized by the failure of adjacent anchors.

The following load conditions are considered in the design of the anchorage system:

1. Prestress
2. Internal Pressure
3. Shrinkage and creep of concrete
4. Thermal gradients (normal and MHA)
5. Dead load
6. Earthquake
7. Hurricane and tornado wind
8. Vacuum

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

1. Initial inward curvature of the liner plate between anchors due to fabrication and erection inaccuracies.

2. Variation of anchor spacing

3. Misalignment of liner plate seams

4. Variation of liner plate thickness

5. Variation of liner plate material yield strength

6. Variation of Poisson's ratio for liner plate material

7. Cracking of concrete in anchor zone

8. Variation of the anchor stiffness

The anchorage system satisfies the following conditions:

1. The anchor has sufficient strength and ductility so that its energy absorbing capability is sufficient to restrain the maximum force and displacement resulting from the condition where a panel with initial outward curvature is adjacent to a panel with initial inward curvature.

2. The anchor has sufficient flexural strength to resist the bending moment which would result from Condition 1.

3. The anchor has sufficient strength to resist radial pullout force.

When the liner plate moves inward radially as shown in Figure 5.1-17, the sections develop membrane stress due to the fact that the anchors have moved closer together. Due to initial inward curvature, the section between 1 and 4 will deflect inward giving a longer length than adjacent sections and some relaxation of membrane stress will occur. It may be noted here that Section 1-4 cannot reach an unstable condition due to the manner in which it is loaded.

The first part of the solution for the liner plate and anchorage system is to calculate the amount of relaxation that occurs in Section 1-4, since this value is also the force across Anchor 1 if it were infinitely stiff.

This solution is obtained by solving the general differential equation for beams and the use of calculus to simulate relaxation or the lengthening of Section 1-4, Figure 5.1-17. Sheet 1 shows the symbols for the forces that result from the first step in the solution.

Using the model shown in Figure 5.1-17, Sheet 2, and evaluating the necessary spring constants, the anchor is allowed to displace.

The solution yields a force and displacement at Anchor 1, but the force in Section 1-2 is (N)-K_R (Plate) δ_1 and Anchor 2 is no longer in force equilibrium.

The model shown in Figure 5.1-17, Sheet 2, is used to allow Anchor 2 to displace and then to evaluate the effects on Anchor 1.

The displacement of Anchor 1 was $\delta_1 + \delta'_1$ and the force on Anchor 1 is K_c $(\delta_1 + \delta'_1)$. Then Anchor 3 is not in force equilibrium and the solution continues to the next anchor.

After the solution is found for displacing Anchor 2 and Anchor 3, the pattern is established with respect to the effect on Anchor 1 and by inspection, the solution considering an infinite amount of anchors is obtained in the form of a series solution.

The preceding solution yields all necessary results. The most important of these are the displacement and the force on Anchor 1.

Various patterns of welds attaching the angle anchors to the liner plate have been tested for ductility and strength when subjected to a transverse shear leads such as N, and are shown in Figure 5.1-18.

Using the results from these tests together with data from tests made for the Fort St. Vrain PSAR, Amendment No. 2 and Oldbury vessels, Reference 6, a range of possible spring constants are evaluated for the Turkey Point liner. By using the solution previously obtained together with a chosen spring constant, the amount of energy required to be absorbed by the anchor is evaluated.

Division of the amount of energy that the system can absorb by the most probable maximum energy gives the factor of safety.

The following lists the variations in plate yield strength, thickness, and anchor spacing, etc. are studied:

Case I - A plate with a yield stress of 32 ksi and no variation in any other parameters.

Case II - A 1.25 times increase in yield stress and no variation in any other parameters.

Class III - A 1.25 times increase in yield stress, a 1.16 increase in plate thickness and a 1.08 increase for all other parameters.

Class IV - A 2.0 times increase in yield stress with no variation of any other parameters.

Class V - Same as Class III except that the anchor spacing has been doubled to simulate the case of a missing or failed anchor.

The factors of safety obtained for these cases are shown in Table 5.1.3-1.

The detailed analysis and design of Liner Plate Anchors is described in Bechtel Topical Report B-TOP-1 of October, 1969, submitted for the Palisades Project (Docket 50-255). The analysis was expanded in Supplement 1 to B-TOP-1 in November, 1969 and Supplement 2 to B-TOP-1 in December, 1969. These reports are NON-PROPRIETARY.

Liner plate anchors affected by the Reactor Vessel Closure Head Project are analyzed in calculation 7012-CALC-C-002. The results are equivalent to the Bechtel Topical Report B-TOP-1 results dated December, 1972. C22

(g) <u>Supports</u>

In designing for structural bracket loads applied perpendicular to the plane of the liner plate, or loads transferred through the thickness of the liner plate, the following criteria and methods

are used:

(1) The liner plate is thickened to reduce the predicted stress level in the plane of the liner plate. The thickened plate with the corresponding thicker weld attaching the bracket to the plate also reduces the probability of the occurrence of a leak at this location.

(2) Under the application of a real tensile load applied perpendicular to the plane of the liner plate, no yielding is to occur in the perpendicular direction. By limiting the predicted strain to 90% of the minimum ASTM yield value, this criterion is satisfied.

(3) The allowable stress in the perpendicular direction is calculated using the above allowable predicted strain in the perpendicular direction together with the predicted stresses in the plane of the liner plate.

(4) In setting the above criteria, the reduced strength and strain ability of the material perpendicular to the direction of rolling (in plane of plate) are also considered for the brackets that do not penetrate the liner reinforcing plate. In this case, the major stress is normal to the plane of the liner plate

(5) The material quality is assured by ultrasonic examination of the reinforcement plates for lamination defects.

(h) Missiles

The containment structure is designed to resist the external missiles listed in Appendix 5E. The analysis for missile penetration is based on Reference 7.

5.1.3.3 Analysis of the Containment Structure for Reactor Vessel Closure Head Replacement:

Replacement of the Reactor Vessel Closure Head requires the creation of a construction opening in the shell wall of the Containment Structure. The structural analysis required to accomplish this task consists of a finite element model which explicitly represents the vertical tendons, hoop tendons and opening geometry. The ANSYS computer program was used for this analysis. The structure was analyzed for the load combinations given in the UFSAR. Additional load combinations were added, per ACI 318-63, describing the structural loadings while the containment opening was in place. Each load combination was applied to the model in multiple load steps. Each step represents a significant point of change as the structure was undergoing creation and repair.

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9. Safety Evaluation JPN-PTN-SECJ-94-027, “Units 3 & 4 Containment Structure Re-analysis,” Revision 0.

TABLE NO. 5.1.3-1

FACTORS OF SAFETY FOR ANCHOR SYSTEM
(by Energy Calculations)

Case	Nominal Plate Thickness (in)	Initial Inward Displacement (in)	Anchor Spacing L_1 (in)	Anchor Spacing L_2 (in)	Factor of Safety Against Failure
i	.25	.125	15	15	37.0
ii	.25	.125	15	15	19.4
iii	.25	.125	15	15	9.9
iv	.25	.125	15	15	6.28
v	.25	.25	30	15	4.25

5.1.4 IMPLEMENTATION OF CRITERIA

This section documents the manner in which the design criteria have been met by the designer. Various types of documentation are presented.

1994 Re-analysis:

As stated in Section 5.1.3, a containment structure re-analysis was completed in 1994. The re-analysis determined the minimum required prestress forces in hoop, dome, and vertical tendon groups. This re-analysis was performed using the existing design basis requirements as stated in Appendix B.

Appendix 5H provides a summary of the containment re-analysis methodology, analytical techniques, references, and results.

Original Analysis

Section 5.1.4.1 describes the isostress plots and tabulations of predicted stresses for the various materials. The isostress plots of the homogeneous uncracked concrete structure indicate the general stress pattern for the structure as a whole, under various loading conditions. More specific documentation is made of the predicted stresses for all materials in the structure. In these tabulations, the predicted stress is compared with the allowable to permit an easy comparison and evaluation of the adequacy of the design. Sections 5.1.4.2 and 5.1.4.3 illustrate the actual details used in the design to implement the criteria.

5.1.4.1 Results of Analysis

1994 Re-analysis:

The 1994 re-analysis encompasses the cylindrical shell, buttresses, ring girder and dome, and the major penetrations. Figure 5.1-9, Sheets 3 and 4, Figure 5.1-14, and summary of stress results in Table 5.1.4-1, Sheet 4 and 5 (with the exception of Load Case III as explained below) are considered historical information. Refer to Appendix 5H for the updated stress results/information relative to the cylindrical shell, buttresses, ring girder and dome areas, and the major penetrations.

The 1994 re-analysis did not include a new evaluation of the base slab since it is not affected by the post-tensioning system. The base slab was included in the 3-D model to provide a realistic boundary condition for the model. Therefore, the original base slab design/analysis, as summarized in Sections 5.1.3, 5.1.4 and Table 5.1.4-1, Sheet 6, remains unchanged.

The initial prestressing condition (loads imposed on the containment structure due to initial jacking operation of tendons) is noted as "$D + F_{initial}$" in Table 5.1.4-1. This loading condition has occurred and the original analysis results are not affected by the 1994 re-analysis. Therefore, this loading condition was not included in the re-analysis. Table 5.1.4-1, Sheet 3, and isostress plots shown in Figure 5.1-9, Sheet 2 are unaffected.

The Initial Structural Integrity Test (ISIT) load condition "D+F+1.15P" was analyzed in the original containment structure analysis using the estimated level of prestressing at the time of ISIT. This load condition, with this level of prestressing, has occurred and was not included in the 1994 re-analysis. Therefore, the results of the original analysis for this loading condition, as summarized in Table 5.1.4-1 (Load Case III) and Figure 5.1-9, Sheet 1 are unaffected by the 1994 re-analysis. The 1994 re-analysis included this loading condition with the minimum required level of prestressing. Refer to Appendix 5H for the 1994 re-analysis results.

Original Analysis

The isostress plots, Figures 5.1-9 and 5.1-10 show the three principal stresses and the direction of the principal stresses normal to the hoop direction. The principal stresses are the most significant information about the behavior of the structure under the various conditions and were a valuable aid for the final design.

The plots are prepared by a cathode-ray tube plotter. The data for plotting is taken from the stress output of the finite element computer program for the following design load cases:

$D + F$

$D + F + 1.15P$

$D + F + 1.5P + T_A$

$D + F + T_A$

The above axisymmetric loading conditions yield the highest stresses at various locations of the structure.

The ratio between predicted and allowable stress is calculated for both hoop and meridional stresses. The decimal equivalent of the larger of the two ratios is tabulated in Table 5.1.4-1, for various materials. This table is prepared for a presentation of the combined stresses of the axisymmetric and non-axisymmetric loading cases. These stresses are computed on the basis of cracked concrete sections where applicable, in the manner described in section 5.1.3.1. No stresses are shown for the tendons as these have an almost constant stress level regardless of the loading condition.

The controlling load combination for radial shear stress is $1.05D + F + 1.25E + T_A$ at Section N-O, Table 5.1.4-1, Sheet 1. The shear for each component of loading force is as follows:

$$1.05D + F = +7144 \text{ lbs/inch}$$
$$1.25P = -17900 \text{ lbs/inch}$$
$$1.25E = \pm 7550 \text{ lbs/inch}$$
$$T = +2562 \text{ lbs/inch}$$

The sum of forces for design case based on flexure plus shear, using positive sign with earthquake shear, since under this condition the extreme fiber has tensile forces, is - 644 lbs/inch. However, the absolute maximum value of radial shear is - 15744 lbs/inch.

The radial shear force components for load combination $D + F + P + T_A + E$ are as follows:

$D + F = +7144$ lbs/inch
$P = -14320$ lbs/inch
$E = \pm 6040$ lbs/inch
$T_A = 2562$ lbs/inch

Sum of the forces is +1426 lbs/inch and -10654 lbs/inch.

5.1.4.2 Liner Plate Design Provisions

The liner plate is anchored as shown in Figure 5.1-1 with anchorage in both the longitudinal and hoop direction. The anchor spacing and welds are designed to preclude failure of an individual anchor. The load deformation tests referred to in Section 5.1.3.2 indicate that the alternate stitch fillet weld used to secure the anchor to the liner plate would first fail in the weld and not jeopardize the liner plate leak tight integrity.

Erection and fabrication inaccuracies are controlled by specified tolerances given in Section 5.1.6.1.

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

5.1.4.3 Penetration Details

Typical penetration details are shown in Figure 5.1-2 and 5.1-3.

Horizontal and vertical bonded reinforcement is provided to help resist membrane and flexural loads at the penetrations. This reinforcement is located on both the inside and outside face of the concrete. Stirrups are also used to assist in resisting shear loads.

Local crushing of the concrete due to deflection of the reinforcing or tendons is precluded by the following details:

(1) The surface reinforcements either have a very large radius such as hoop bars concentric with the penetration or are practically straight, having only standard hooks as anchorages where necessary.

(2) The tendons are bent around penetrations at a minimum radius of approximately 20 feet. Maximum tendon force at initial prestress is 850 kips, which results in a bearing stress of about 880 psi on the concrete.

It is also important to note that the deflected tendons are continuous past the openings and are isolated from the local effects of stress concentrations by virtue of being unbonded.

In accordance with ASME B & PV Code, Section III, all penetration reinforcing plates and the weldment of the pipe closure to it are shop stress relieved as a unit. This code requirement and the grouping of penetrations into large shop assemblies permit a minimum of field welding at penetrations.

5.1.4.4 Prestress Losses

In accordance with Section B.l.8 of the Design Criteria, Appendix 5B, the following categories and values of prestress losses are considered in the design:

Type of Loss	Assumed Value
Seating of Anchorage	None
	f_{cpi} in/in
Elastic Shortening of Concrete	3.0×10^{6}
Creep of Concrete	0.433×10^{-6} in/in/psi
Shrinkage of Concrete	100×10^{-6} in/in
Relaxation of Prestressing Steel[(1)]	8% of $0.65 f_s$ = 12.5 Ksi
Frictional Loss	K = 0.0003, μ = 0.156

There is no allowance for the seating of the BBRV anchor since no slippage occurs in the anchor during transfer of the tendon load into the structure. Sample lift-off readings will be taken to confirm that any seating loss is negligible.

The loss of tendon stress due to elastic shortening is based on the change in strain in the initial tendon relative to the last

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[(1)]Evaluations performed during the 20th year tendon surveillance determined that tendon wire relaxation loss is approximately 12%. For further details see safety evaluation JPN-PTN-SECJ-94-027 (Reference 9 in Section 5.1.3) and supporting documentation.

tendon stressed.

The concrete study conducted at the University of California, Appendix 5D, indicates an actual creep value of 0.340 x 10^{-6} in/in/psi. Conversion of the unit creep data to hoop, vertical and dome stress gives these values of stress loss in the tendons:

Hoop	- 14.8 Ksi
Vertical	- 7.4 Ksi
Dome	- 14.8 Ksi

A single creep loss figure of 650 x 10^{-6} in/in at 1500 psi (f_{cpi}) was used throughout the structure. This results in a prestress loss of 19.2 ksi.

The value used for shrinkage loss represents only that shrinkage that could occur after stressing. Since the concrete is well aged at the time of stressing, little shrinkage is left to occur and add to prestress loss.

The value of relaxation loss is based on information furnished by the tendon system vendor, the Prescon Corporation, on the basis of the tests conducted by Shinko Company, the Japanese supplier of wire for the containment tendons.

Frictional loss parameters for unintentional curvature (K) and intentional curvature (μ) are based on full-scale friction test data. This data indicates actual values of K = 0.0003 and μ = 0.125 versus the design values of K = 0.0003 and μ = 0.156.

As part of the Reactor Vessel Closure Head Replacement, a temporary construction opening was created and repaired in the containment building. This construction opening required replacement of tendons and concrete located within the opening area with new material. This new material was designed and furnished to perform 100% compatible with the existing structure. As previously stated for the original construction, loss of prestress over time was attributed to creep, concrete shrinkage and tendon wire relaxation properties. Based on testing, replacement material values for these properties falls within the bounds of the original design. These values were listed below:

Type of Loss	Value
Creep of Concrete (30 year)	0.386 x 10^{-6} in/in/psi
Shrinkage of Concrete	(0.04% expansive)
Relaxation of Prestressing Steel (30year)	6.82%

Creep of concrete values were based on a concrete study performed by S&ME, Inc (Knoxville, TN). Conversion of the unit creep data to vertical and hoop gives the following 30 year values of stress loss in the tendons: Hoop-16.19 ksi, Vertical-8.0 ksi.

Concrete shrinkage losses were negligible due to the use of an expansive concrete mix. An assumed value of -3.0 ksi was conservatively used in loss comparisons.

The value of relaxation loss was based on information furnished by the tendon system vendor, Precision Surveillance Corporation, based on tests conducted by the supplier of wire, KISWIRE, Ltd.

Assuming that the jacking stress for the tendons is 0.80 f'_s or 192,000 psi and using the above prestress loss parameters, the following tabulation shows the magnitude of the design losses and the final effective prestress at end of 40 years for a typical dome, hoop, and vertical tendon.[5]

	Dome (Ksi)	Hoop (Ksi)	Vertical (Ksi)	Allowable (Ksi)
Temporary Jacking Stress	192	192	192	192
Friction Loss	19	21.3[1]	21	
Seating Loss	-	0	0	
Elastic Loss (average)	14.7	15.3	6.6	
Creep Loss	19.2	19.2	19.2[4]	
Shrinkage Loss	3.0	3.0	3.0	
Relaxation Loss[3]	12.5	12.5	12.5	
Final Effective Stress [2]	123.6	120.7	129.7	144.0

(1) Average of adjacent tendons

(2) This force does not include the effect of pressurization which increases the prestress force.

(3) See footnote (1) in listing at beginning of Section 5.1.4.4.

(4) To determine tendon surveillance lift-off acceptance criteria, the creep loss for the vertical tendons has been adjusted. For further details, see Reference 11 of safety evaluation JPN-PTN-SECJ-94-027 (Reference 9 on Page 5.1.3-38).

(5) The 40-year prestress losses depicted in the tabulation were utilized to calculate 60-year prestress losses for license renewal and the 80-year prestress losses for subsequent license renewal. C31

To provide assurance, of achievement of the desired level of Final Effective

Prestress and that ACI 318-63 requirements are met, a written procedure was prepared for guidance of post-tensioning work. The procedures provided nominal values for end anchor forces in terms of pressure gage readings for calibrated jack-gage combinations. Force measurements were made at the end anchor, of course, since that is the only practical location for such measurements.

The procedure required the measured temporary jacking force, for a single tendon, to approach but not exceed 850 kips. ($0.8f'_s$). Thus the limits set by ACI-318-63 2606 (a) 1, and of the prestressing system supplier, were observed. Additionally, benefits were obtained by in place testing of the tendon to provide final assurance that the force capability exceeded that required by design. During the increase in force, measurements were required of elongation changes and force changes in order to allow documentation of compliance with ACI 318-63 2621 (a). The procedures required that the prestressing steel be installed in the sheath before stressing for a sufficient time period that the temperatures of the prestressing steel and concrete reached essential equilibrium, to establish conformance with ACI 318-63 2621 (e). The jacking force of $0.8f'_s$ further provided for a means of equalizing the force in individual wires of a tendon to establish compliance with ACI 318-63 2621 (b). The procedures required compliance with ACI 318-63 such that, if broken wires resulted from the post-tensioning sequence, compliance with section 2621 (d) was documented. Each of the above procedures contributed to assurance that the desired level of Final Effective Prestress would be achieved.

The requirements of ACI 318-63 2606 (a) 2 state that f_s should not exceed $0.7f'_s$ for "post-tensioning tendons immediately after anchoring".

Tendons affected by the Reactor Vessel Closure Head Replacement construction opening were retensioned in accordance with the original tensioning procedures.

Paragraph 2606 (a) 2 of ACI 318-63 refers to "tendons" rather than to an individual tendon. Further, the paragraph does not refer to the location to be considered for the determination of f_s in the manner, for example, of the "temporary jacking force" referred to in paragraph 2606 (a) 1. Two interpretations were therefore required. Both interpretations had to consider the effect of the resultant actions on both the prestressing system and structure.

The first interpretation was that the location for measurement of the seating force, used in calculating f_s, was at the end anchor and just subsequent to the measurement of the "temporary Jacking force" referred to in ACI 2606 (a) 1. The advantages of this location are several. One is that it is a practical one and thus the possibility for achieving valid measurements could be made without the added complexity of additional measuring devices. The third advantage is that measurements at this location provide assurance that the calculated f_s does not anywhere exceed the maximum f_s ($0.8f'_s$) to which that tendon has been subjected.

Several possible cases were considered for the second interpretation so as to allow anchoring of an individual tendon without exceeding the requirement stated for "tendons" collectively in ACI 318-63 2606 (a) 2. One such case assumed that the anchoring force for the typical tendon was that for a tendon anchored midway through the prestressing sequence. It further assumed that the losses to be assumed were one half of the sum of elastic losses, and of the creep, shrinkage and relaxation predicted to occur during the entire prestressing sequence. This interpretation however was not considered to be practical nor enforceable since it resulted in changing the seating forces as the actual, (as compared to the scheduled), time length of the prestressing period was dictated by weather, and manpower availability.

Another case considered was that of anchoring each tendon at a measured force of 850 kips ($0.8f'_s$). Although there was no apparent detrimental effect to the prestressing system or structure, insertion of shims would be almost impossible. Further, it was concluded that this case would not establish compliance with ACI 318-63.

The case adopted was to seat each tendon with a measured "pressure" reading for the jack, at "lift off" of the end anchor, of 775 kips (between 0.72 and 0.73 f'_s). This procedure had several advantages.

One advantage was that the force on the containment and the tendon was within the bounds of those for which it had been tested and resulted in no known detrimental effects. The second advantage was that the stressing procedure was simplified, since the stressing crews did not have to accommodate a large number of different anchoring force requirements. The third advantage was that, at the completion of stressing the last tendon, the expected losses were such that the average f_s at the end anchors of the tendons would be less than $0.7f'_s$, thus establishing compliance with ACI 318-63 2606 (a) 1 and 2. The fourth advantage was that the percentage loss of prestressing force was less than would be the case if the tendons were anchored in such a manner the calculated value of f_s nowhere exceeded $0.7f'_s$.

The latter advantage deserves special mention since it plays a strong role in assuring that the Final Effective Prestress

equaled or exceeded the desired values. For example, if the f_s at anchorage of the tendons were 0.1 f'_s, creep and shrinkage of concrete could result in the loss of almost all of the prestressing force. Assuming that the total losses due to creep, shrinkage and elastic shortening equals 0.1 f_s, then the Final Effective Prestress would be 20 % less than an Initial Prestress equivalent to 0.5 $f5'_s$. If the Initial Prestress were equivalent to 0.7 $f5'_s$, the Final Effective Prestress, neglecting relaxation for the moment, would be about 86% of the Initial Prestress. Clearly, the assurance (that the concrete creep and shrinkage losses have been properly accounted for) increases as the f_s for the anchored tendons and tendon increases. However, this design was
committed to meeting the ACI 318-63 requirement and the anchorage force for the <u>tendons</u> was kept at or below 0.7 f'_s in accordance with the interpretation described.

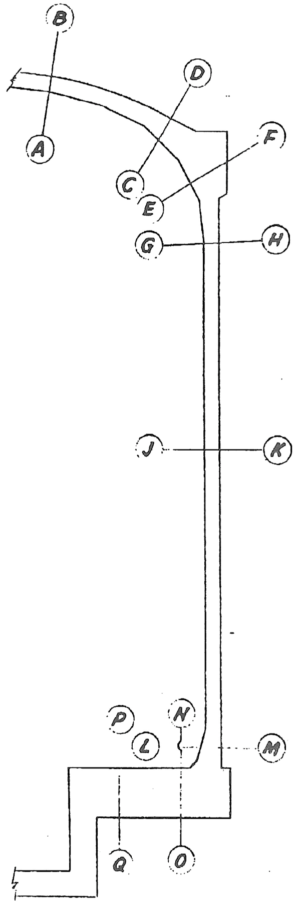
5.1.4.5 <u>Miscellaneous Considerations</u>

In various cases, it has been the designer's decision to provide structural adequacy beyond that required by the design criteria submitted in the PSAR. Those cases are as follows:

(a) Section B.1.5 of Appendix 5B, Design Criteria, requires a minimum of 0.15% bonded steel reinforcing (steel In two perpendicular directions) for any location. At the base of the cylinder, the controlling design case requires 0.25% vertical reinforcing. As result of pursuing the recommendation of the AEC staff to further investigate current research on shear in concrete, several steps have been taken:

(1) The work of Dr. Alan H. Mattock has been reviewed and he has been retained as a consultant on the implementation of the current research being conducted under his direction. The criteria has been updated in accordance with his recommendations.

(2) Concurrently with reviewing Dr. Mattock's work, the firm of T. Y. Lin, Kulka, Yang and Associate has been consulted to review the detailed design of the cylinder to slab connection. Based on their recommendation, approximately 0.5% reinforcing has been used rather than the 0.25% reinforcing indicated by the detailed design analysis for the vertical wall dowels. This increase would insure that there is sufficient flexural steel to place the section within the lower limits of Mattock's test data (approximately 0.3%) to prevent flexural cracking from adversely affecting the shear capability of the section.



KEY ELEVATION

SHOWING LOCATIONS OF REFERENCE SECTIONS

Structural Data

Location	Concrete		Reinforcing Steel		
	f'_c-psi	t-in	Type	P_m-%	P_h-%
A	5000	39	A615 GR. 40	.066	.066
B	5000	39	A615 GR. 40	.256	.256
C	5000	60	A615 GR. 40	.100	.090
D	5000	60	A615 GR. 40	.237	.211
E	5000	148	A615 GR. 40	.038	.038
F	5000	148	A615 GR. 60	.087	.087
G	5000	50	A615 GR. 40	.193	.321
H	5000	50	A615 GR. 40	.630	.193
J	5000	45	A615 GR. 40	-	-
K	5000	45	A615 GR. 60	.185	.185
L	5000	78	A615 GR. 60	.500	-
M	5000	78	A615 GR. 60	.500	.174
N	4000	126	A615 GR. 60	.381	.211
O	4000	126	A615 GR. 60	.381	.265
P	4000	126	A615 GR. 60	.293	.212
Q	4000	126	A615 GR. 60	.293	.264

NOTES

1. Loading cases I, II, & III are working stress analysis where as loading cases IV, V, & VI are yield stress analysis.

2. For notation and allowable stresses see Sheet 2.

3. The steel stresses shown for the load cases including T_A are based on cracked section analysis.

4. Deviations in allowable stresses are in accordance with Section B.1 of Appendix 5B.

5. All concrete extreme fiber stresses (σ_e) are shown for the inside surface. Outside surface stresses are indicated by ( ). The stresses listed are the controlling stresses for that section.

6. Computed vs. allowable ratios for cases IV, V and VI include appropriate Ø factors, e.g. $\frac{\sigma_a}{\phi\, \epsilon_a}$

7. Allowable shear stresses include stirrups wherever applicable.

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FLORIDA POWER & LIGHT COMPANY
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CONTAINMENT STRUCTURE SUMMARY OF CONCRETE & REINFORCING STEEL STRESSES

TABLE 5.1.4-1
Sheet 1

C29

NOTATION

D	DEAD LOAD
F	PRESTRESS
P	INTERNAL PRESSURE
E	DESIGN EARTHQUAKE
E'	NO-LOSS-OF-FUNCTION EARTHQUAKE
T_A	ACCIDENT TEMPERATURE
f'_c	CONCRETE STRENGTH
f_y	STEEL REINFORCING YIELD STRENGTH
f_a	ALLOWABLE CONCRETE AXIAL STRESS
f_{ce}	ALLOWABLE CONCRETE COMBINED AXIAL & FLEXURAL STRESS
v	ALLOWABLE CONCRETE SHEAR STRESS INCLUDING STIRRUPS IF APPLICABLE
f	ALLOWABLE STEEL STRESS
σ_a	NOMINAL MEMBRANE STRESS
σ_c	COMBINED AXIAL & FLEXURAL NOMINAL STRESS
τ	NOMINAL SHEAR STRESS
h	SUBSCRIPT INDICATING HOOP DIRECTION
m	SUBSCRIPT INDICATING MERIDIONAL DIRECTION
P_h	HOOP STEEL PERCENTAGE
P_m	MERIDIONAL STEEL PERCENTAGE
+	TENSILE STRESSES
-	COMPRESSIVE STRESSES

ALLOWABLE STRESSES

	WORKING STRESS DESIGN	YIELD STRESS DESIGN
SHELL CONCRETE	f_a = 1500 psi	fa = Øa f'_c = (0.85) (5000) = 4,250 psi
	f_{ce} = 2250 psi	fce = Øce f'_c = (0.90) (5000) = 4,500 psi
	f_{ce_i} = 3000 psi	
BASE CONCRETE	f_a = 1200 psi	fa = Øa f'_c = (0.85) (4000) = 3,400 psi
	f_{ce} = 1800 psi	fce = Øce f'_c = (0.90) (4000) = 3,600 psi
STEEL A615 GR. 40	f_s = 20,000 psi	fs = Ø fy = (0.90) (40,000) = 36,000 psi
A615 GR 60	f_s = 30,000 psi	fs = Ø fy = (0.90) (60,000) = 54,000 psi

Note: For allowable shear stress see Appendix 5B.

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TURKEY POINT PLANT

CONTAINMENT STRUCTURE SUMMARY OF CONCRETE & REINFORCING STEEL STRESSES

TABLE 5.1.4-1
Sheet 2

C29

D & F INITIAL (Stresses in psi) Case 1

	Section	CONCRETE							LINER PLATE	
		Meridional			Hoop			Shear	Meridional	Hoop
		σ Outside	σ Inside	σ Axial	σ Outside	σ Inside	σ Axial	τ	σ_m	σ_h
Shell	A - B	-1190	-987	-1082	--986	-758	-866	63	-24,900	-20,170
Shell	C - D	-334	-1020	-733	-337	-420	-397	52	-26,400	-13,870
Shell	E - F	-249	-597	-454	-292	-369	-343	60	-15,700	-11,570
Shell	G - H	-3	-839	-471	-296	-476	-408	84	-27,200	-16,410
Shell	J - K	-614	-632	-630	-1079	-1152	-1140	0	-18,400	-33,560
Shell	L - M	+81	-1120	-480	-6	-243	-135	156	-31,400	-10,700
Base	N - O	+14	-389	-90	-148	-139	-140	57	-9,610	-4,010
Base	P - Q	-116	-46	-81	-158	+18	-69	27	-2,030	-520

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TURKEY POINT PLANT

CONTAINMENT STRUCTURE - SUMMARY OF CONCRETE REINFORCING STEEL & LINER PLATE STRESSES

TABLE 5.1.4-1
Sheet 3

C29

SECTION	LOAD CASE	CONCRETE								REINFORCING STEEL				LINER PLATE	
		COMPUTED					COMPUTED VS. ALLOWABLE			COMPUTED		COMPUTER VS. ALLOWABLE		COMPUTED	
		σ_{cm}	σ_{ch}	σ_{pm}	σ_{ph}	τ	σ_c/fce	σ_c/fa	τ/v	σ_m	σ_h	σ_m/fs	σ_h/fs	σ_m	σ_h
A - B	II - D+F+T_A	-1,398	-1,106	-1290	-980	72	.465	.860	.421	-20,120	+9,900	-	.330	-47,800	-41,800
	III - D+F+1.15 P	-348	(-274)	-360	-287	25	.115	.240	.075	-	-	-	-	-7,400	-5,700
	IV - D+F+P+T_A	-741	-608	-416	-310	40	.330	.278	.270	-	-	-	-	-31,400	-24,600
	V - 1.05 D+F+1.5 P+T_A	-427	-364	-46	+5	24	.095	.011	.156	18,200	33,000	.500	.915	-10,300	-9,700
	VI - 1.05 D+F+1.25 P+1.25 E +T_A	-586	-484	-236	-170	34	.130	.055	.221	2,500	19,500	.695	.541	-22,200	-23,300
	VII - 1.05 D+F+P+E'+T_A	-743	-610	-416	-310	44	.165	.098	.288	9,100	8,100	.253	.225	-30,340	-25,440
C - D	II - D+F+T_A	-1,201	-1,413	-744	-450	48	.471	.496	.889	+2,360	+9,650	.079	.320	-41,500	-41,500
	III - D+F+1.15 P	(-490)	-373	-372	-347	85	.220	.284	.218	-	-	-	-	-4,900	-7,900
	IV - D+F+P+T_A	-664	-946	-415	-442	140	.420	.294	.870	-	-	-	-	-32,000	-32,160
	V - 1.05 D+F+1.5 P+T_A	-312	-894	-240	-403	185	.199	.095	.545	-	-	-	-	-24,200	-35,900
	VI - 1.05 D+F+1.25 P+1.25 E +T_A	-492	-920	-326	-423	164	.204	.100	.443	-	-	-	-	-28,100	-31,230
	VII - 1.05 D+F+P+E'+T_A	-666	-946	-415	-442	143	.210	.104	.362	-	-	-	-	-32,000	-32,000
E - F	II - D+F+T_A	-787	-1003	-425	-355	60	.334	.284	.476	-4,340	-1,670	-	-	-33,500	-35,800
	III - D+F+1.15 P	-475	(-390)	-320	-374	56	.211	.249	.228	-	-	-	-	-900	-6,600
	IV - D+F+P+T_A	-435	-923	-338	-435	57	.410	.290	.514	-	-	-	-	-27,900	-34,400
	V - 1.05+F+1.5 P+T_A	-390	-880	-270	-422	61	.196	.100	.176	-	-	-	-	-16,700	-35,200
	VI - 1.05 D+F+1.25 P+1.25 E +T_A	-394	-900	-304	-429	60	.200	.100	.217	-	-	-	-	-25,100	-35,800
	VII - 1.05 D+F+P+E'+T_A	-435	-921	-340	-436	58	.202	.103	.158	-	-	-	-	-28,000	-38,800

NOTE:
The containment structure re-analysis (completed in 1994) encompasses the cylindrical shell and dome areas. With the exception of the results of Load Case III, refer to Appendix 5H for the updated stress results/information relative to the cylindrical shell and dome areas. The remaining information on this sheet is considered historical for the original analysis.

TBL.00001

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FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE - SUMMARY OF CONCRETE REINFORCING STEEL & LINER PLATE STRESSES

TABLE 5.1.4-1
Sheet 4

C29

SECTION	LOAD CASE	CONCRETE								REINFORCING STEEL				LINER PLATE	
		COMPUTED					COMPUTED VS. ALLOWABLE			COMPUTED		COMPUTER VS. ALLOWABLE		COMPUTED	
		σ_{em}	σ_{eh}	σ_{bm}	σ_{bh}	τ	σ_e/f_{ce}	σ_b/f_a	τ/v	σ_m	σ_h	σ_m/f_s	σ_h/f_s	σ_m	σ_h
G - H	II - D+F+T_A	-1,613	-1,572	-515	-444	89	.538	.334	.760	+22,300	+25,300	.743	.843	-52,000	-51,100
	III - D+F+1.15 P	-278	-259	-126	-235	23	.123	.157	.589	-	-	-	-	-7,100	-6,000
	IV - D+F+P+T_A	-803	-796	-180	-287	7	.356	.191	.140	20,000	19,600	1,000	.980	-27,900	-49,300
	V - 1.05 D+F+1.5 P+T_A	-390	-622	-10	-177	34	.138	.041	.047	27,500	23,300	.765	.647	-16,700	-49,600
	VI - 1.05 D+F+1.25 P+1.25 E +T_A	-560	-697	-95	-232	13	.155	.054	.022	24,500	23,900	.680	.633	-25,100	-49,650
	VII - 1.05 D+F+P+E'+T_A	-736	-772	-180	-287	7	.172	.067	.060	26,600	19,600	.740	.544	-28,000	-49,300
J - K	II - D+F+T_A	-1,546	-2,351	-825	-1440	0	.784	.960	0	-2,060	-10,560	-	-	-49,700	-69,160
	III - D+F+1.15 P	-219	-370	-232	-371	0	.165	.247	0	-	-	-	-	-5,000	-7,800
	IV - D+F+P+T_A	-723	-892	-285	-480	0	.396	.320	0	23,900	-	.795	-	-36,080	-42,800
	V - 1.05 D+F+1.5 P+T_A	-480	-382	-90	-70	0	.107	.021	0	21,200	16,400	.590	.456	-34,900	-24,500
	VI - 1.05 D+F+1.25 P+1.25 E +T_A	-590	-612	-190	-280	0	.136	.066	0	14,100	5,000	.392	.139	-33,940	-36,400
	VII - 1.05 D+F+P+E'+T_A	-700	-845	-291	-490	0	.188	.115	0	8,000	-	.222	-	-32,000	-30,000
L - M	II - D+F+T_A	-1,051	-919	-510	-144	195	.350	.340	.515	-2,820	+22,520	-	.751	-35,200	-36,200
	III - D+F+1.15 P	-202	-80	-208	-86	42	.089	.139	.126	-	-	-	-	-6,000	-3,400
	IV - D+F+P+T_A	-672	-637	-268	-174	75	.298	.179	.216	5,000	13,900	.167	.464	-30,600	-28,600
	V - 1.05+F+1.5 P+T_A	-242	-570	-126	-150	14	.127	.035	.187	700	9,700	.019	.270	-18,300	-24,200
	VI - 1.05 D+F+1.5 P+T_A	-731	-701	-345	-270	81	.163	.081	.670	6,500	20,100	.180	.556	-23,600	-20,200
	VII - 1.05 D+F+P+E'+T_A	-1095	-775	-490	-340	130	.244	.115	.373	13,000	33,800	.361	.940	-23,100	-19,500

NOTE:
The containment structure re-analysis (completed in 1994) encompasses the cylindrical shell and dome areas. With the exception of the results of Load Case III, refer to Appendix 5H for the updated stress results/information relative to the cylindrical shell and dome areas. The remaining information on this sheet is considered historical for the original analysis.

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FLORIDA POWER & LIGHT COMPANY
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CONTAINMENT STRUCTURE - SUMMARY OF CONCRETE REINFORCING STEEL & LINER PLATE STRESSES
TABLE 5.1.4-1
Sheet 5

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SECTION	LOAD CASE	CONCRETE								REINFORCING STEEL				LINER PLATE	
		COMPUTED					COMPUTED VS. ALLOWABLE			COMPUTED		COMPUTED VS. ALLOWABLE		COMPUTED	
		σ_{cm}	σ_{ch}	σ_{am}	σ_{ah}	τ	σ_c/f_c	σ_a/f_a	τ/v	σ_m	σ_h	σ_m/f_s	σ_h/f_s	σ_m	σ_h
N - O	II - D + F + T_A	-650	-395	-97	-229	-138	.361	.072	.522	+15,820	+4,560	.527	.152	-15,500	-8,540
	III - D + F + 1.15 P	-191	- 49	- 3	-24	79	.106	.020	.320	1,600	-	.053	-	- 2,500	-1,800
	IV - D + F + P + T_A	-274	-395	-50	-48	83	.220	.042	.327	-	9,800	-	.326	- 5,200	-6,200
	V - 1.05 D + F + 1.5 P + T_A	- 70	-456	-12	- 3	144	.127	.003	.558	-	12,600	-	.233	+ 860	-4,500
	VI - 1.05 D + F + 1.25 P + 1.25 E + T_A	-314	-380	-42	-58	179	.106	.017	.720	1,200	14,800	.022	.274	-5,160	-4,700
	VII - 1.05 D + F + P + E' + T_A	-492	-301	-67	-99	182	.137	.029	.716	8,400	15,500	.155	.287	-9,050	-5,400
P - Q	II - D + F + T_A	-406	-193	-84	-69	-12	.226	.047	.295	+2,200	+6,080	.073	.203	-8,030	-4,420
	III - D + F + 1.15 P	-193	- 90	- 5	13	0	.107	.011	0	4,000	2,240	.133	.075	-3,200	-1,100
	IV - D + F + P + T_A	-593	-438	-84	7	16	.330	.070	.206	21,200	19,300	.706	.643	-10,400	-6,400
	V - 1.05 D + F + 1.5 P + T_A	-659	-484	-48	35	30	.183	.014	.612	30,900	28,900	.572	.535	-14,450	-7,800
	VI - 1.05 D + F + 1.25 P + 1.25 E + T_A	-849	-565	-74	3	41	.236	.022	.773	39,000	33,600	.721	.621	-18,800	-9,450
	VII - 1.05 D + F + P + E' + T_A	-935	-598	-96	-29	40	.260	.028	.500	41,700	32,300	.770	.597	-20,800	-11,550

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FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE - SUMMARY OF CONCRETE REINFORCING STEEL & LINER PLATE STRESSES

TABLE 5.1.4-1
Sheet 6

5.1.5 PENETRATIONS

5.1.5.1 ACCESS LOCKS AND DOORS

(a) The personnel access lock consists of a 9'-0" diameter steel cylinder with a 3'-4" x 6'-8" door at each end, mechanically interlocked so that one door cannot be opened unless the other is closed. Technical Specifications allow doors at both ends to be open during fuel movements or core alterations when certain proceduralized administrative controls are in place. Doors are pressure-seating type, opening towards the inside of containment.

The position of the far door is indicated at each door and also remotely in the control rooms. Each door is provided with double gaskets.

The lock is shop tested for structural integrity in accordance with ASME B & PV Code, Section III proof test requirements.

A locking mechanism, resistant to forced entry, operated by a special key kept under administrative control, is installed. During normal operation the personnel access lock will be kept locked, and access to the containment will be restricted to authorized personnel. When more frequent access to the containment is necessary, requiring the access lock to remain unlocked, alternative security procedures will be in effect.

(b) The personnel escape lock has a 5'-0" diameter barrel, with a 2'-6" diameter circular door at each end. Mechanical means are provided to enable the operation of either door from inside and outside of the containment structure, as well as from inside of the lock.

A locking mechanism, resistant to forced entry from outside the containment, operated by a special key kept under administrative control, is installed and will be kept locked except for entry by authorized personnel. The design of the locking mechanism will permit unrestricted egress from the containment.

(c) A 14'-0" diameter equipment hatch provides access to the containment interior at the mezzanine level. The door is provided with double gaskets and is secured by bolts which can be opened only from the inside during reactor shutdown. The integrity of the seals can be checked from the outside by pressurizing the annular space between the two gaskets.

Figure 5.1-5 shows the principal features of the equipment hatch.

5.1.5.2 PIPING AND VENTILATION PENETRATIONS

All piping and ventilation penetrations are of the rigid welded type and are solidly anchored to the containment wall, thus precluding any requirement for expansion bellows. Penetrations and anchorages are designed for the forces and moments resulting from operating conditions or postulated pipe rupture as shown in Table 5.4.3-1. External guides and stops are provided as required to limit displacements, and resist bending, and torsional moments to prevent rupture of the penetrations and the adjacent liner plate. Each penetration assembly and its connection to the piping are designed to withstand the maximum pressure in the open end annulus between the pipe and the penetration assembly resulting from the rupture of the pipe.

For typical details of piping penetrations see Figure 5.1-2.

5.1.5.3 ELECTRICAL PENETRATIONS

Electrical penetrations consist of carbon steel pipe canisters with stainless steel or carbon steel header plates welded to each end. High voltage conductors utilize single conductor hermetically sealed glass or ceramic bushings welded to both header plates. Thus, each canister provides a highly effective barrier against leakage. A flange on each canister is welded to a carbon steel sleeve penetrating the containment wall. Heat conduction and radiation paths are sufficient to prevent damage to seals or conductors during field welding of the canisters to the containment liner.

The canister with two welded headers permits pressure and leakage tests to be performed simply and reliably both at the shop and after installation. A tap, convenient to the exterior of the containment is provided for pressure testing the canister. The welds of the canister to the nozzle are tested during the containment Integrated Leak Rate Test.

For typical details of electrical penetrations see Figure 5.1-3.

5.1.6 CONSTRUCTION

5.1.6.1 Construction Methods

a. Governing Codes:
The following codes of practice are used to establish standards of construction procedures:

ACI 301-66	Specification for Structural Concrete for Buildings
ACI 318-63	Building Code Requirements for Reinforced Concrete
ACI 347-63	Recommended Practice for Concrete Formwork
ACI 605-59	Recommended Practice for Hot Weather Concreting
ACI 613-54	Recommended Practice for Selecting Proportions for Concrete
ACI 613-59	Recommended Practice for Measuring, Mixing and Placing Concrete
ACI 315-65	Manual of Standard Practice of Detailing Reinforced Concrete Structures
ASME -	Boiler and Pressure Vessel Code, Sections III, VIII, and IX
AISC -	a) "Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings", adopted April 17, 1963
	b) "Code of Standard Practice for Steel Buildings and Bridges", revised February 20, 1963
PCI -	Inspection Manual

b. Concrete:

Cast-in-place concrete is used for the containment shell and base slab. In general, the concrete placement in the walls is done in ten feet high lifts with vertical joints at the radial center line of each of the six buttresses. Cantilevered jump forms on the exterior face, and the steel liner on the interior face serve as forms for the wall concrete.

The dome liner plate, temporarily supported by 18 radial steel trusses and purlins, serves as an inner form for the initial 8" thick pour in the dome. The weight of the subsequent pour is supported by the initial 8" pour. The trusses are lowered free of the dome liner plate after the initial 8" of concrete has reached design strength and then the balance of the dome concrete is placed.

An 18" thick cover concrete is placed over the floor liner plate after the plate is installed and the welds tested. The horizontal and the vertical construction joints are prepared for the next pour by sandblasting, cleaning and wetting.

Cast-in-place concrete was used for the repair of the Reactor Vessel Closure Head Replacement construction opening. The concrete closure placement was made in one continuous lift. The steel liner plate serves as the interior concrete form. Exterior formwork was constructed and tied to the liner plate.

c. Reinforcing Steel and "CADWELD" Splices:

Prior to concrete placing, visual inspection of the shop fabricated reinforcing steel is performed to ascertain dimensional conformance with the design specifications and drawings. This is followed by a check "in place" performed by the placing inspector to assure the dimensional and location conformance.

Whenever required, mechanical splices are made by the Cadweld Process using clamping devices, sleeves, charges, etc., as specified by the manufacturer for the "T" series connections. All personnel engaged in making the splices are trained and supervised by the manufacturer's representative and have to pass all the necessary qualification tests and procedures before production splicing. Prior to splicing operations, bar ends are inspected for damaged deformations and are power brushed to remove all loose mill scale, rust and other foreign material. Immediately before the splice sleeve positioning, bar ends are preheated to ensure complete absence of moisture.

All completed splices are visually inspected at both ends of the splice for sound and nonporous filler material. The filler metal should be present for the full 360 degrees. It is usually recessed 1/4" from the end of the sleeve due to the packing material. Based on test results, an acceptance criteria has been established for the voids in the filler metal at either or both ends of the splice sleeve such that the void area shall not exceed three (3) square inches per end of splice. The area of the void is assumed to be the circumferential length as measured at the inside face of the sleeve times the maximum depth of wire probe minus 3/16". The minimum strength of the Cadweld joints, as verified by tests on sample bars, is equal to or greater than 125% of the ASTM specified minimum yield strength of the reinforcing bars, which is in accordance with the ACI-318.

Reactor Vessel Closure Head Replacement Mechanical Rebar Splices were constructed using Bar Grip XL cold-swaged couplers made by Barsplice Products Incorporated. This product was qualified as a full tension-compression splice, meeting the requirements of ASME section III, Division 2, Subsection CC, and was capable of developing not less than 125% of the specified yield strength of the bars in question.

All personnel engaged in the mechanical splicing operation were trained to the manufacturer's procedure and must pass all the necessary qualification tests before production splicing. Bar ends were measured and marked to ensure proper engagement with the coupler sleeve.

Where a mechanical splice was not used, a direct butt fusion weld splice was used. These splices were welded and inspected in accordance with AWS D1.4-98, Structural Welding Code –Reinforcing Steel. All personnel engaged in the splicing operation were qualified per an approved Welding Procedure Specification in accordance with AWS D1.4-98.

For both mechanical and welded splices, the minimum strength of sample bars (sister splices) as verified by tests was equal to or greater than 125% of the ASTM specified minimum yield strength of the sample bars.

d. Post-tensioning System

The post-tensioning used is the BBRV system as furnished by the Prescon Corporation. For typical details, see Figure 5.1-19.

Each tendon nominally consists of ninety (90)-1/4" diameter button headed wires, two stressing washers and two shims. The tendons sheathing system consists of spirally wound sheet metal tubing connected to a mild steel "Trumplate" (bearing plate and trumpet).

Tendons are delivered to the site coated with a temporary rust inhibitor and encased in polyethylene bags. Each tendon is shop fabricated to its exact length, with one end shop button headed and threaded through the stressing washer and the other end non-headed.

The tendon installation and stressing operations are carried out as follows:

1.a. For Reactor Vessel Closure Head Replacement containment opening closure, a sheathing rabbit is run through the sheathing both prior to and following placement of the concrete.

1.b. To assure a clear passage for the tendons, a "sheathing rabbit" is run through the sheathing both prior to and following placement of the concrete.

2. The tendons are uncoiled and pulled through the sheathing, non-headed end first.

3. The non-headed end of the tendon is pulled out with enough length exposed so that field attachment of the stressing washer and button heading may be performed. To allow this operation, the trumpet on the opposite end has an enlarged diameter to permit the pulling in of the shop headed end with its stressing washer.

4. The stressing washer is attached and the tendon wires button headed.

5. The shop-headed end of the tendon is pulled back, shims are installed, and the stressing jack is attached to the field headed end.

6. The stressing is done by jacking to the required stressing force, and placing incremental shims of predetermined thickness corresponding to the calculated elongation. Proper tendon stress is assured by comparing the jack pressure and the tendon elongation against previously calculated values. The vertical tendons are stressed from one end, while the horizontal and dome tendons are stressed from both ends.

7. The filler material is pumped into the sheathing and the filler retention cap is firmly attached. In its final position, the cap is full of filler material and completely covers the stressing washer and the shims.

Tendons replaced during the Reactor Vessel Closure Head Replacement were furnished by Precision Surveillance Corporation. The replaced components were 100% compatible replacements for the original Prescon manufactured tendons.

Where required, lap splices on reinforcing steel installed as part of the Reactor Vessel Closure Head Replacement containment opening closure were Class B full tension lap splices. A513 Type 5 carbon steel tube was used for the replacement tendon sheathing in closing the construction opening following the Reactor Vessel Closure Head Replacement.

During Reactor Vessel Closure Head Replacement, selected tendons outside of the temporary construction opening were de-tensioned. Tendons located within the construction opening were de-tensioned and removed. At the completion of construction opening closure, tendons are re-tensioned. Tensioning to 80% of F_{pu} and lock-off at 70% F_{pu} was performed for all vertical tendons and tensioning to 80% of F_{pu} and lock-off at 58.3% of F_{pu} for new horizontal tendons and tensioning to 75% of F_{pu} and lock-off at 58.3% of F_{pu} for existing horizontal tendons was performed.

e. Liner Plate:

Construction of the liner plate conforms to the applicable portions of Part UW of Section VIII, paragraphs UW-26 through UW-38 inclusive, applied in their entirety. In addition, the qualification of all welding procedures and welders are performed in accordance with part A of Section IX of the ASME B&PV Code.

All stiffener angle welding is visually inspected to ensure that quality and general workmanship meet the requirements of the applicable welding specification.

The erection sequence of the liner plate is as follows: The wall liner plate is erected in 60 degree segments and 10 feet high lifts. This pattern is followed up to the dome springline and then the steel dome trusses are installed followed by the dome liner plate. During the period that the wall liner plate is being erected, the floor liner is placed and welded.

The tolerances on erection are as follows: The radial location of any point on the liner plate may not vary from design radius by more than ± 1 1/2". A 15 foot long template curved to the required radius is used to verify that the following tolerances are not exceeded:

1. A maximum 3/4" deviation when placed against the completed surface of the shell within a single plate section.
2. A maximum 1" deviation when placed across one or more welded seams.

Maximum inward deflection (toward the center of the structure) of the 1/4" plate between the vertical angle stiffeners is 1/16", when measured with a 15" straight edge placed horizontally and 1/8" placed across the weldseams at the buttresses.

Reactor Vessel Closure Head replacement Containment Opening Liner Plate Repair and Fabrication: The liner plate and backing stiffeners removed to facilitate reactor vessel closure head replacement were reused. Testing for the repair was per ASME Boiler and Pressure Vessel Code, Section XI, Subsection IWL, IWE, and IWA of the 1992 Edition with 1992 Addenda.

5.1.6.2 Materials

a. Concrete:

Ingredients	Applicable Specification
Cement	ASTM C-150-64 Florida Type II
Air Entraining Agent	ASTM C-260 -63T (Neutralized Vinsol Resin Airecon)
Water Reducing Agent	ASTM C-494-62T Type D (Retardwell, Union Carbide)
Aggregate	ASTM C-33-64 (Fine and Coarse Aggregate, Miami Oolite)

No Calcium Chloride is used in the concrete.

Strengths	
Base Slab	4000 psi at 28 days
Walls and Dome	5000 psi at 28 days
Interior Concrete	3000 psi and 5000 psi at 28 days, and 7500 psi at 90 days

Principal Placement Properties	
Slump	2" at form (with 3" slump permitted in limited areas of high congestion)
Slump Tolerance	ASTM C-94-65
Air Content	3-5% at mixer
Temperature	70F Maximum

b. Reinforcing Steel:

ASTM-A15 Intermediate Grade (A 615, GR. 40)
ASTM-A408 Intermediate Grade (A 615, GR. 40)
ASTM-A432 High Strength (A 615, GR. 60)

c. <u>Prestressing Tendons and Associated Hardware:</u>

Material	Material Specifications
Tendon Wires	ASTM - A42I Type BA
Bearing Plate	ASTM - A-107-C-1045
Stressing Washer	ASTM - A-107, C-1045
Shims	AISI - A-107, C-1045 special quality
Tendon Sheathing 24 gage	Ungalvanized ferrous metal

d. <u>Liner Plate:</u>

1/4" Liner plate conforms to ASTM Specification A- 36. Plates thicker than 1/4" conform to ASTM A-442 or A-516.

e. <u>Penetrations and Assemblies:</u>

Elements resisting containment pressure:

Seamless Pipe Material	ASTM - A333
Welded Pipe Material	ASTM - A155 Class I (where possible, made from A-516 plate)

Plate Material used for the following:

Airlocks and Equipment Hatch	ASTM-A-516 Fire Box Quality
Crane Bracket Reinforced Plate	
Liner Reinforced Plates at Penetrations	
Truss Brackets Reinforced Plate	

In all of the above materials, specimens are Charpy V-Notch impact tested and meet the requirements of Paragraph N-1211

(a) of Section III of the ASME B & PV Code at a test temperature of 0 F.

f. Major Component Supports:

Structural Shapes	ASTM - A-441, A-302 B Mayari R-50
Reactor Support Rollers	AISI 440C Modified
Reactor Support Bearing Plates	AISI 52100
Bolts	Maraging Nickel Steel

g. Structural Stiffeners and Anchors and Other Non-Pressure Miscellaneous Parts Conform to A-36 Material:

Penetration	ASTM - A- 307
Anchor Bolts	ASTM - A- 307
Welding Electrodes	ASTM - A- 233 Type E-6010 and E-7018
Filler Material	ASTM - A-558 and A-559
Truss Bolts	ASTM - A-325
Structural Steel for Crane Brackets Inserts and Supports	ASTM - A-36 (Original), A-992 (equiv), A-572 GR 50 (Equiv) C26

h. Sheathing Filler:

The tendon sheathing filler material used has the following specified limitations for deleterious water soluble salts:

	Allowable Maximum	Test Method
Chlorides (C1)	2 ppm	ASTM Method D-512-62T (Limit of Accuracy 0.5 ppm)

Nitrates (NO_3)	4 ppm	ASTM Method D-992-52 (Limit of accuracy 0.01 mg per liter)
Sulfides (S)	2 ppm	ASTM Method D-1255 (Limit of accuracy 1 ppm)

NO-OX-1D 490 and 500 are used as temporary rust inhibitors on the tendons and the interior of the sheathing.

"Visconorust 2090P-4" produced by Viscosity Oil is used as sheathing filler material and properties are listed in Table 5.1.6-1.

i. Concrete Mixes:

The concrete mix design and the measuring, mixing, and placing of concrete are based on the codes and specifications listed in Section 5.1.6.1.

Tests on the design mix have been run at the University of California, College of Engineering, Berkeley, by Prof. David Pirtz, to determine uniaxial creep, modulus of elasticity, Poisson's ratio, coefficient of thermal expansion, diffusivity, specific heat, and compressive strength. The selected mixes are used at Turkey Point with only minor field variation.

"Retardwell", manufactured by the Union Carbide Company, has been selected as water reducing agent on the basis of the shrinkage and compressive strength tests conducted by Pittsburgh Testing Laboratory.

j. Protective Coatings

Inorganic zinc primer and modified phenolic finish coating have been used on the containment liner plate and the structural steel inside the containment. For repair of defective coatings or application of new coatings on the containment liner plate or the structural steel inside the containment, approved coatings will be used.

For repair of defective coatings or application of new coatings inside containment concrete surfaces, approved coatings will be used.

Physical characteristics of the materials used are adequate to resist exposure due to both normal operating condition and accident (MHA) condition during unit life. Exposures include ionizing radiation, high temperature and pressure (air-steam atmosphere), impingement from jets or sprays and abrasion due to traffic.

Chemical characteristics include resistance to containment atmosphere and to substances used for decontamination and chemical spray following an accident (MHA).

5.1.6.3 Replacement Concrete

Reactor Vessel Closure Head Replacement Construction Opening Concrete

The concrete mix design for the Reactor Vessel Closure Head Replacement construction opening meets or exceeds the engineering properties of the original concrete at the place of application. The testing regime covers all original requirements for containment structure concrete plus testing to verify the shrinkage characteristics of the mix. The development efforts of the replacement concrete insure that the repair mix was compatible with the existing concrete and performs its designed function over the life of the structure.

Ingredients	Applicable Specification
Cement	ASTM C-150-02
Air Entraining Agent	ASTM C-260-02(Master Builders MBAE 90)
Water Reducing Agent	ASTM C-494-02(Master Builders High Range Water Reducer, Glenium 3030)
Aggregate	ASTM C-33-02(Fine and Coarse Aggregate)

Strength

Construction Patch	5000 psi at 28 days

Principle Placement Properties

Slump	8”+/-1.5” after High Range Water Reducing Admixture
Slump Testing	ASTM C-143-02
Air Content	2.5%-5.0% by ASTM C-231-02

Reinforcing Steel

All new rebar was ASTM A-615 Gr.60

Prestressing Tendons and Associated Hardware

Materials	Material Specifications
Tendon wires	ASTM A421, Type BA, Grade 240
Shims	ASTM A656, Type 7, Grade 80 ASTM A656, Type 7, Grade 70 ASTM A737, Grade C, or ASTM A633, Grade E
Tendon Sheathing	ASTM A513, Type 5

Replacement tendons installed during Reactor Vessel Closure Head Replacement were furnished by Precision Surveillance Corporation. The replaced components were a 100% compatible replacement for the original Prescon manufactured components. The new anchor head material was AISI 4140, heat treated, and the wire button heads were slightly larger than the original Prescon button heads. These new components have been previously qualified and used at Calvert Cliffs Nuclear Power Plant and Oconee Nuclear Station.

Anchor Head Materials

ITEM	VALUE
Material	AISI 4140
Yield	89 KSI
Ultimate	118 KSI
Elongation	12%
Reduction in Area	20%
Hardness	Rc 29 to 33

During Reactor Vessel Closure Head Replacement, tendons in the temporary construction opening were relaxed and/or removed. At the completion of the outage, the tendons were re-tensioned by jacking as described in 5.1.6.1(d).

Concrete Mixes:

For the concrete mix used in the Reactor Vessel Closure Head Replacement containment opening repair, pre-placement testing was performed to determine uniaxial creep, modulus of elasticity, Poisson's ratio, coefficient of thermal expansion, diffusivity, and compressive strength. This mix was used at Turkey Point with only minor field variation. Glenium 3030, manufactured by Master Builders, Inc. was selected as the water reducing agent for this mix on the basis of testing conducted by S&ME, Inc., Knoxville, Tennessee.

TENDON SHEATHING FILLER MATERIAL PHYSICAL & CHEMICAL PROPERTIES

Physical Properties:

Physical Information -
Weight per Gallon @ 60˚F. 7.3-7.8 lbs/gal.
Specific Gravity @ 60˚F. 0.88-0.94 ASTM D-287
Melting Point F 125-140 ASTM D-938
Flash Point F (COC) 420 Min. ASTM D-92
Viscosity SUS F
@ 150˚F 100-120
@ 210˚F 150-225

Moisture Saturation Retention 2-3%

Thermal Coefficient of Expansion 0.005

Penetration (Cone)	ASTM D-937
@ 60˚F	250 min.
@ 77˚F	220-260

Thermal Conductivity
Approximately 0.12 Btu/(Hr)(ft2)(F/ft) thickness

Specific Heat
Approximately 0.51 Btu/(1b)(F)

Shrinkage Factor
150˚F -70˚F Approximately 4%

Chemical Properties

Nitrate	(Water Soluble)	<4 PPM
Chloride	(Water Soluble)	<2 PPM
Sulfide	(water Soluble)	<2 PPM
Major Constituents		Fully saturated petroleum hydrocarbon.

Reactor vessel Closure Head Replacement Construction Opening Tendons:

Tendon Sheathing filler material used for replacement tendons contains the following properties:

+-

TENDON SHEATHING FILLER MATERIAL PHYSICAL & CHEMICAL PROPERTIES

Grease shall be Visconorust 2090P-4 Casing Filler as manufactured by Viscosity Oil Co. Grease shall comply with the following specifications:

Physical Properties	Tests	Specifications
Wt. per Gallon ˚F (˚C)		7.3-7.8 lbs.@60(15.6)
Specific Gravity ˚F (˚C)	ASTM D-1298	0.88-0.94@60(15.6)
Congealing Point ˚F (˚C)	ASTM D-938	135 (57)min.
Flash Point ˚F (˚C)	ASTM D-92	420(215) min.
Viscosity SUS ˚F (˚C)	ASTM D-88	130@210 (98.8)
Consistency (Cone Penetration) ˚F (˚C)	ASTM D-937	170-200@77(25)
Thermal Conductivity (approx.)		0.10Btu/H Ft^2˚F/Ft
		(0.149 Kcal/H M^2 ˚C/M)
Specific Heat (approx.)		0.510 Btu/lb/˚F
		(0.157 cal/gm˚C)
Total Base Number (mgKOH/G)	ASTM D-974 (Modified)	35 min.
Water Content (wt.%)	ASTM D-95	0.4 max
Heat of Fusion (approx.)		63.2 Btu/lb
		(35.1 cal/gm)
Thermal Cubical Expansion		0.0004/˚F
Chemical Properties (Note: special preparations of test sample required)		
Water Soluble Chloride Ions	ASTM D-512	2ppm max.
Water Soluble Nitrate Ions	ASTM D-992-78	4ppm max.
Water Soluble Sulfide Ions	APHA 4500 S^{2-} (17th ed.)	2ppm max.
Accelerated Tests		
Corrosion Resistance		
5% Salt Fog@ 0.5 mil(0.0127mm)	ASTM B-117	300 hrs. min.
Oil Separation 30 hrs ˚F(˚C)	FTMS 791 C Method 321.3	2% wt. 100 (37.8) max.
Radiation Resistance Gamma Rays		1x10^{7}Rads
Removal		Petroleum Solvent
ASTM-American Society of Testing Materials, Philadelphia, Pa.		
APHA-Public Health Association- Standard Methods, Washington DC		
FTMS-Federal Test Methods- Standards, Washington DC		

TABLE 5.1.6-2

THIS TABLE HAS BEEN DELETED

5.1.7 CONTAINMENT TESTING

Pre-operational tests are conducted to establish the Initial Leakage Rate and the structural integrity of the containment structure.

5.1.7.1 <u>Pre-Operational Leakage Rate Tests</u>

These tests are made after completion of the containment construction and installation of all penetrations in the containment shell with the containment isolation valves in closed condition.

The objectives of these tests are:

(a) To determine the initial integrated leak rate for comparison with the 0.25% day by free volume at the original transient analysis calculated peak accident pressure and temperature, 49.9 psig and 276°F respectively.

(b) To determine the characteristic leak rate variation with pressure by testing at a minimum of one other pressure so as to allow retesting at pressures less than design pressure.

(c) To institute a performance history summary for both local leak and integrated leak rate tests.

The guidelines established for the tests are:

(a) The methods and type of equipment used during the initial tests can be used for subsequent retests, thus avoiding test result variations due to changes of the methods or the equipment as far as possible.

(b) The leak test equipment is calibrated before the initial test. If the equipment cannot remain in place for subsequent retests, it is made such that it can be recalibrated in place, or is replaceable with a similar calibrated device.

(c) The equipment consists of the necessary flow meters, pumps,pressure sensors, temperature sensors, and moisture sensors needed to operate "absolute" `method of leak rate testing.

(d) The leak rate is measured by integrating the leakage for a period sufficient for data acquisition and processing to establish and verify the leak rate. In addition to the calculated leakage, the integrated leakage is verified by pumping back and measuring the quantity of air that is required to bring the containment back to the original pressure, or by bleeding off a measured quantity of air through flow meters.

Prior to the integrated leak rate test, local leak tests are made on electrical penetrations, across valve seats and along valve stems, and across resilient seals where these items are a part of the containment envelope during the MHA. The containment is pressurized to 14 psig and a local leak survey is performed. The methods used for the local leak tests are the soap bubble, pressure decay (or rise), halogen gas, and sonic detection, as appropriate for the individual item being tested.

An integrated leak rate test is then made at 50% Pp, (Pp=the maximum peak pressure calculated for the design basis accident analysis) and a subsequent test at 100% Pp. The tests utilize the absolute pressure system. All instrumentation is calibrated by accepted methods to assure the required accuracy and precision over the anticipated range of use.

5.1.7.1.1 <u>Reactor Vessel Closure Head Replacement Leakage Testing</u>:

Following the Reactor Vessel Closure Head Replacement containment opening closure, a Type A Integrated Leakage Rate Test, ILRT, was performed in accordance with requirements of 10 CFR 50 Appendix J, Technical Specifications and station procedure. Structural inspections were performed in accordance with ASME Boiler and Pressure Vessel Code, Section XI, Subsection IWE & IWL, 1992 Edition with 1992 Addenda.

5.1.7.2 Structural Integrity Tests

The objectives of these tests are to:

(a) Provide direct verification that the structural integrity, as a whole, is equal to or better than that necessary to sustain the forces imposed by two different and large loading conditions.

(b) Provide direct verification that the in-place tendons (the major strength elements) have a strength of at least 80% of gross ultimate tensile strength and that the concrete has the strength needed to sustain a strain range from high initial average concrete compression when unpressurized, to low average concrete compression when pressurized.

(c) Acquire detailed strain data which is compared with the analytical predictions.

To achieve objective (a), the response of the structure is measured during and immediately after post tensioning to determine if there is an indication of unanticipated and continued deformation under load. While the pressurization of the containment is done in convenient intervals, the response of the structure is measured at 10, 20, 30, 40, 50, 55, and 63.25 (115% of 55) psig. Measurements during depressurization are made at approximately 15 psi intervals. De facto indication that the structure is capable of withstanding the internal pressure results from these tests.

To achieve objective (b), each individual tendon is tensioned in place to 80% of the guaranteed ultimate tensile strength and then anchored at a lower load which is still in excess of that predicted to exist at test pressure.

To achieve objective (c), modern data acquisition and handling methods are used, as described in Appendix 5C, to provide for rapid test results and to enhance objectivity.

From the present knowledge of the analytical uncertainties, it is expected than an agreement will be found between the test results and the analytical predictions within the following range:

Cylinder at Equator	15%
Dome	15%
Bottom Slab	25%
Bottom Slab to Wall Junction	25%
Dome to Wall Junction	20%
Around Openings	30%
Localized Stress Concentration	100%

If the measured strains fall noticeably beyond the above-mentioned ranges of error, a review and investigation will be made to determine the cause of such discrepancies.

Reactor Vessel Closure Head Replacement Structural Integrity Test:

At the completion of the containment opening repair process the containment structure was subjected to a Type A Integrated Leakage Rate Test, ILRT. A Structural Integrity Test, SIT, was not performed based on the applicability of NRC Inspection Manual, Inspection Procedure 71007, Appendix B. This Appendix stipulates that only one of the two tests (ILRT or SIT) must be performed. Containment measurements were made before, during, and following the ILRT to demonstrate structural integrity.

5.1.7.3 Test Procedures and Instructions

The test procedure is such that the structural leak tight integrity tests can be carried out in the same time period. To record and transmit the test requirements, a step-by-step test procedure is contemplated by data acquisition, verification, reduction and collation instructions as well as data interpretation standards. Detailed instructions are prepared for use of other systems used in the combined test, such as for the pressurization system.

5.1.7.4 Tendon Surveillance

Provisions are made for an in-service tendon surveillance program, throughout the life of the plant that will maintain confidence in the integrity of the containment structure. (See Subsection 16.2.1.4 for program description relating to license renewal. See Subsections 17.2.1.3 and 17.2.2.31 for the aging management programs relating to subsequent license renewal.) C31

The following quantity of tendons have been provided over and above the structural requirements:

Horizontal	-	Three 120 degree tendons comprising one complete hoop system.
Vertical	-	Three tendons spaced approximately 120 degrees apart.
Dome	-	Three tendons spaced approximately 120 degrees apart.

Prior to the twentieth year tendon surveillance, inspections and lift-off readings were performed at each surveillance period on the same tendon samples selected originally for inspection. During the twentieth year and twenty fifth year surveillances, inspections and lift-off readings were performed on five horizontal, four vertical, and three dome tendons. The tendons chosen for surveillance were a random but representative sample.

Beginning with the thirtieth year tendon surveillance, inspections are performed on five horizontal, four vertical, and four dome tendons in accordance with the requirements of ASME Section XI, Subsection IWL, and Code of Federal Regulations 10 CFR 50.55a. The tendons chosen for surveillance are a random sample selected in accordance with Subsection IWL.

The surveillance program for structural integrity and corrosion protection includes the following operations to be performed alternately at Unit 3 and at Unit 4 as specified in Subsection IWL:

(a) Lift-off readings will be taken for all of the thirteen tendons.

(b) One tendon of each directional group will be relaxed and one wire from each relaxed tendon will be removed as samples for inspection. Since these tendons are re-tensioned to their original lift-off forces these samples need not be replaced.

(c) After the inspection, the tendons will be retensioned to the stress level measured at the lift-off reading and then checked by a final lift-off reading.

(d) Should the inspection of one of the wires reveal any significant corrosion (pitting, or loss of area), further inspection of the other two sets will be made to determine the extent of the corrosion and its significance to the load-carrying capacity of the structure. Samples of corroded wire will be tested to failure to evaluate the effects of any corrosion on the tensile strength of the wire.

The inspection of the four vertical tendons in the wall is sufficient to indicate any tendon corrosion that could possibly appear longitudinally along the full height of the structure. Therefore, the thirteen tendons arranged as described will provide adequate corrosion surveillance.

The anchorage details permit some degree of accessibility for inspection of all tendons in the containment structure. Corrective action will be taken if and when so indicated by the surveillance program, and an adequate containment structure will be maintained throughout the life of the plant.

The following steps are taken to protect the tendons and the reinforcing steel in the containment structure from corrosion due to stray current and moisture environment.

A tendon protection sheathing filler compound encloses the whole length of every tendon. This compound will not deteriorate during the life of the unit. As its chemical composition is about 98% petroleum jelly, it will possess the normal stability of the linear hydrocarbons subjected to normal ambient temperature levels. The electrical resistivity of the compound is relatively high. This prevents the possibility of galvanic corrosion that would be detrimental to the tendons. Anodic corrosion centers that could develop on the surface of tendons surrounded by a good electrolyte material will not form in the presence of the protective sheathing filler.

All metallic components such as the tendon trumplate, reinforcing bars and liner plate are interconnected to form an electrically continuous cathodic structure, thereby avoiding inherent difficulties associated with isolation and interference of these members. This interconnection of the steel work with the liner plate ensures that cathodic protection currents will not be allowed to flow through any isolated member to cause electrolytic corrosion.

The cathodic protection system is designed to protect the interconnected liner, reinforcing bars, and tendon trumplates. The original system used four deep anode ground beds for each containment and various types of reference electrodes which are not replaceable. It was determined by testing that the original system was reaching the end of its useful life as anodes and reference electrodes were expended or had become non functional. The 1994 replacement containment cathodic protection system uses a deep ground anode bed drilled beneath the containment. This anode system is designed for a 20 year life and has provisions for future replacement if necessary. Reference electrode test locations have been added to the tendon inspection gallery to obtain potential gradient data. Refer to Section 8.2 for additional discussion.

5.1.8 CONTAINMENT INTERIOR STRUCTURES

5.1.8.1 General

The containment interior consists mainly of the following structures:

(a) The reactor primary shield wall. It is a 7'-0" thick cylindrical wall that encloses the reactor vessel and provides biological shielding and structural support.

(b) The lower secondary compartment. It encloses the reactor coolant loops and consists of the secondary shield walls that support the intermediate floor at Elevation 30'-6".

(c) The upper secondary compartments. Three of these compartments enclose one reactor coolant loop each and another encloses the pressurizer. The compartment walls provide secondary biological shielding and structural support for the operating floor at Elevation 58'-0".

(d) The refueling cavity.

Typical cross-sections through the containment interior are shown in Figure 5.1-1.

5.1.8.2 Design Basis

(a) The stresses in any portion of the interior structures, under the action of dead loads, live loads, thermal loads and design earthquake loads will be below the allowable stresses as given by the ACI Building Code (ACI 318-1963) and the AISC Manual of Steel Construction (6th edition).

(b) The stresses in any portion of the interior structures, under the action of dead loads, live loads, thermal loads and maximum hypothetical earthquake loads will be below the elastic limit.

(c) The stresses in any portion of the interior structures, under the action of dead loads, live loads, thermal loads and maximum hypothetical accident loads will be below the elastic limit. Under such loads the containment boundaries, i.e. the liner plate and the pipe penetrations, will be unimpaired and the required engineered safeguard systems will be protected.

The elastic limit for steel structures is the minimum yield point as given by the ASTM standards; for reinforced concrete structures, it is the stress condition at the yield point of the reinforcing steel.

The maximum hypothetical accident loads consist of the pipe rupture thrust reactions, jet impingement forces, and/or the differential pressures across the compartment walls as applicable.

The reactor primary shield wall is designed to withstand proper combinations of dead loads, live loads, thermal loads, earthquake loads and accident loads. Accident loads are pipe rupture reaction forces and compartment differential pressures.

The secondary shield walls are designed to resist general failure under combined effect of jet impingement forces and compartment differential pressures.

(d) Where high operational concrete temperatures exist, as at the equipment supports, high strength concrete is specified to provide for the loss of strength and the locked-in thermal stresses.

5.1.8.3 Missile Protection

All high pressure equipment that could generate missiles during or immediately after an MHA is surrounded by barriers to prevent such missiles from damaging the containment liner, the pipe penetrations and the required engineered safeguard systems. Principal barriers against missiles are the reinforced concrete primary and secondary shield walls and the operating floor.

Additionally, shielding is located above the reactor vessel head to block any missile that could be generated by the control rod drive mechanisms. The original concrete CRDM missile shields were removed due to the installation of the integrated head assembly (IHA). The IHA utilizes an integrated steel missile shield that provides the necessary blockage of any missile potentially generated by the control rod drive mechanisms.

Penetration was checked by the Petry formulas:

$$D = KAp \log_{10} \left(1 + \frac{V^2}{215{,}000}\right)$$

D=depth of penetration into an infinite thickness (ft.)

$$Ap = \text{sectional pressure} \; \frac{(\text{missile weight})}{(\text{maximum cross-sectional area})} \quad (\text{psf})$$

V=terminal striking velocity (ft/sec)

K=experimentally obtained material coefficient for penetration

Penetration into a reinforced concrete slab of finite thickness:

$$D' = D\{1 + e^{[-(T/D-2)]}\}$$

D' =actual depth of penetration (ft.)

T =thickness of slab (ft.)

For a description of the hypothetical missiles, their sizes, weights and velocities, see Appendix 5E.

To ensure the IHA missile shield is capable of withstanding a missile impact and adequately performing its design function, a penetration evaluation as well as a finite element strain evaluation were performed. The penetration evaluation uses the USNRC Standard Review Plan for guidance in determining the minimum required thickness of the IHA missile shield to ensure it can absorb the impact energy without perforation (Reference 1). A non-linear transient finite-element analysis evaluated the missile shield for the overall effects expected from the impact of a CRDM missile.

REFERENCES

1. AREVA Document 38-5041408 (ADVENT Document 03009TR-05), Revision 01, “Integrated Head Assembly Missile Shield Design”.

5.1.9 MAJOR COMPONENT SUPPORT SYSTEM

The support system for the reactor vessel, steam generators, and the reactor coolant pumps is designed for all the operating, seismic, and accidental loads. In case of a pipe rupture in the reactor coolant system, excessive movement of the reactor vessel is prevented, and the two non-ruptured loops with their associated safeguard systems are protected from damage. A break in a steam line is not permitted to rupture reactor system piping, and vice versa.

5.1.9.1 Design Basis

The following load combinations are used for design of the Major Component Support System:

$$S = (D + L + T)$$
$$1.33S = (D + L + T + E)$$
$$1.50S = (D + L + T + E')$$
$$Y = (D + L + T + R)$$

S = Allowable Stresses
D = Dead Load Stresses
L = Live Load Stresses
T = Temperature Stresses where they apply
E = Design Earthquake Stresses
E'= Maximum Design Earthquake Stresses
R = Stresses due to pipe rupture reactions
Y = Yield stresses of the materials used for the supports as allowed by the ASTM and the ASME Codes.

Reduction in yield strength is also considered due to the high temperature that the supports will sustain.
F_y = Yield strength of the Material
Allowable Compressive Stress, $S_c = 0.6\ F_y$
Allowable Bearing Stress, $S_b = 0.9\ F_y$
Allowable Shearing Stress, $S_v = 0.4\ F_y$
The materials used are listed in paragraph 5.1.6.2 (f.)

5.1.9.2 Design Loads

The following elements/loads affect the design of the support system:

1. Thermal movements

   To minimize thermal stresses during unit heat-up and cool-down, the three loops of the system are allowed to expand and contract as freely as possible in the direction of movement, while restraining the vessels from excessive deflections in the other directions under the various loading conditions.

   The center line of the reactor vessel is considered to be fixed in space. During unit heat-up the components of the three loops expand. The reactor vessel expands radially and the main pipes grow in the direction of their axes away from the reactor vessel. The steam generators and the pumps expand radially about their own vertical axes, and grow vertically as well. During the hot functional tests the system, and component, expansion and contraction movement will be measured to ensure the predicted values are met.

2. Normal Operating Loads

   These are the dead loads of the vessels and pipes while operating, and the thermal loads that are induced in the supports due to the partially restrained thermal growth and the geometrical configuration of the system.

3. Seismic Loads

   The loads that are induced in the supports by the acceleration of the vessels due to the design earthquake (E) or the maximum earthquake (E'). A description of the analytical method is in Appendix 5A.

4. Maximum Hypothetical Accident (MHA) Loads

   These are the thrust loadings associated with the hypothetical reactor loop pipe rupture. Both a circumferential and a longitudinal rupture are postulated to occur nonconcurrently anywhere in the system piping. The magnitude and nature of the thrust loads are defined by the NSSS vendor.

Several break locations are considered to determine the maximum possible load that each support can experience. The largest moment that a pipe can transmit to a vessel is taken as the plastic moment capacity of the pipe. For longitudinal breaks the pipe is assumed to remain in one piece, thus distributing the shear loads to both ends.

Reference 3 of FSAR Section 4.2 delineates that the leakage detection systems at Turkey Point Units 3 and 4 satisfy the requirements of Generic Letter 84-04, "Safety Evaluation of Westinghouse Topical Reports Dealing With Elimination of Postulated Pipe Breaks in PWR Primary Main Loops." Therefore, the dynamic loads associated with double ended rupture of main coolant loop piping need not be considered in the design of the Reactor Support Structures.

5.1.9.3 Reactor Vessel Supports

The vessel is supported and restrained on its six nozzles. Each nozzle bears on three rollers set on a girder which is carried by three beams cantilevered from the primary shield wall. A shear lug on either side of the nozzle shoe provides tangential restraint. (See Fig. 5.1.20).

Roller supports permit nearly free thermal growth. Cantilevered steel beams and lateral sheer lugs provide vertical and lateral restraints to resist operating and seismic loads. The non-ruptured loops piping are protected by absorbing the rupture forces in the support system as moments, shears and axial loads. Together, they prevent excessive movements of the vessel. No vertical hold down clamps are provided to resist upward forces since the dead weight of the vessel combined with the stiffness of the unruptured primary loop pipes provide enough resistance against uplift.

5.1.9.4 Steam Generator Supports

Each steam generator has four support lugs near its bottom. Each lug is bolted to the horizontal web of a T-shaped weldment that is vertically supported by twin columns, and horizontally restrained by another plate anchored in the concrete slab surrounding the vessel. The four T-shaped weldments and the associated bearing plates constitute the bottom vertical and lateral support. At the upper lateral restraint, a ring girder transfers lateral loads in all directions to the operating floor slab through embedded steel plates.

Thermal growth of the primary coolant pipes translates the steam generator along the axis of the hot leg. Lubrite plates between the support lugs and the T-shaped weldments, and slotted holes in the horizontal web of the weldments permit this translation with minimal frictional resistance. Radial growth of the vessel is also accommodated in the slotted holes. Vertical growth of the support columns is allowed through slotted holes in the flange of the weldments. At the upper support, the various expected movements of the vessel (translational, vertical, and radial) are combined to determine the inclination of the bearing surfaces in the vertical and horizontal planes.

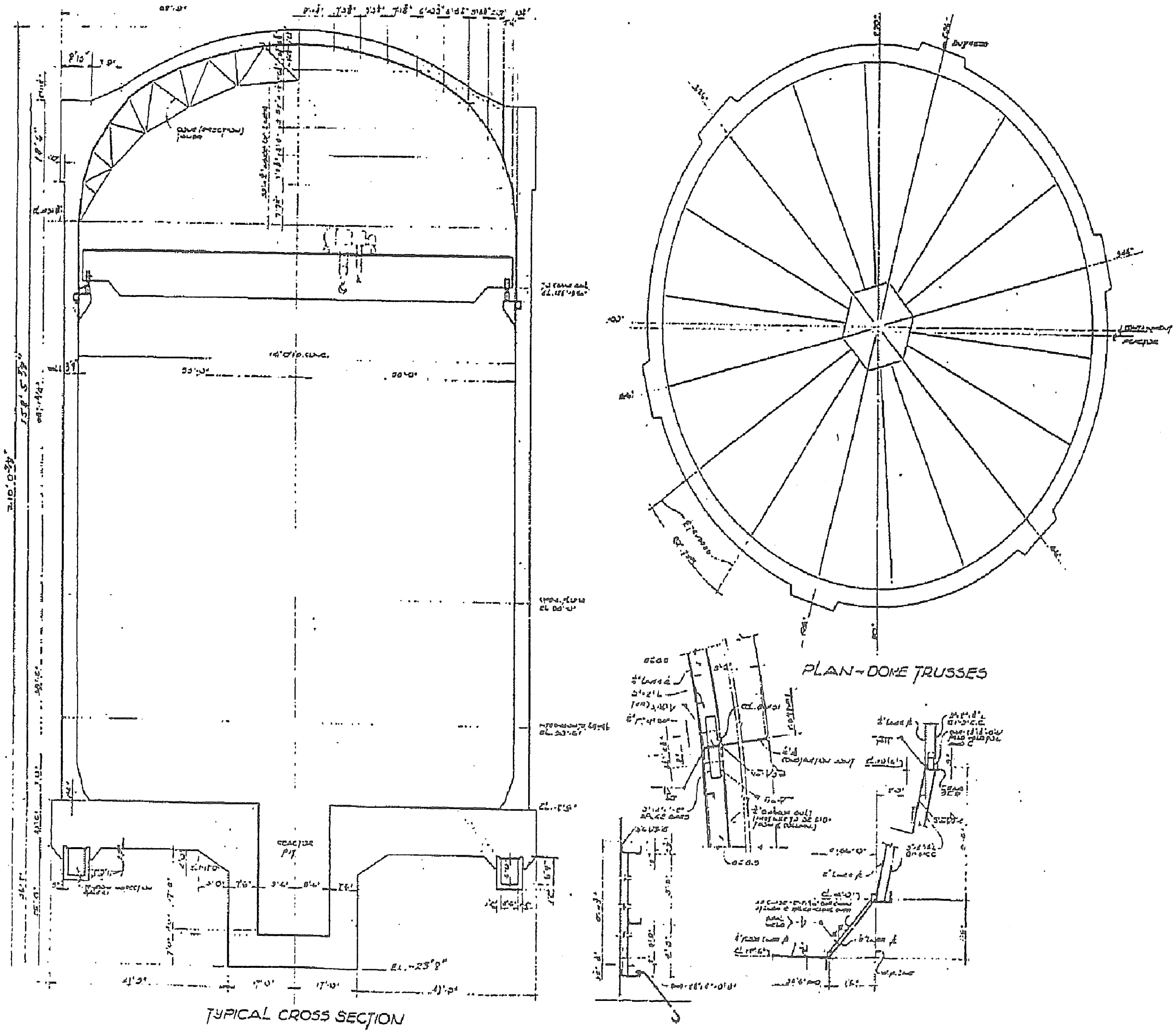
Thrust loads that are associated with primary coolant pipe rupture could be direct forces, overturning moments and torsional moments acting separately or combined. When direct forces are vertical, they are resisted by the columns and bolts. When horizontal, they are resisted by the lower lateral support due to its proximity to the pipe. Overturning moments are resisted by the upper and lower support that provide the required resisting couple. Torsional moments that tend to twist the vessel around its vertical axis are resisted by the four lower lateral supports. Loads associated with a steam pipe rupture are also similarly resisted by the combined action of vertical and lateral supports.

5.1.9.5 Reactor Coolant Pump Supports

The pump has three support lugs, each supported on twin columns with T-shaped plate weldment and laterally restrained into the surrounding concrete, similar to the lower lateral supports for the Steam Generators.

The axial growth of the coolant pipe, radial expansion of pump casing, and the upward growth of the support columns is admitted by the combination of slotted holes and lubrite plates as in the steam generators.

Operating loads, pipe rupture loads, and seismic loads are all resisted by the lateral support and vertical columns.



Alternate stiffeners detail is provided in drawing 5610-C-172.

C22

Revised 08/12/2005

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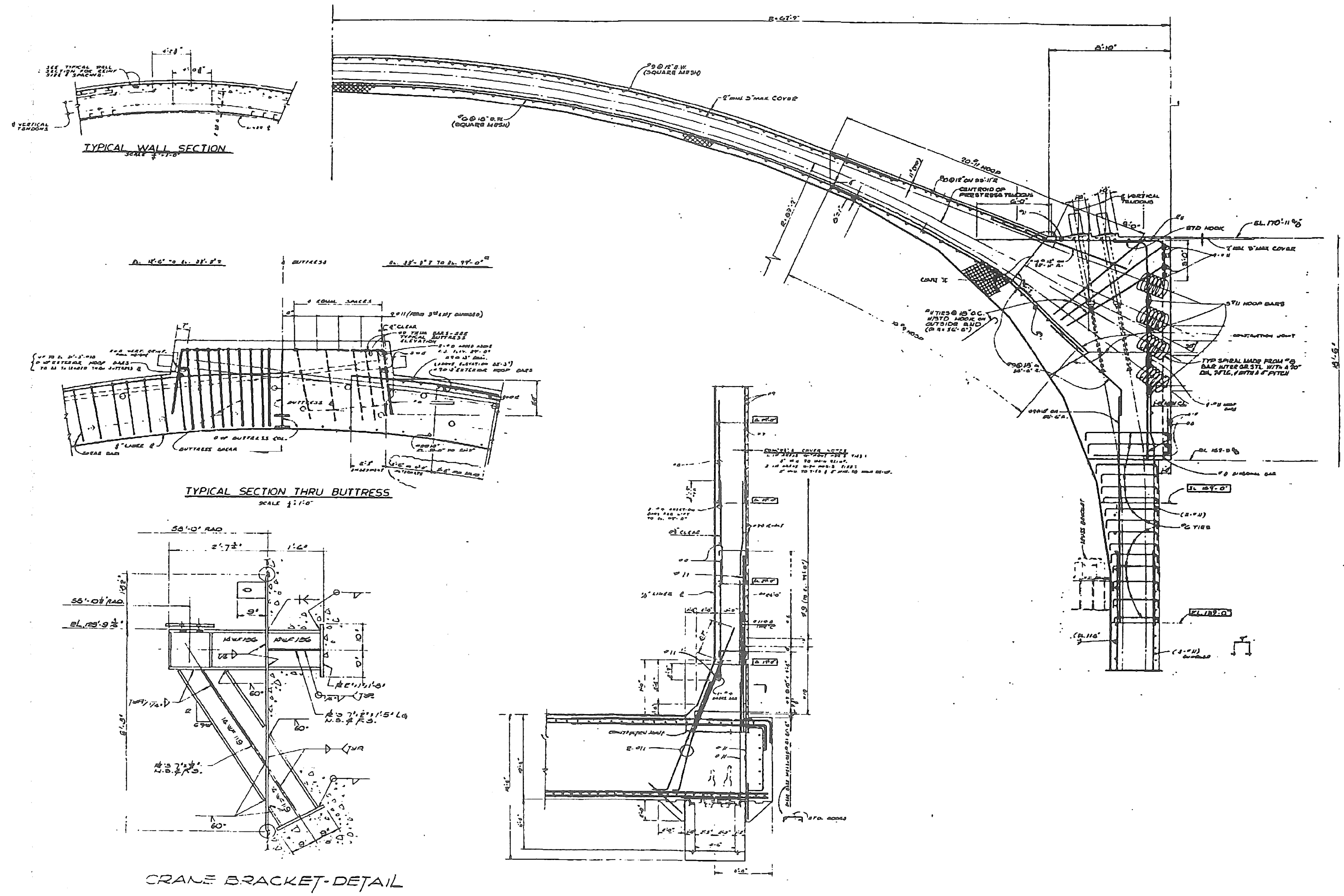
TURKEY POINT PLANT UNITS 3 & 4

CONTAINMENT STRUCTURE
TYPICAL CROSS SECTION
FIGURE 5.1-1
SHEET 1



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TURKEY POINT PLANT

CONTAINMENT STRUCTURE TYPICAL SECTIONS
FIGURE 5.1-1
Sheet 2

FINAL SAFETY ANALYSIS REPORT

FIGURE 5.1-1

REFER TO ENGINEERING DRAWING

5610-C-179

Revised 11/06/04)

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CONTAINMENT STRUCTURE
TYPICAL DETAILS
FIGURE 5.1-1
SHEET 3

FINAL SAFETY ANALYSIS REPORT

FIGURE 5.1-2

REFER TO ENGINEERING DRAWING

5610-C-204

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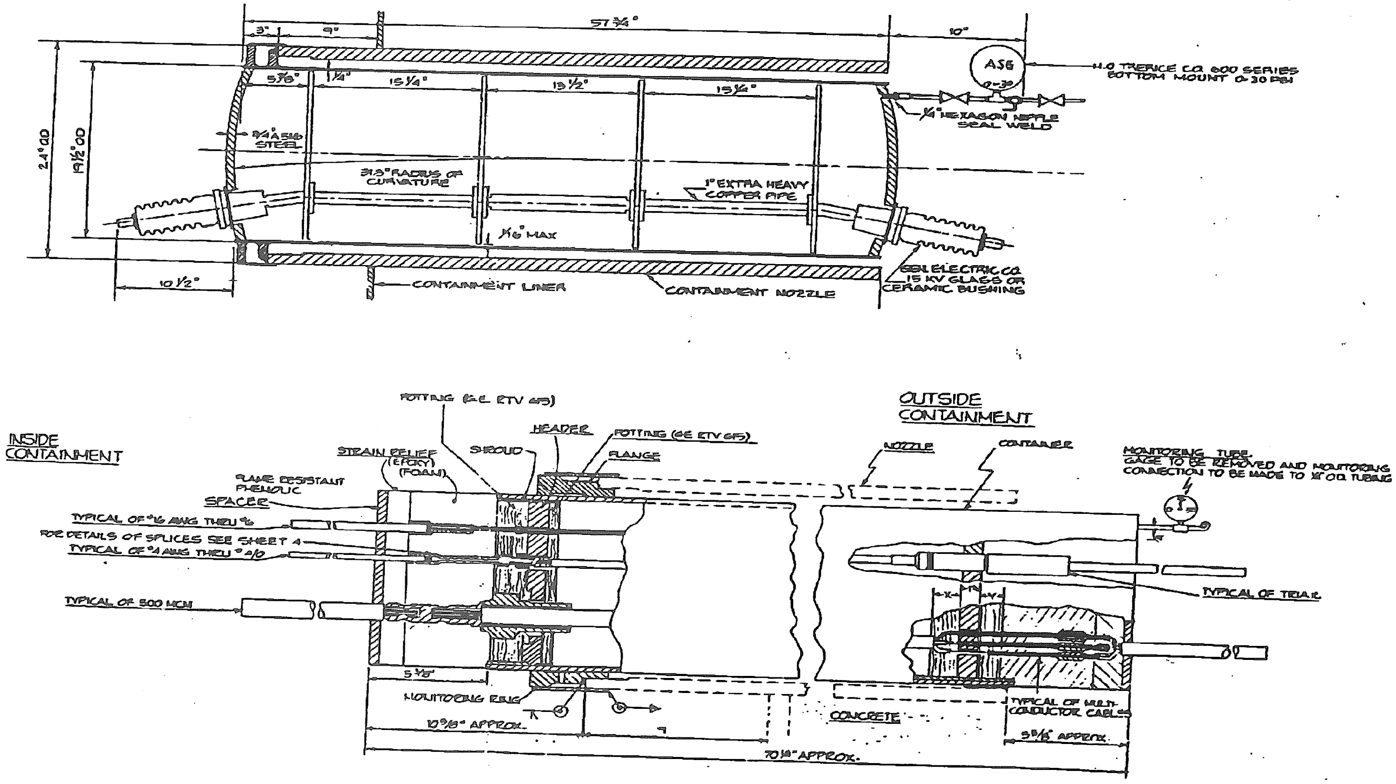
CONTAINMENT STRUCTURE
TYPICAL PIPING PENETRATIONS

FIGURE 5.1-2



REV. 9 (7/91)

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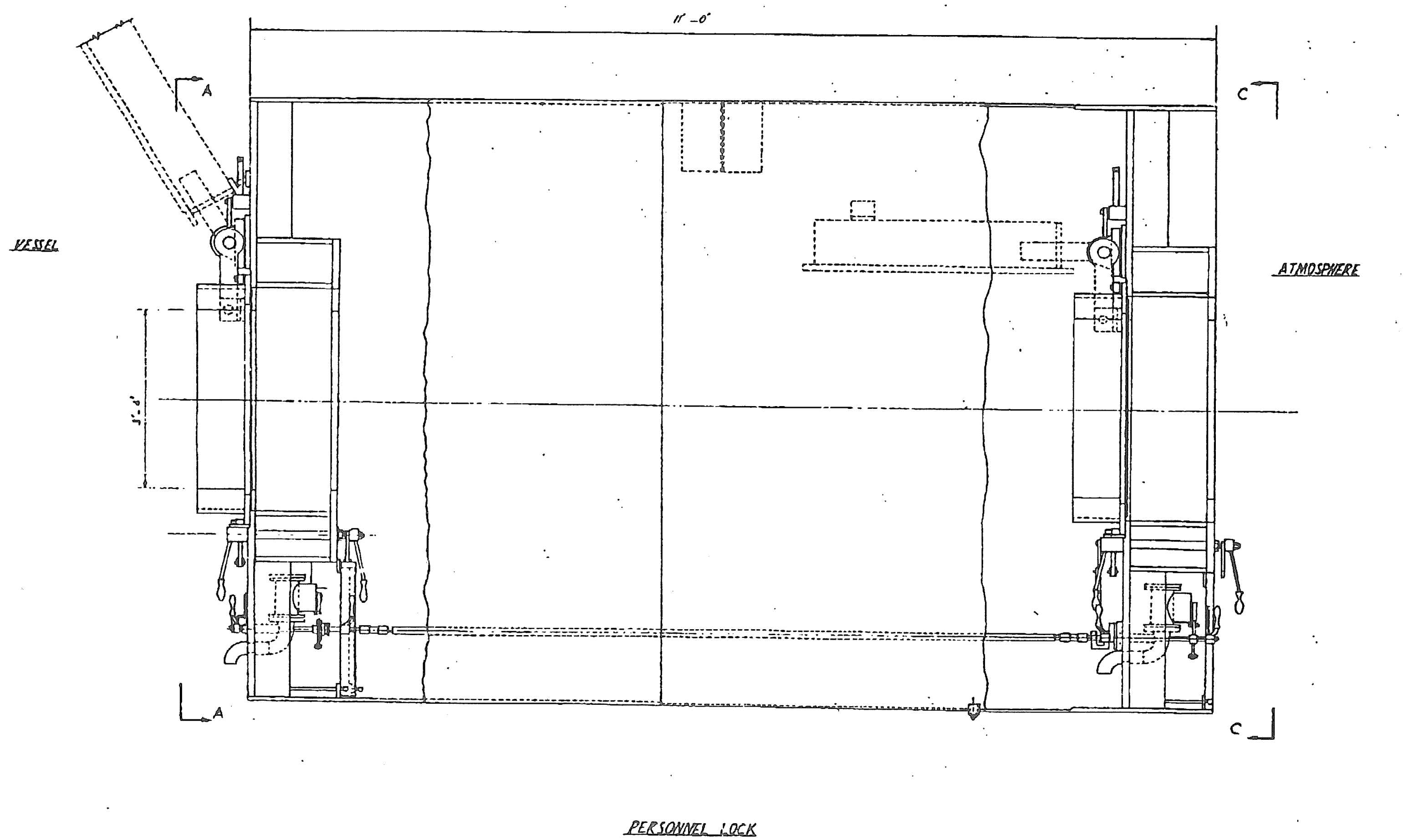
CONTAINMENT STRUCTURES
TYPICAL ELECTRICAL PENTRATIONS
FIGURE 5.1-3



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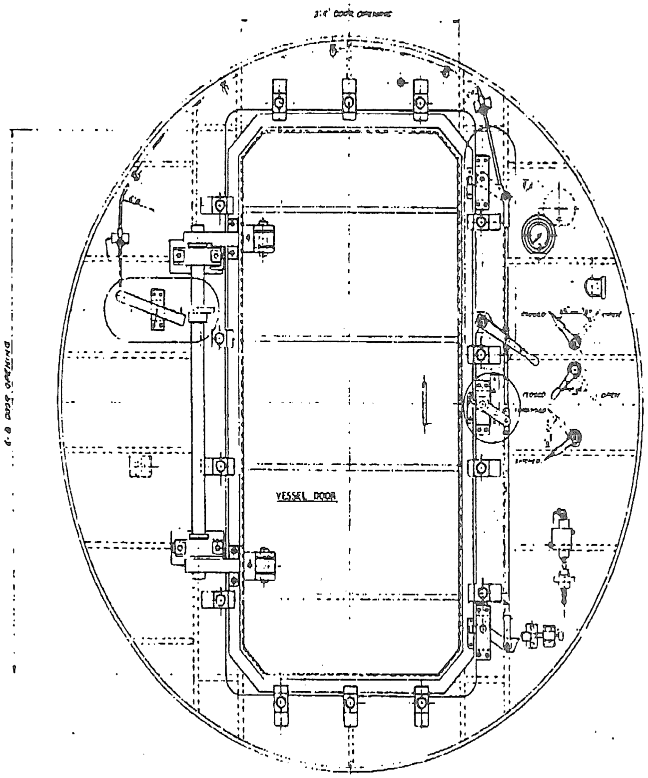
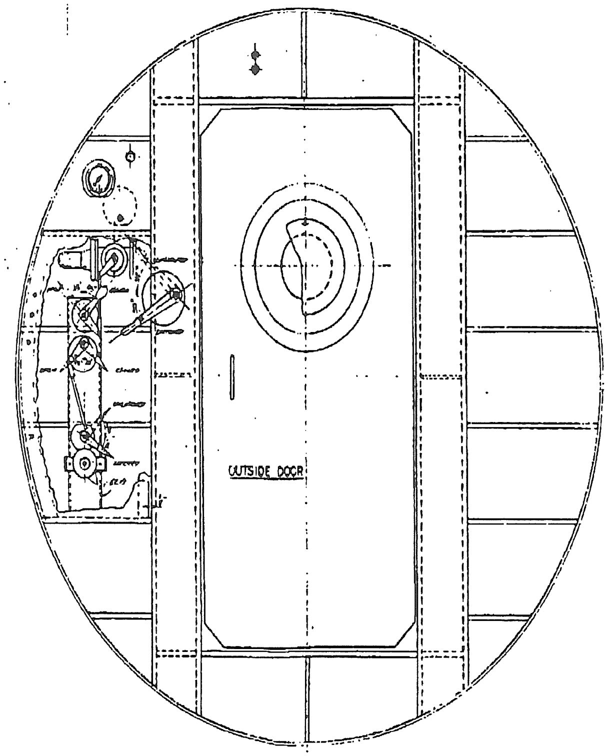
CONTAINMENT STRUCTURE PERSONNEL LOCK - PLAN
FIGURE 5.1-4
Sheet 1



SECTION "A - A"



SECTION "C - C"

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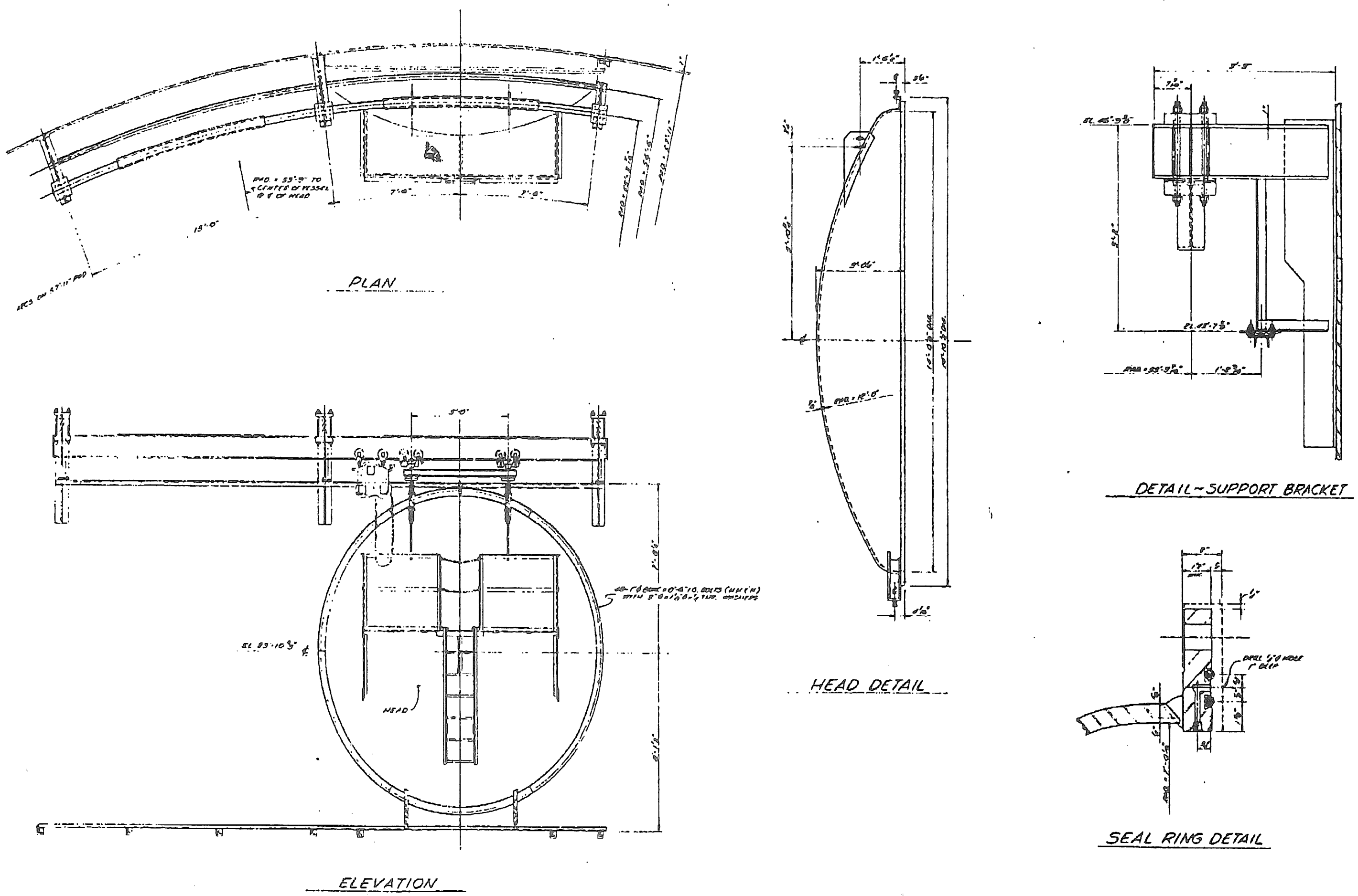
TURKEY POINT PLANT

CONTAINMENT STRUCTURE PERSONNEL LOCK - SECTIONS
FIGURE 5.1-4
Sheet 2



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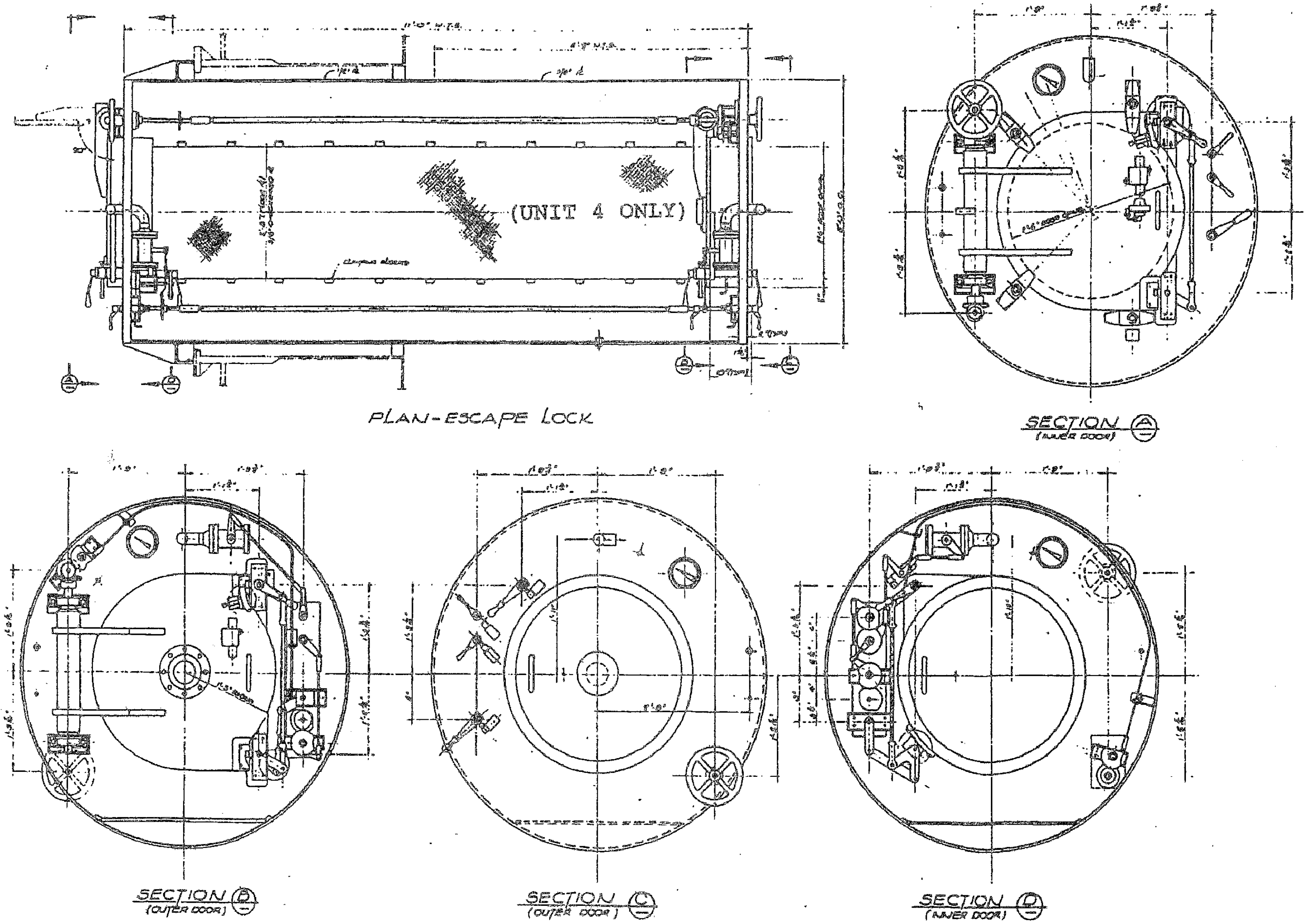
CONTAINMENT STRUCTURE
EQUIPMENT ACCESS HATCH
FIGURE 5.1-5



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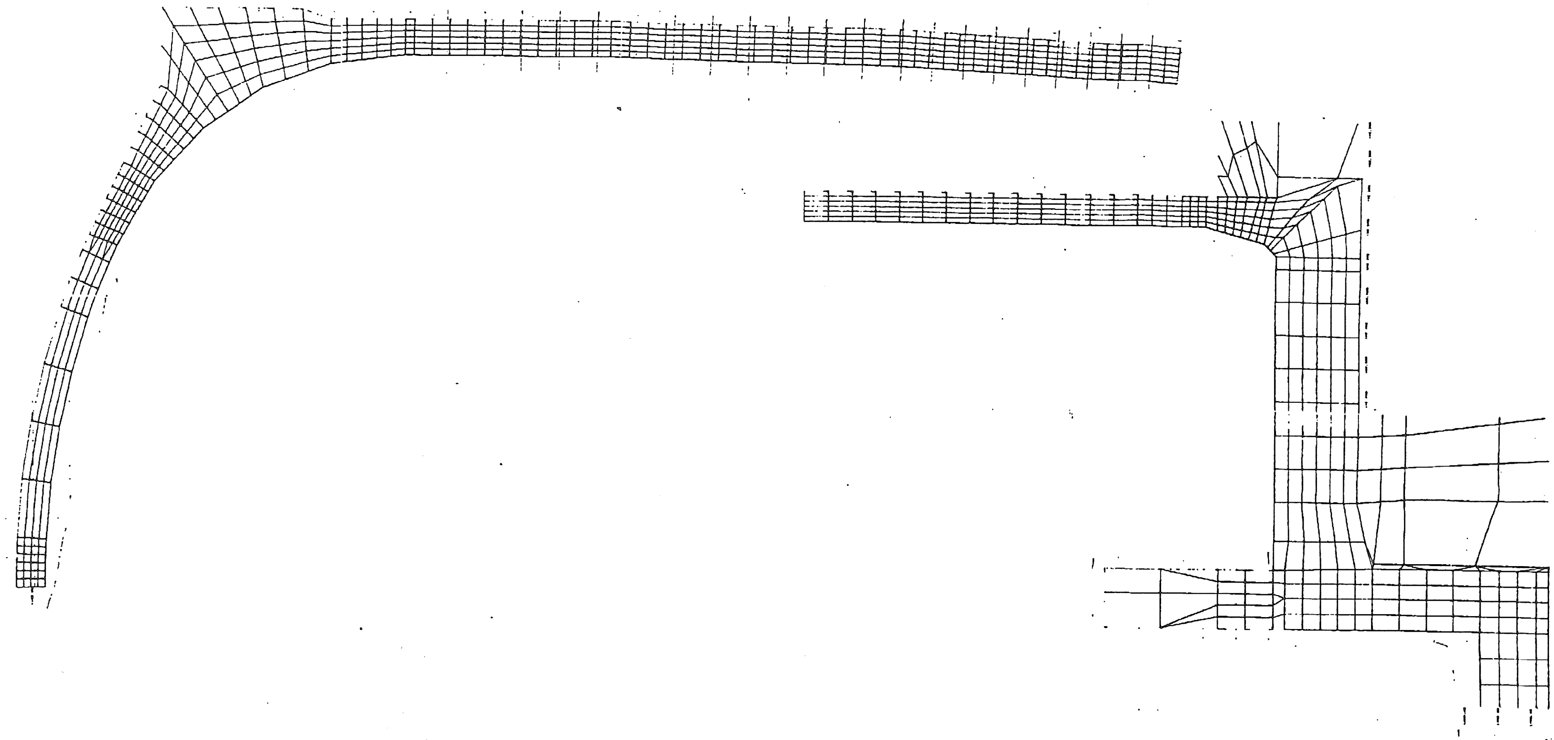
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
ESCAPE LOCK
FIGURE 5.1-6



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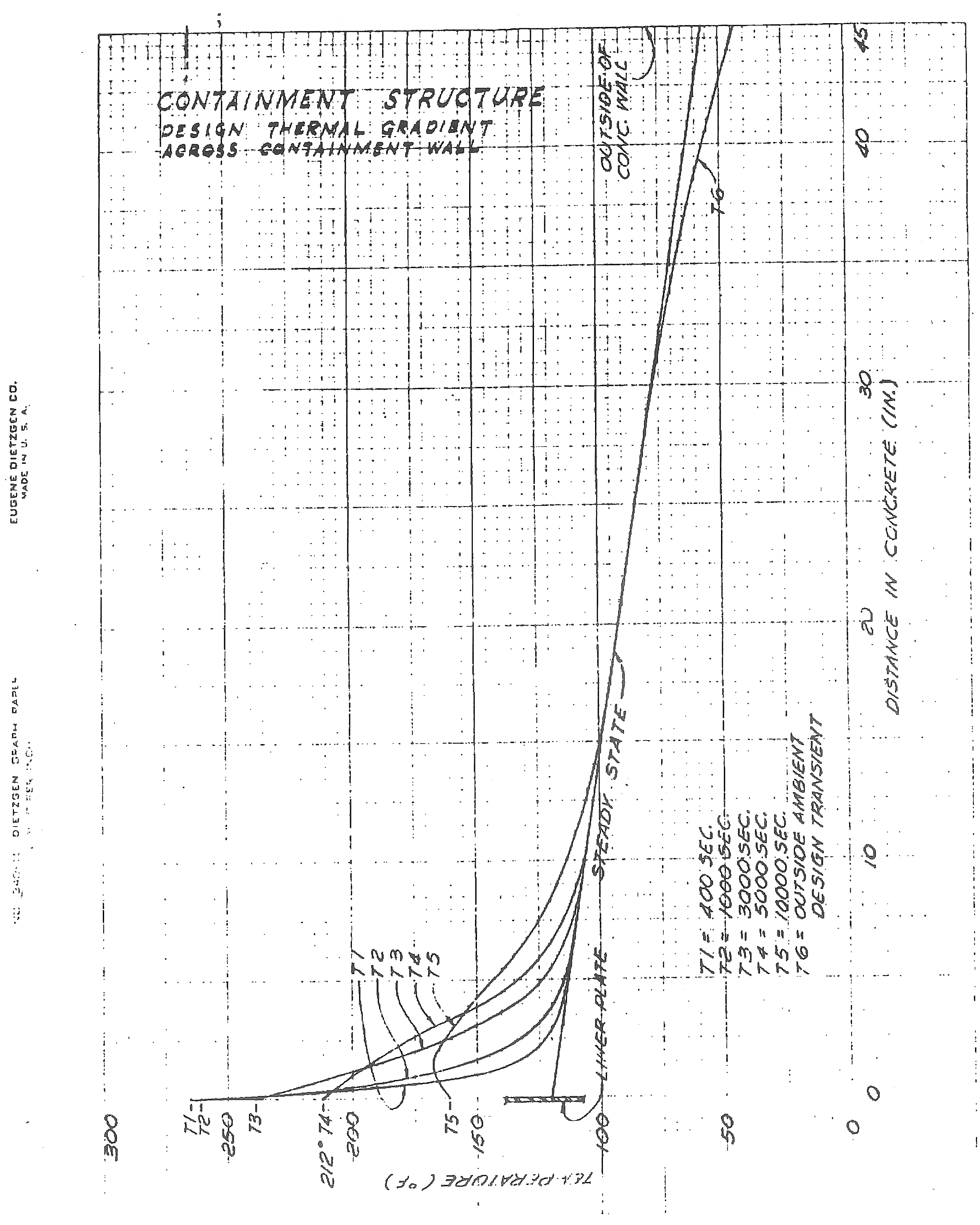
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
FINITE ELEMENT MESH
FIGURE 5.1-7



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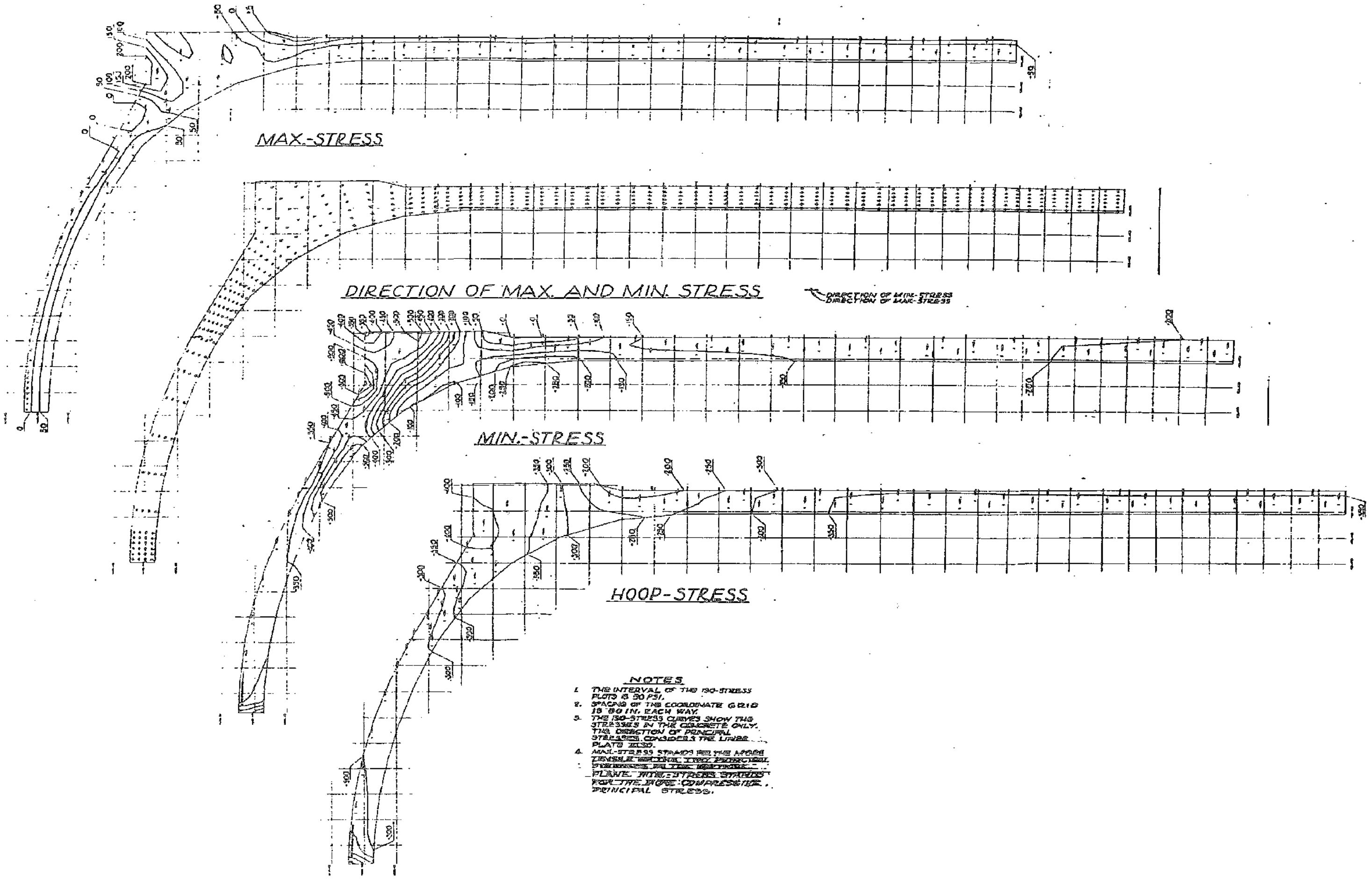
TURKEY POINT PLANT UNITS 3 & 4

CONTAINMENT STRUCTURE
DESIGN THERMAL GRADIENT ACROSS
CONTAINMENT WALL

FIGURE 5.1-8



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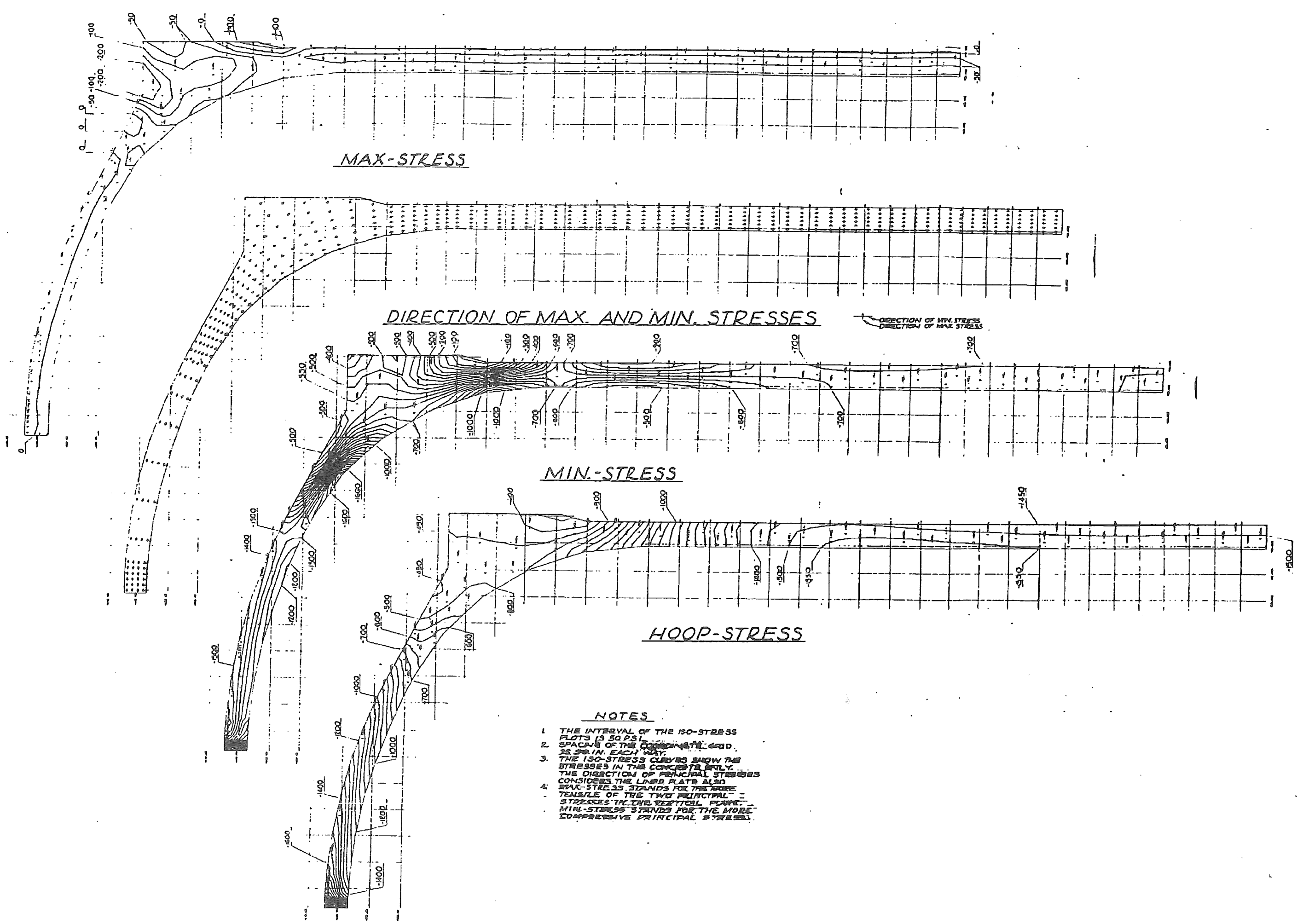
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
ISO - STRESS PLOT WALL
AND DOME (D+F+1.15P)
FIGURE 5.1-9



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
ISO - STRESS PLOT WALL
AND DOME (D+F) INITIAL
FIGURE 5.1-9
Sheet 2

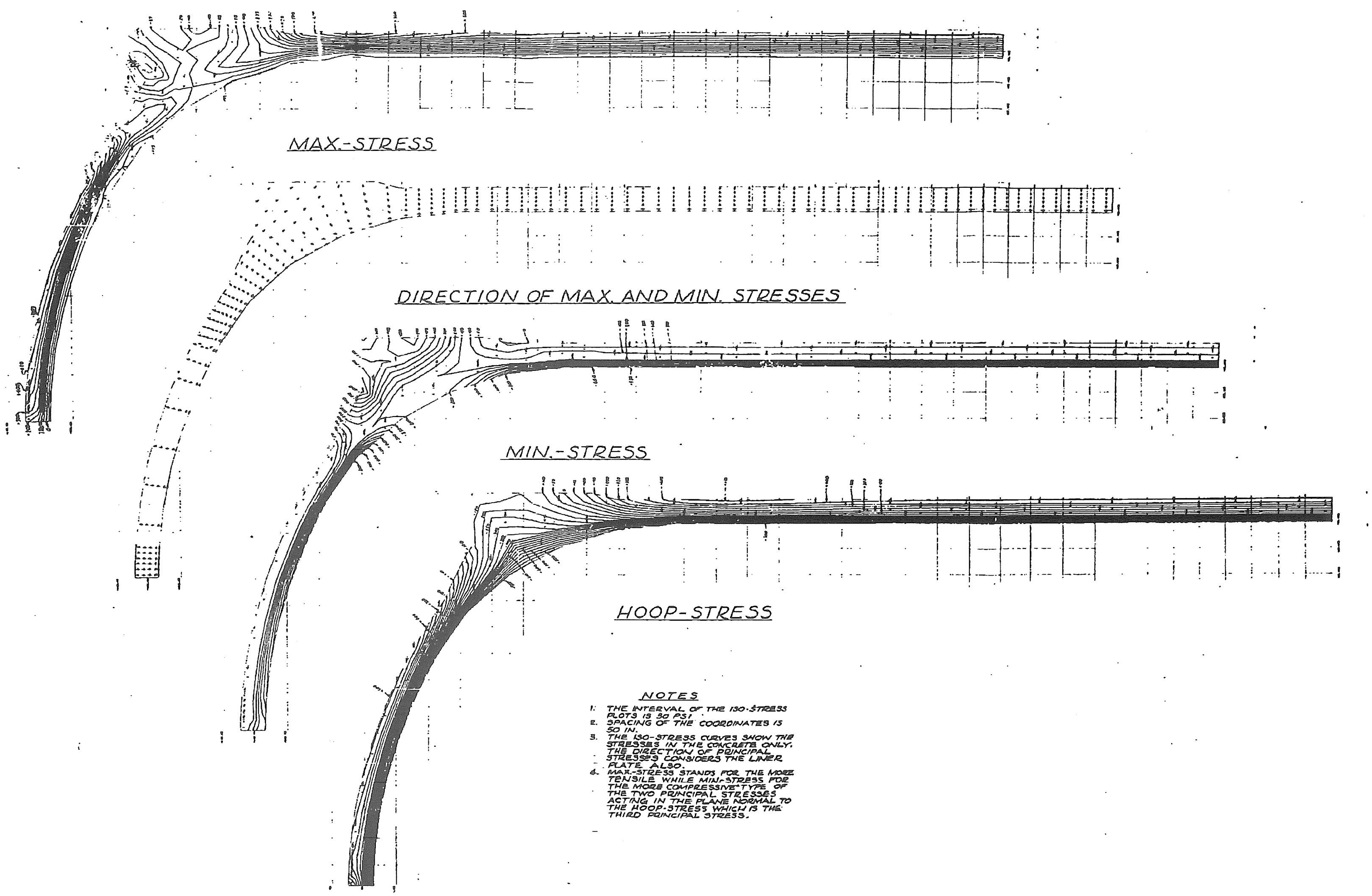
FIGURE 5.1-9
<u>SHEETS 3 and 4</u>

CONTAINMENT STRUCTURE
ISO-STRESS PLOT WALL
AND DOME

(Insert Page In Front of Figure 5.1-9, Sht. 3)

EXPLANATORY NOTE:

The containment structure re-analysis (completed in 1994) encompasses the cylindrical shell and dome areas. Refer to Appendix 5H for the updated stress results/information relative to the cylindrical shell and dome areas. The information on Figure 5.1-9, Sheets 3 and 4, is considered historical for the original analysis.

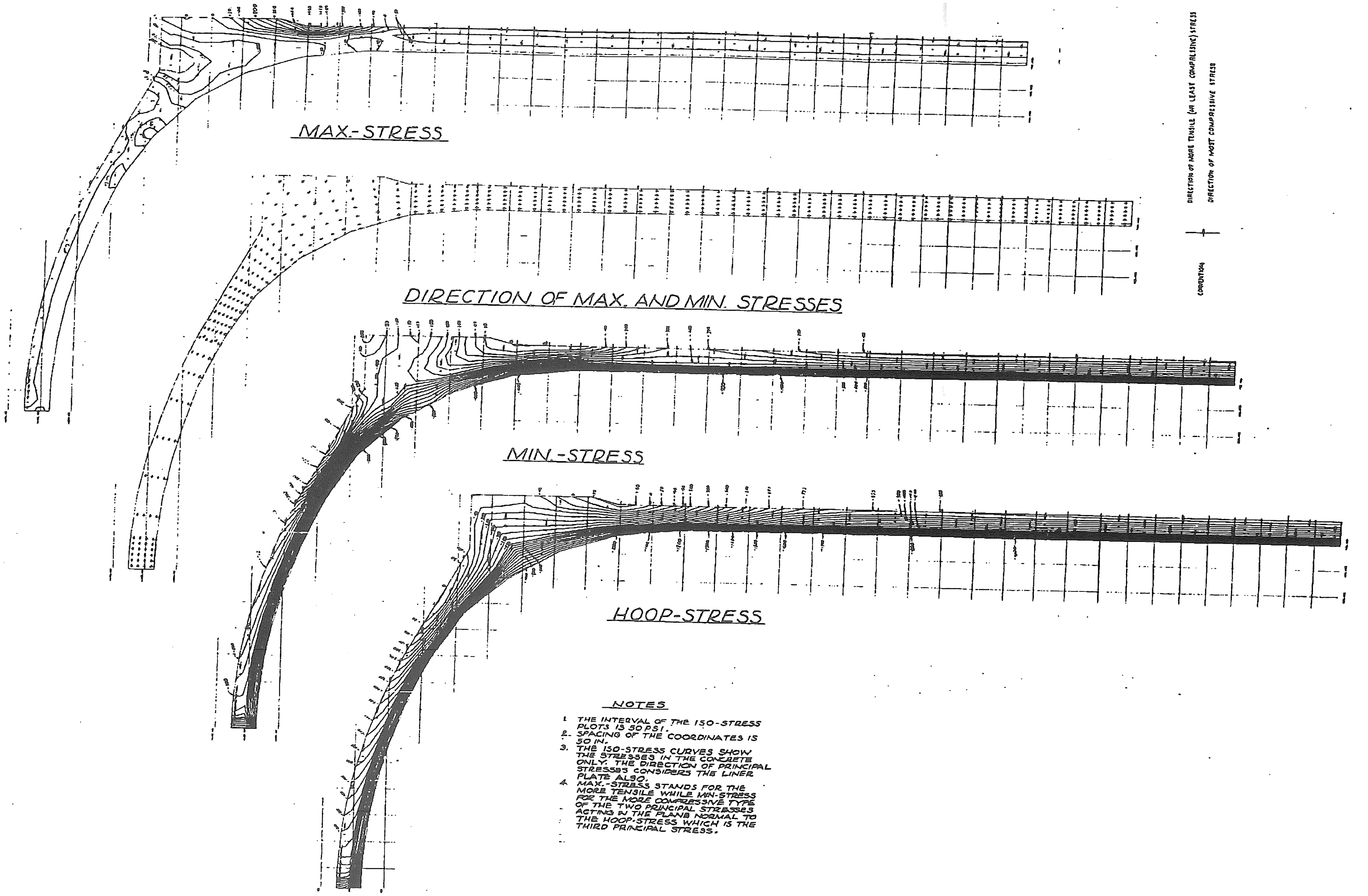


NOTES

1. THE INTERVAL OF THE ISO-STRESS PLOTS IS 50 PSI
2. SPACING OF THE COORDINATES IS 50 IN.
3. THE ISO-STRESS CURVES SHOW THE STRESSES IN THE CONCRETE ONLY. THE DIRECTION OF PRINCIPAL STRESSES CONSIDERS THE LINER PLATE ALSO.
4. MAX.-STRESS STANDS FOR THE MORE TENSILE WHILE MIN.-STRESS FOR THE MORE COMPRESSIVE TYPE OF THE TWO PRINCIPAL STRESSES ACTING IN THE PLANE NORMAL TO THE HOOP-STRESS WHICH IS THE THIRD PRINCIPAL STRESS.

FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
ISO - STRESS PLOT WALL
AND DOME (D+F+1.5P+T_A) INITIAL
FIGURE 5.1-9
Sheet 3



NOTES

1. THE INTERVAL OF THE ISO-STRESS PLOTS IS 50 PSI.
2. SPACING OF THE COORDINATES IS 50 IN.
3. THE ISO-STRESS CURVES SHOW THE STRESSES IN THE CONCRETE ONLY. THE DIRECTION OF PRINCIPAL STRESSES CONSIDERS THE LINER PLATE ALSO.
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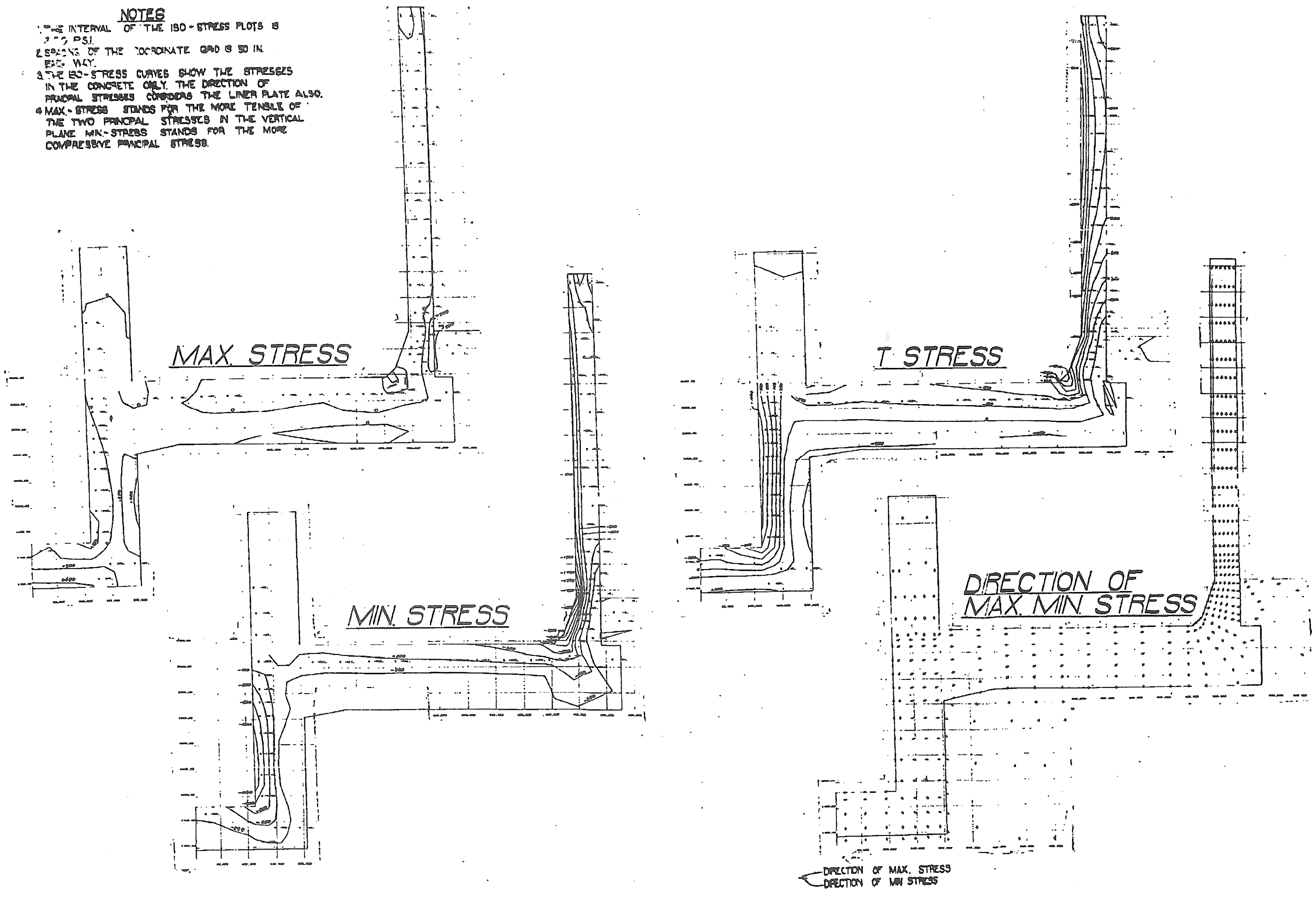
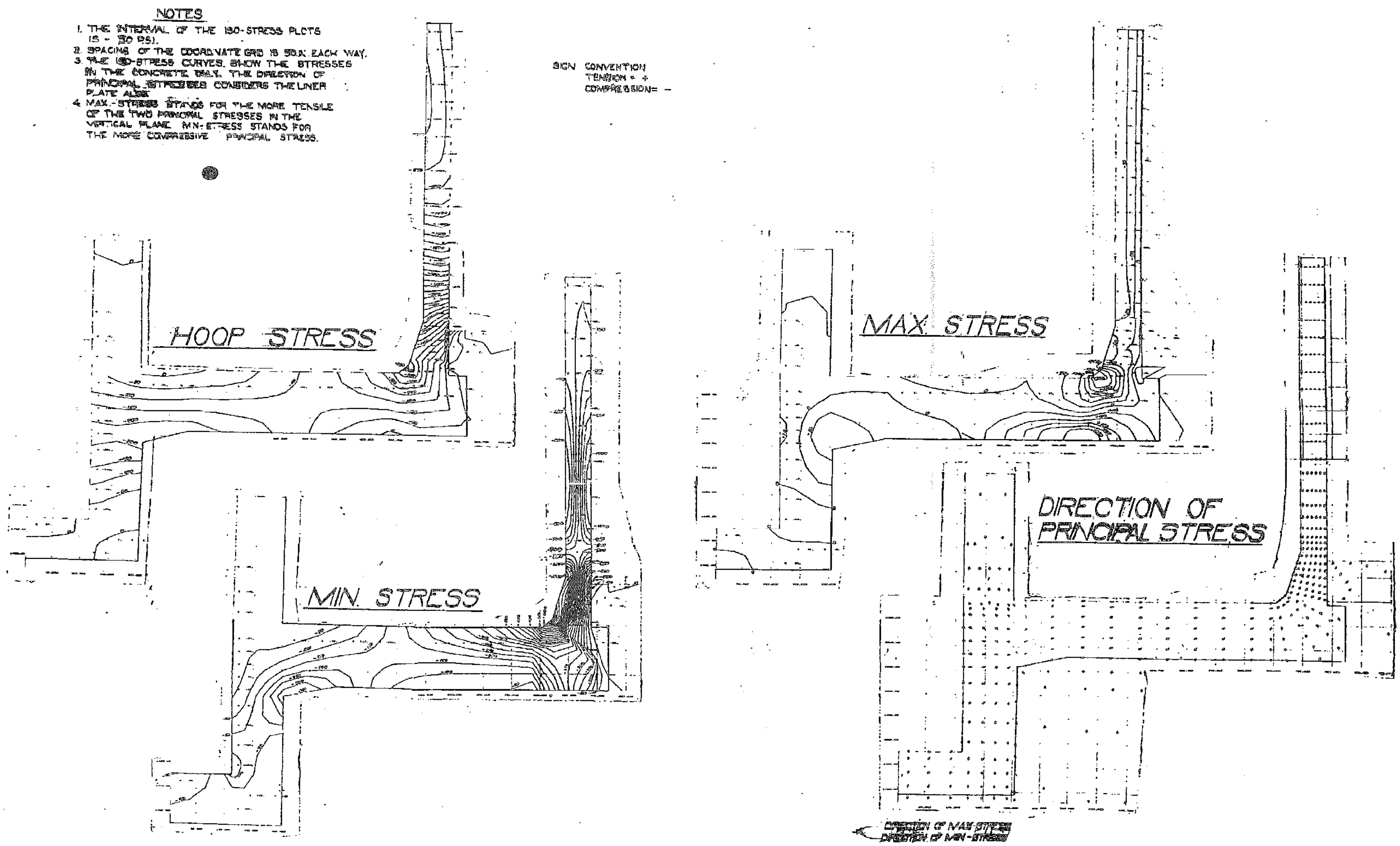
FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
ISO - STRESS PLOT WALL
AND DOME (D+F+T_A)
FIGURE 5.1-9
Sheet 4



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
ISO - STRESS PLOT BASE
AND WALL (D+F+T_A)
FIGURE 5.1-10
Sheet 1

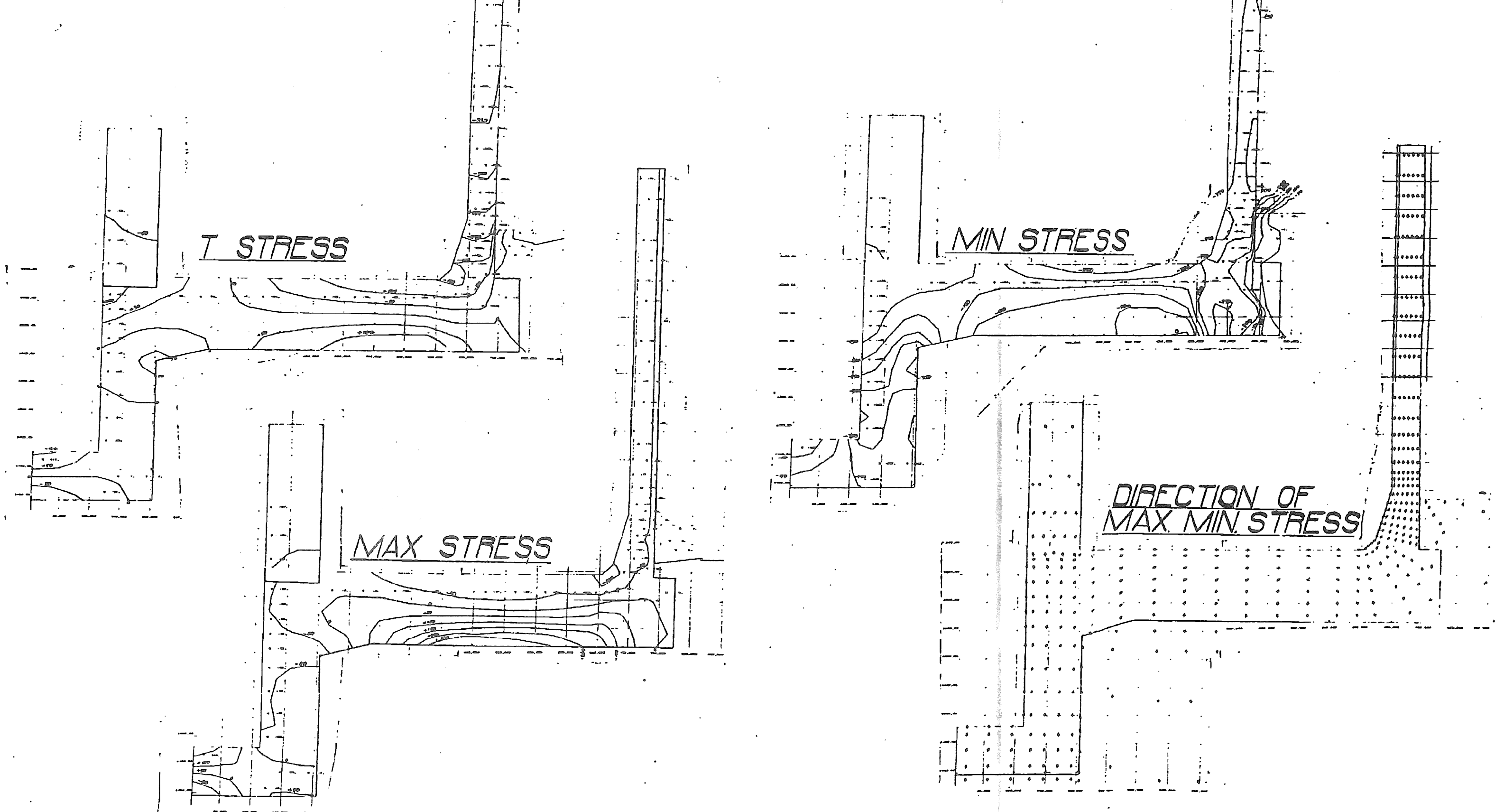


FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
ISO - STRESS PLOT BASE
AND WALL (D+F) INITIAL
FIGURE 5.1-10
Sheet 2

NOTES

1. STRESS INCREMENT = 50 PSI
2. SPACING OF THE COORDINATE GRID IS 50 PSI. EACH WAY.
3. THE ISO-STRESS CURVES SHOW THE STRESSES IN THE CONCRETE ONLY. THE DIRECTION OF PRINCIPAL STRESSES CONSIDERS THE LINER PLATE ALSO.
4. MAX.-STRESS STANDS FOR THE MORE TENSILE OF THE TWO PRINCIPAL STRESSES IN THE VERTICAL PLANE. MIN-STRESS STANDS FOR THE MORE COMPRESSIVE PRINCIPAL STRESS.



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
ISO - STRESS PLOT BASE
AND WALL (D+F+1.15P) INITIAL
FIGURE 5.1-10
Sheet 3

FIGURE 5.1-10
SHEET 4

CONTAINMENT STRUCTURE
ISO-STRESS PLOT WALL
AND DOME

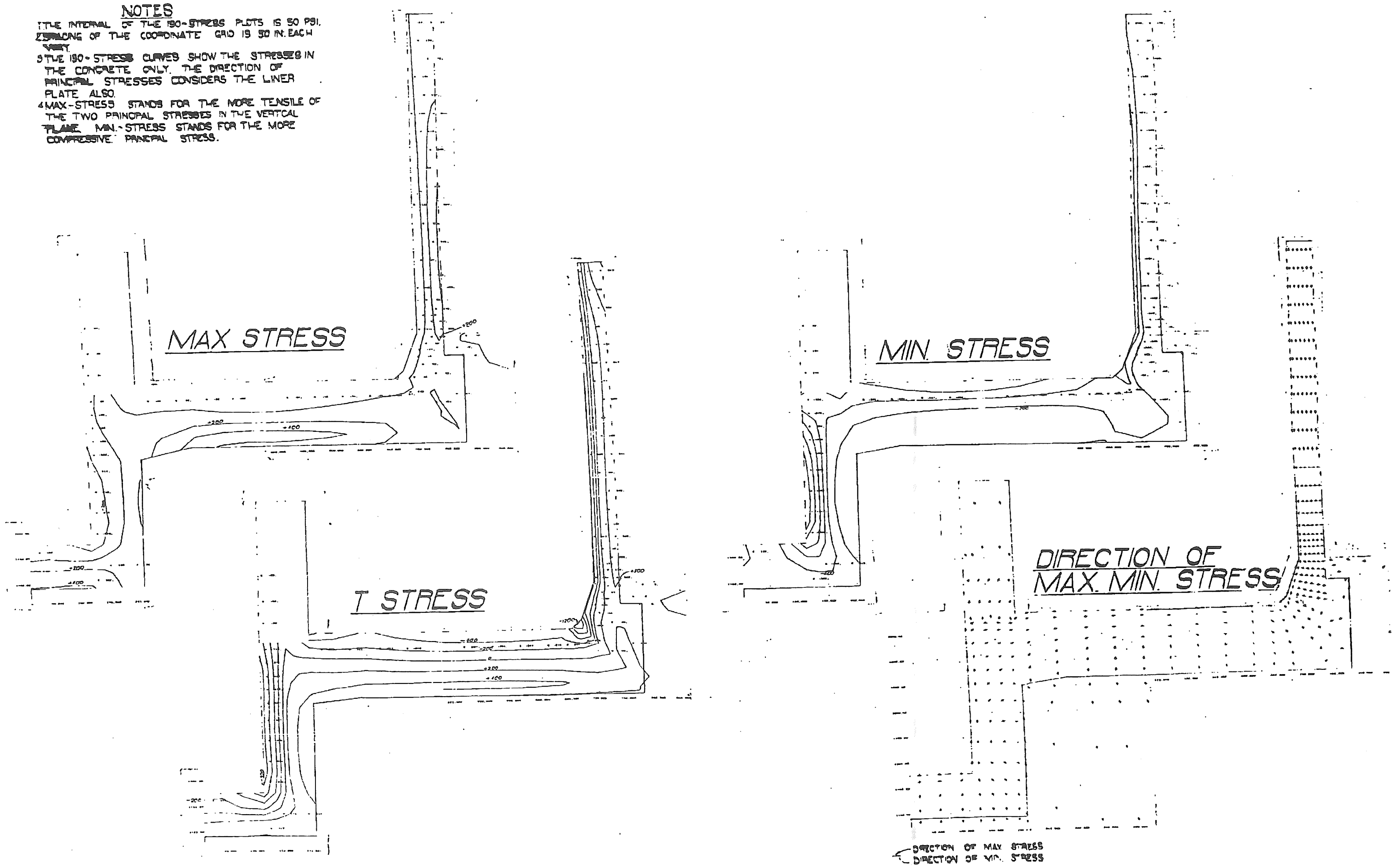
(Insert Page In Front of Figure Sheet 4)

EXPLANATORY NOTE:

The base slab stress results in this figure (Figure 5.1-10, Sht.4) are unaffected by the 1994 containment re-analysis. Refer to Appendix 5H for the update stress results relative to the containment wall.

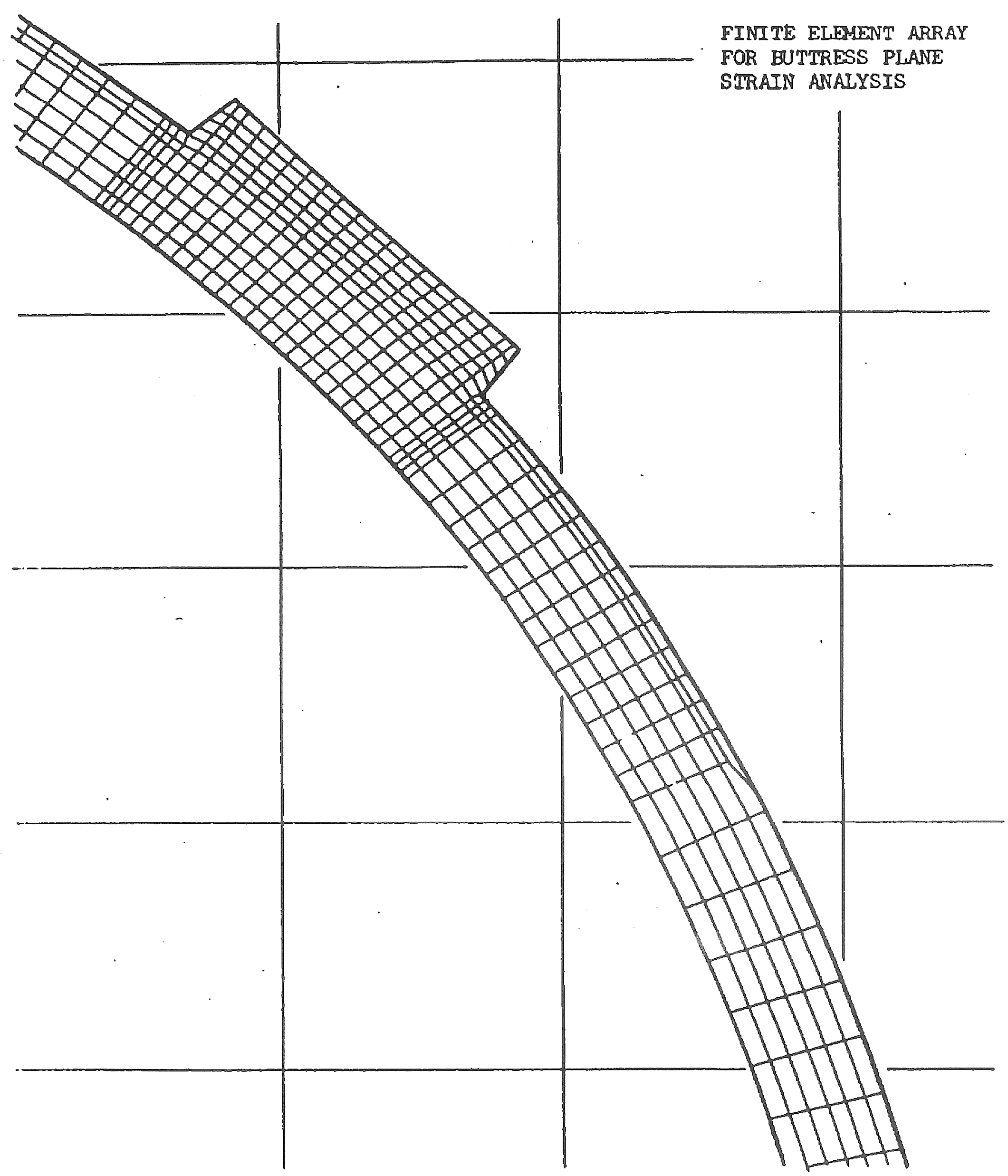
NOTES

1. THE INTERVAL OF THE ISO-STRESS PLOTS IS 50 PSI.
2. SPACING OF THE COORDINATE GRID IS 50 IN. EACH WAY.
3. THE ISO-STRESS CURVES SHOW THE STRESSES IN THE CONCRETE ONLY. THE DIRECTION OF PRINCIPAL STRESSES CONSIDERS THE LINER PLATE ALSO.
4. MAX-STRESS STANDS FOR THE MORE TENSILE OF THE TWO PRINCIPAL STRESSES IN THE VERTICAL PLANE. MIN.-STRESS STANDS FOR THE MORE COMPRESSIVE PRINCIPAL STRESS.



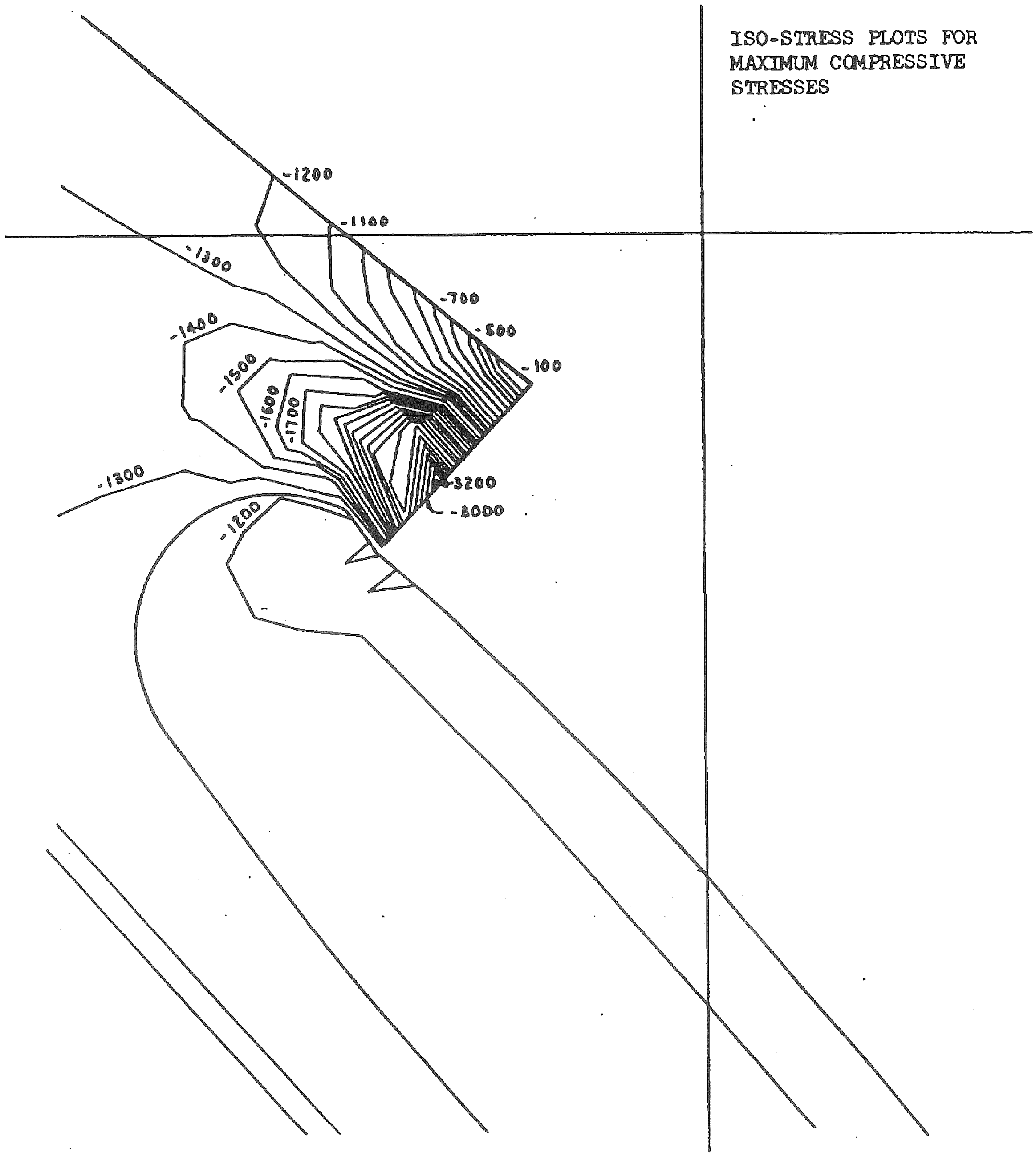
FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
ISO - STRESS PLOT BASE
AND WALL ($D+F+1.5P+T_A$) INITIAL
FIGURE 5.1-10
Sheet 4



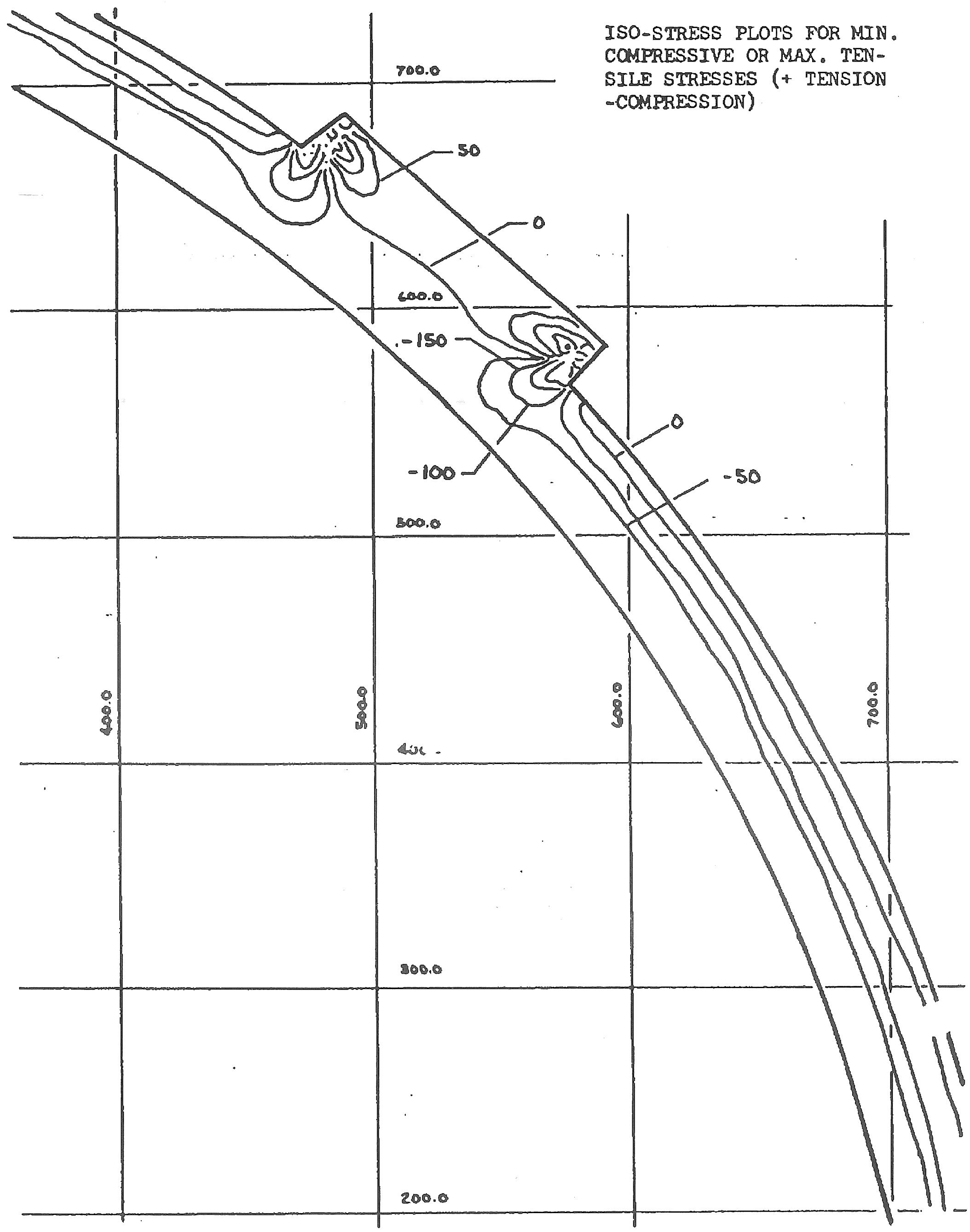
FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

CONTAINMENT STRUCTURE
FINITE ELEMENT MESH FOR BUTTRESS
FIGURE 5.1-11



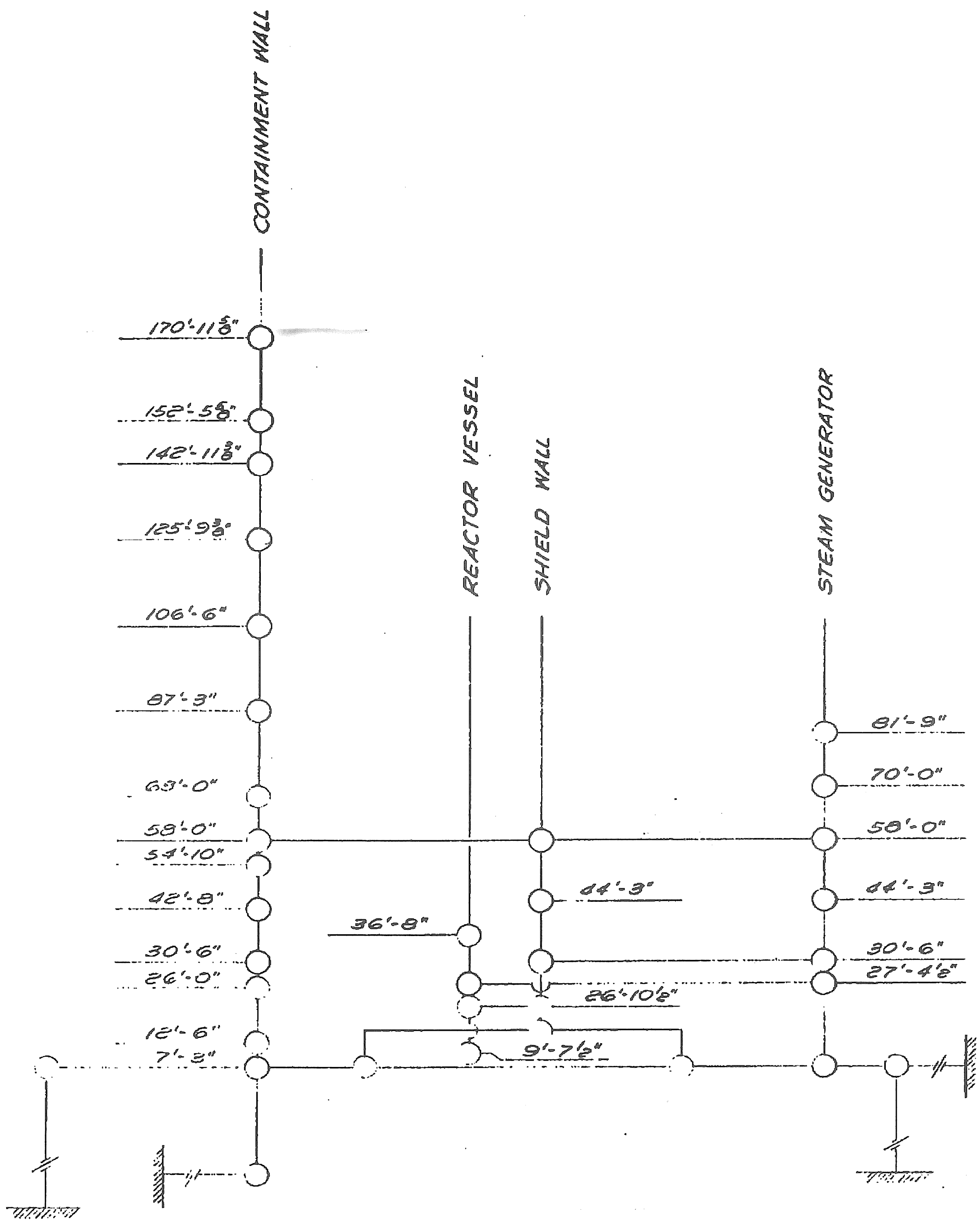
FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

CONTAINMENT STRUCTURE
ISO - STRESS PLOTS FOR BUTTRESS - MAXIMUM COMPRESSIVE STRESSES
FIGURE 5.1-12
Sheet 1



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

CONTAINMENT STRUCTURE
ISO - STRESS PLOTS FOR BUTTRESS - MIM. COMPRESSIVE OR MAX. TENSILE STRESSES
FIGURE 5.1-12
Sheet 2



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

CONTAINMENT STRUCTURE
MATHEMATICAL MASS MODEL
FIGURE 5.1-13

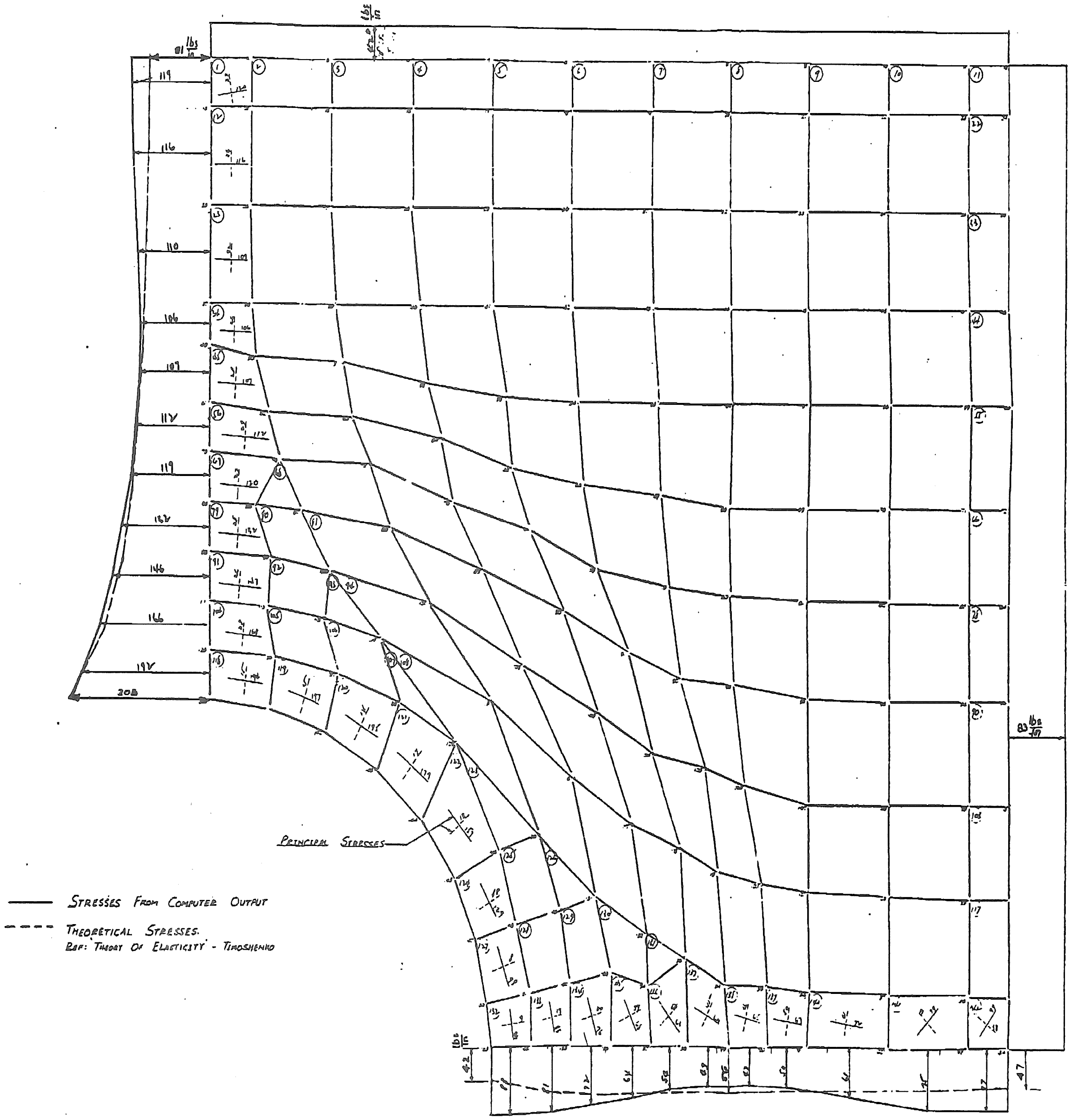
FIGURE 5.1-14

CONTAINMENT STRUCTURE
STRESS PATTERN AT EQUIPMENT
HATCH OPENING

(Insert Page In Front of Figure 5.1-14)

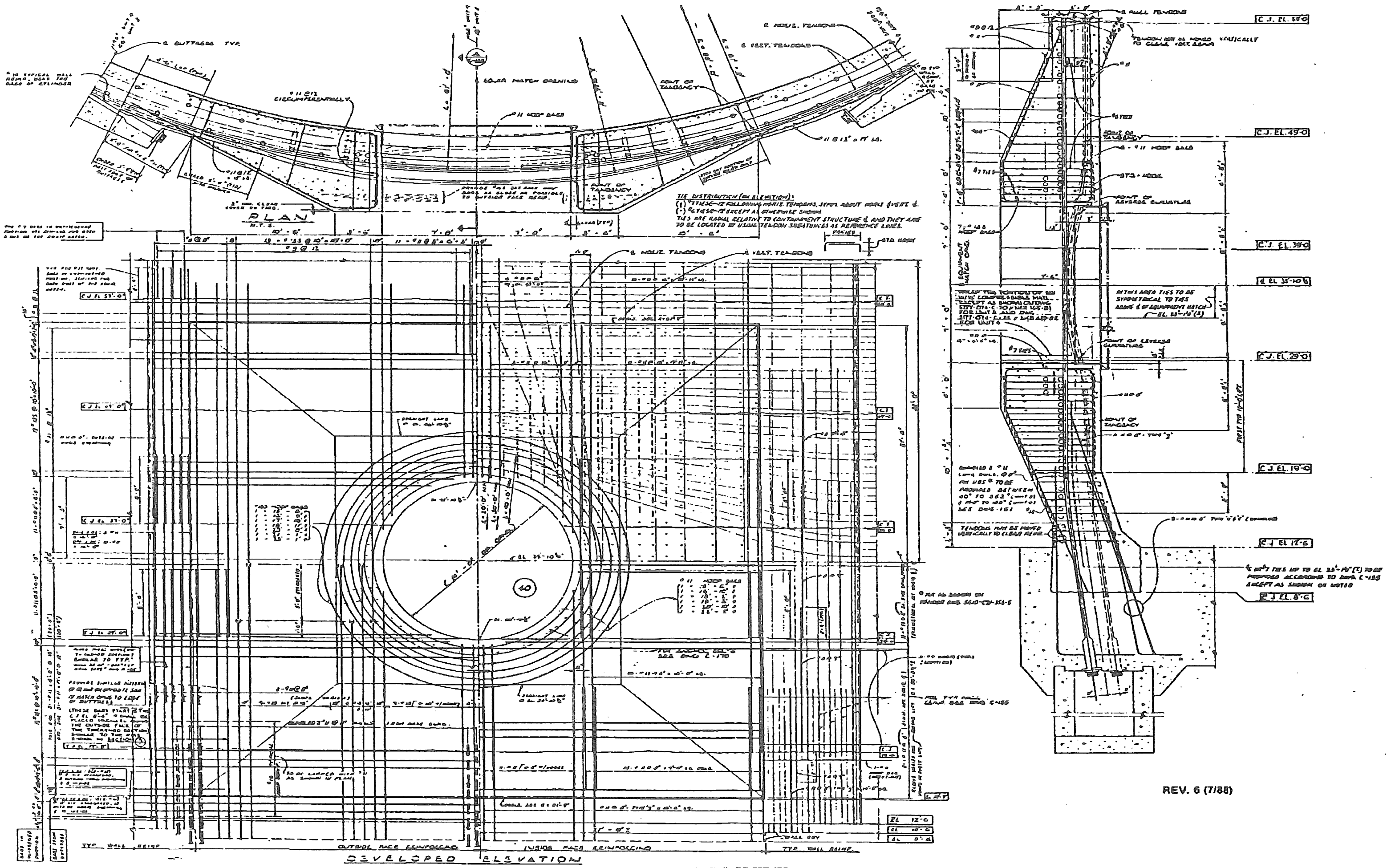
<u>EXPLANATORY NOTE</u>:

The major penetrations (equipment hatch and personnel hatch) have been analyzed as part of the containment structure re-analysis effort. These penetrations were included in the 3-D finite element model to capture the behavior of the shell in the vicinity of these large penetrations. The containment re-analysis effort (completed in 1994) is considered the updated analysis for these major penetrations. The methodology, analytical techniques, and the summary of the results are included in Appendix 5H. The information in Figure 5.1-14 is considered historical for the original analysis.



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
STRESS PATTERN AT EQUIPMENT
HATCH OPENING
FIGURE 5.1-14



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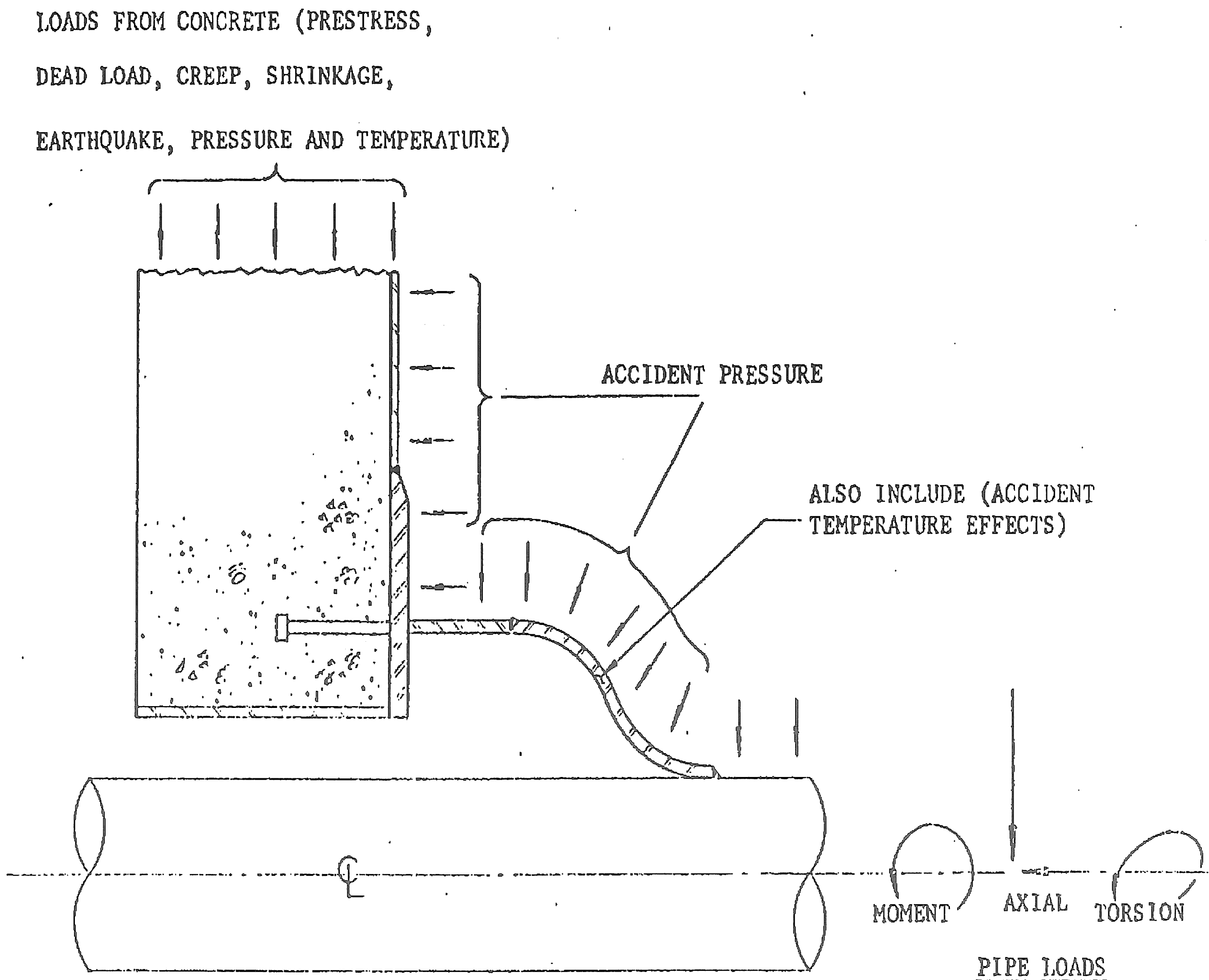
REF DWG: 5610-C-135 (REV. 4)

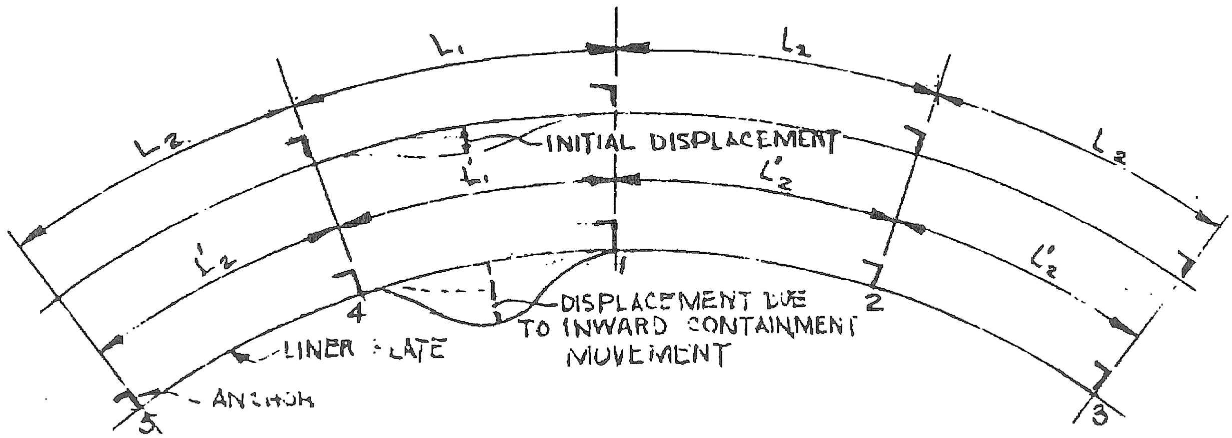
FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

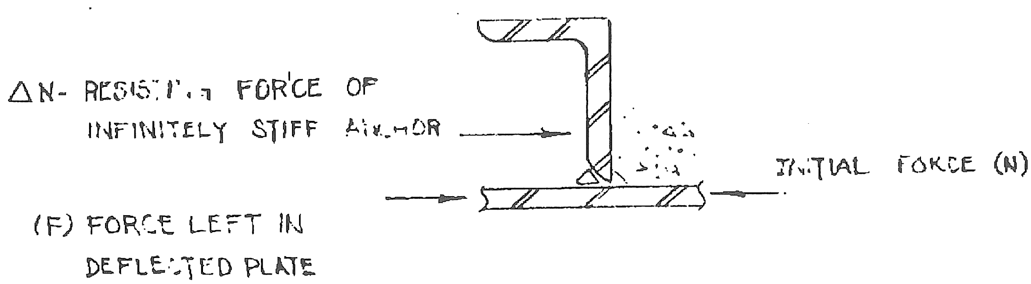
CONTAINMENT STRUCTURE
EQUIPMENT HATCH OPENING
FIGURE 5.1-15



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

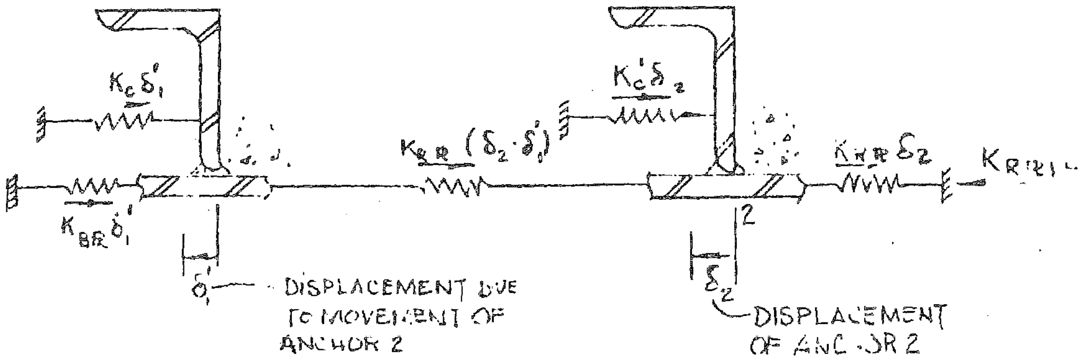
CONTAINMENT STRUCTURE
PENETRATION LOADS
FIGURE 5.1-16

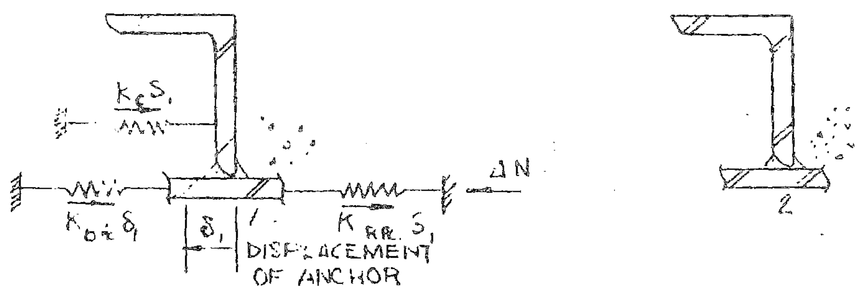




FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
MODEL FOR LINER PLATE ANALYSIS
FIGURE 5.1-17
Sheet 1





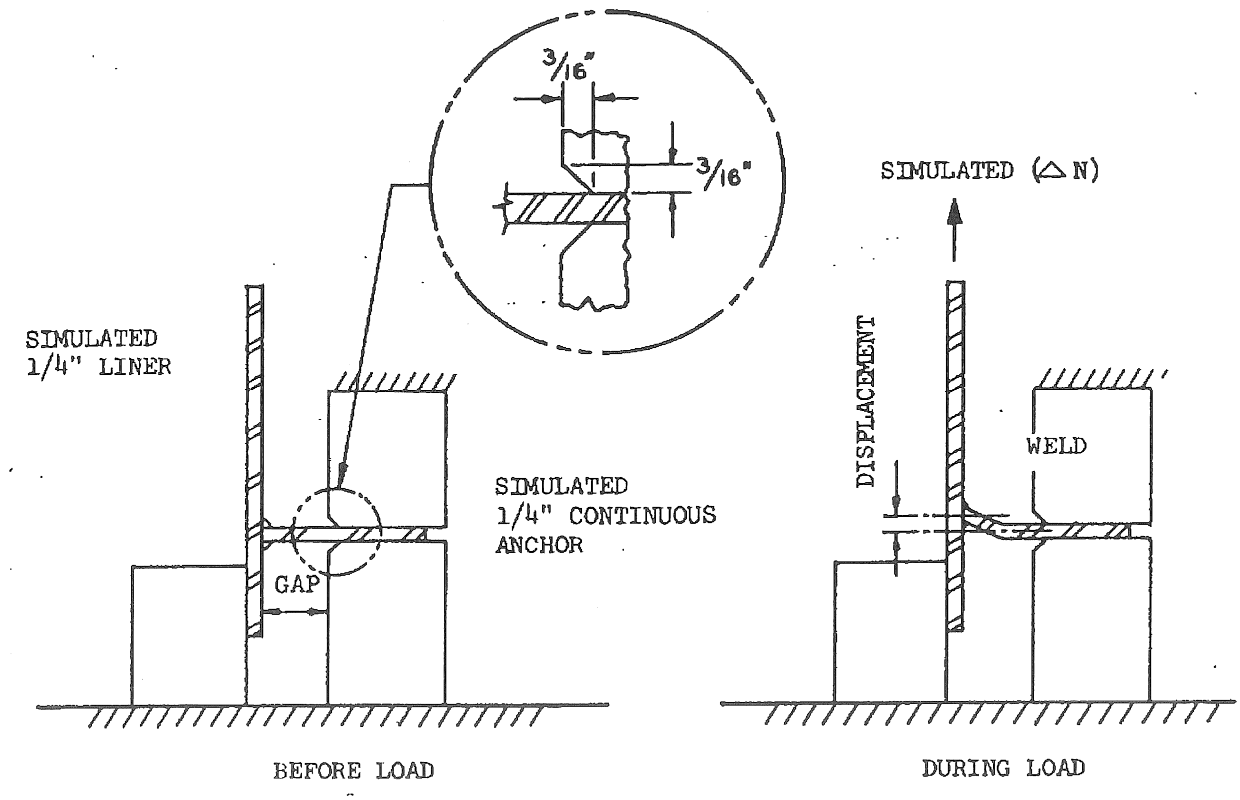
K_c - SPRING CONSTANT OF ANCHOR

K_{BR} - SPRING CONSTANT OF DEFORMED PLATE

K_{RR} - RELAXATION OF SECTION 1-2 DUE TO $\overset{\delta}{X}$.

FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

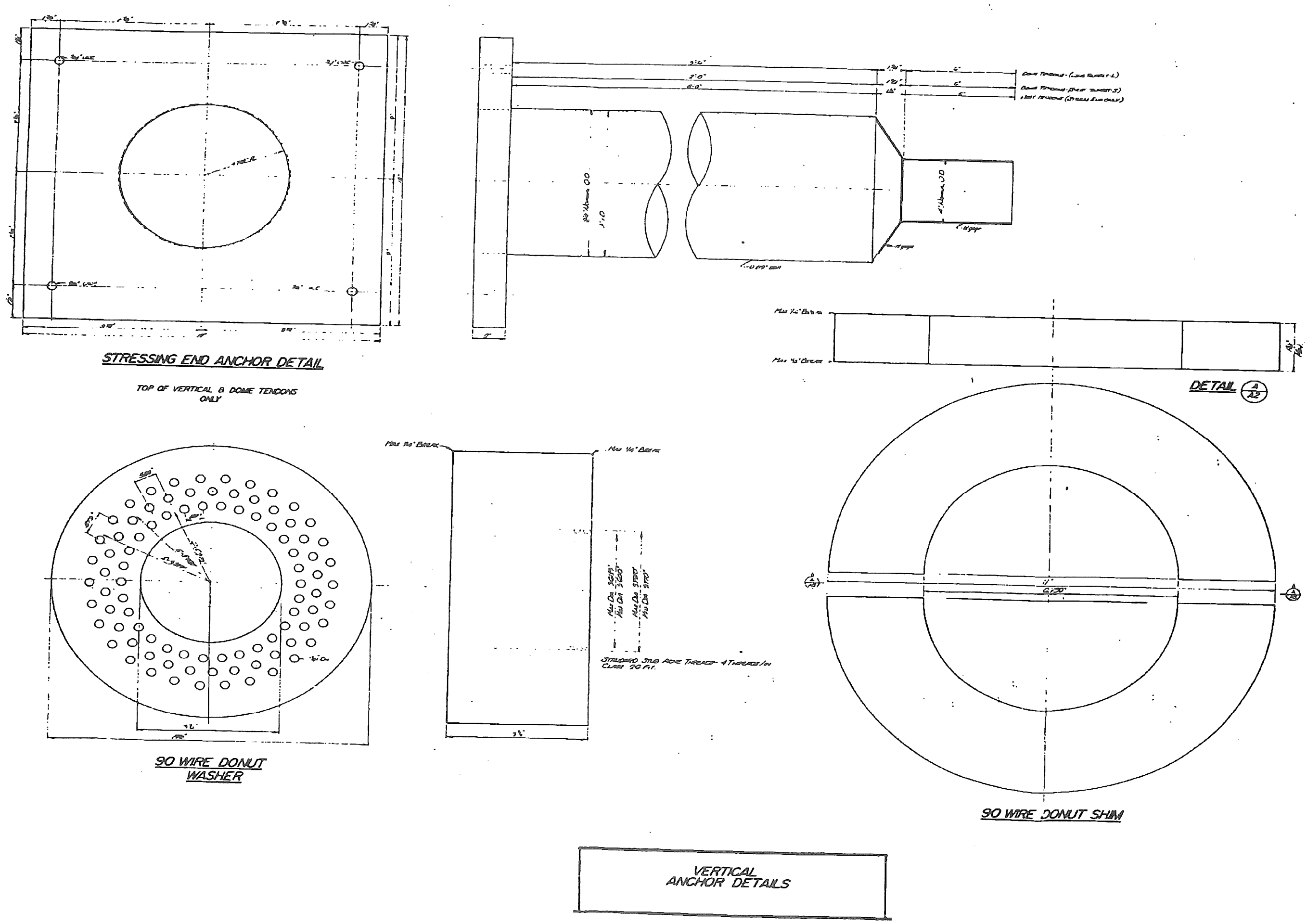
CONTAINMENT STRUCTURE
MODEL FOR LINER PLATE ANALYSIS
FIGURE 5.1-17
Sheet 2



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

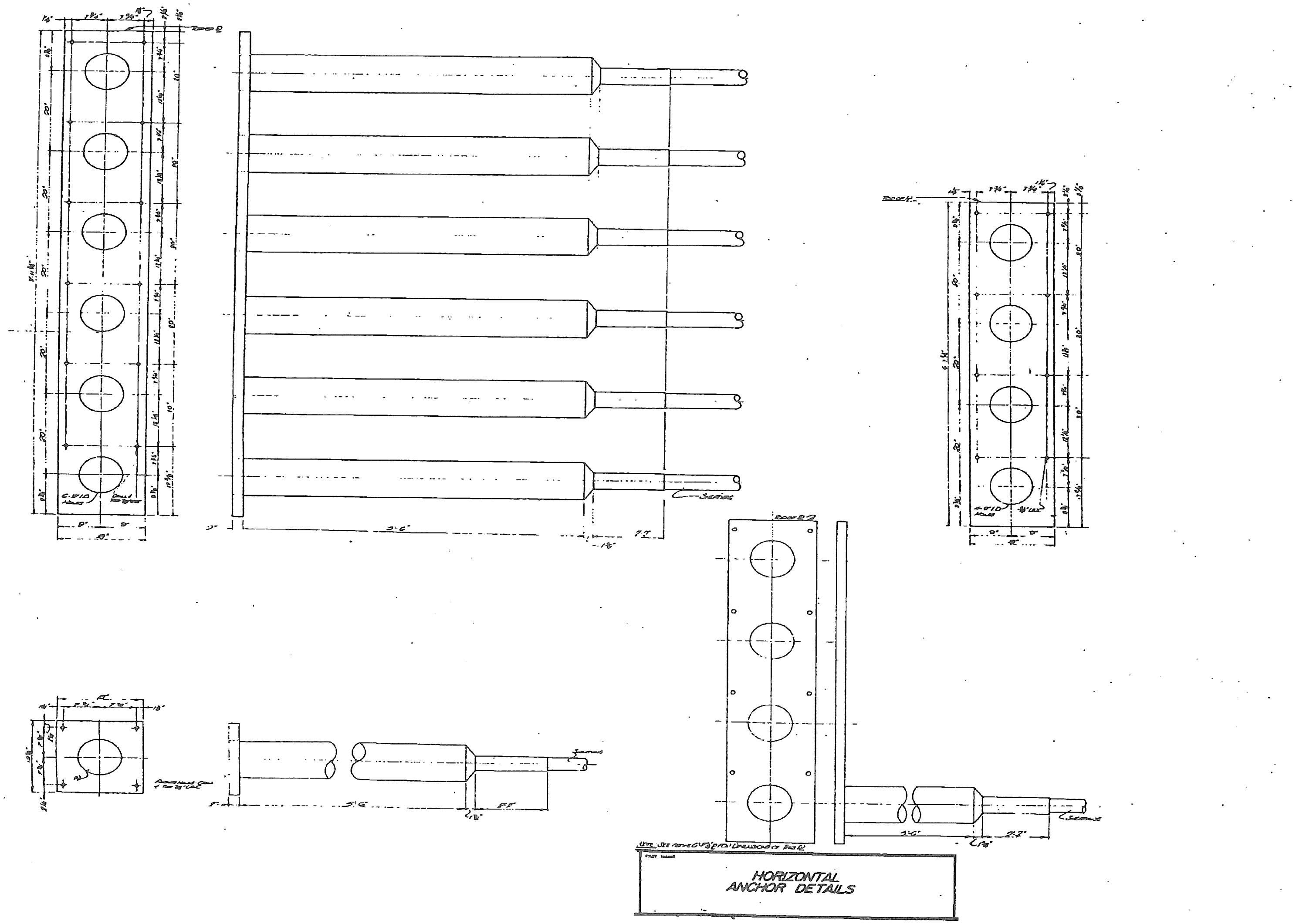
FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
MODEL FOR LINER PLATE ANALYSIS
FIGURE 5.1-18



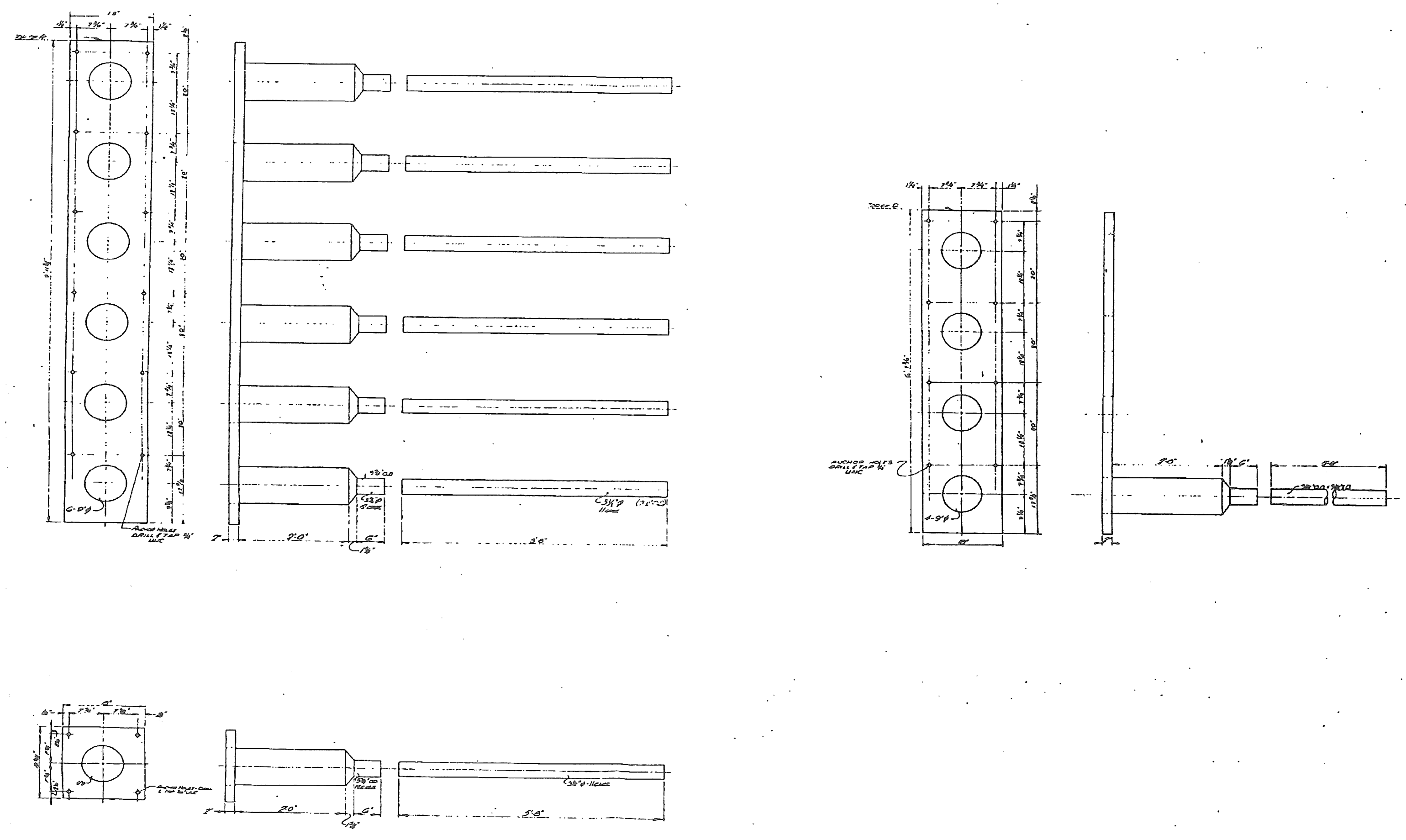
FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
PRESTRESS TENDON HARDWARE ASSEMBLY
FIGURE 5.1-19
Sheet 1



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

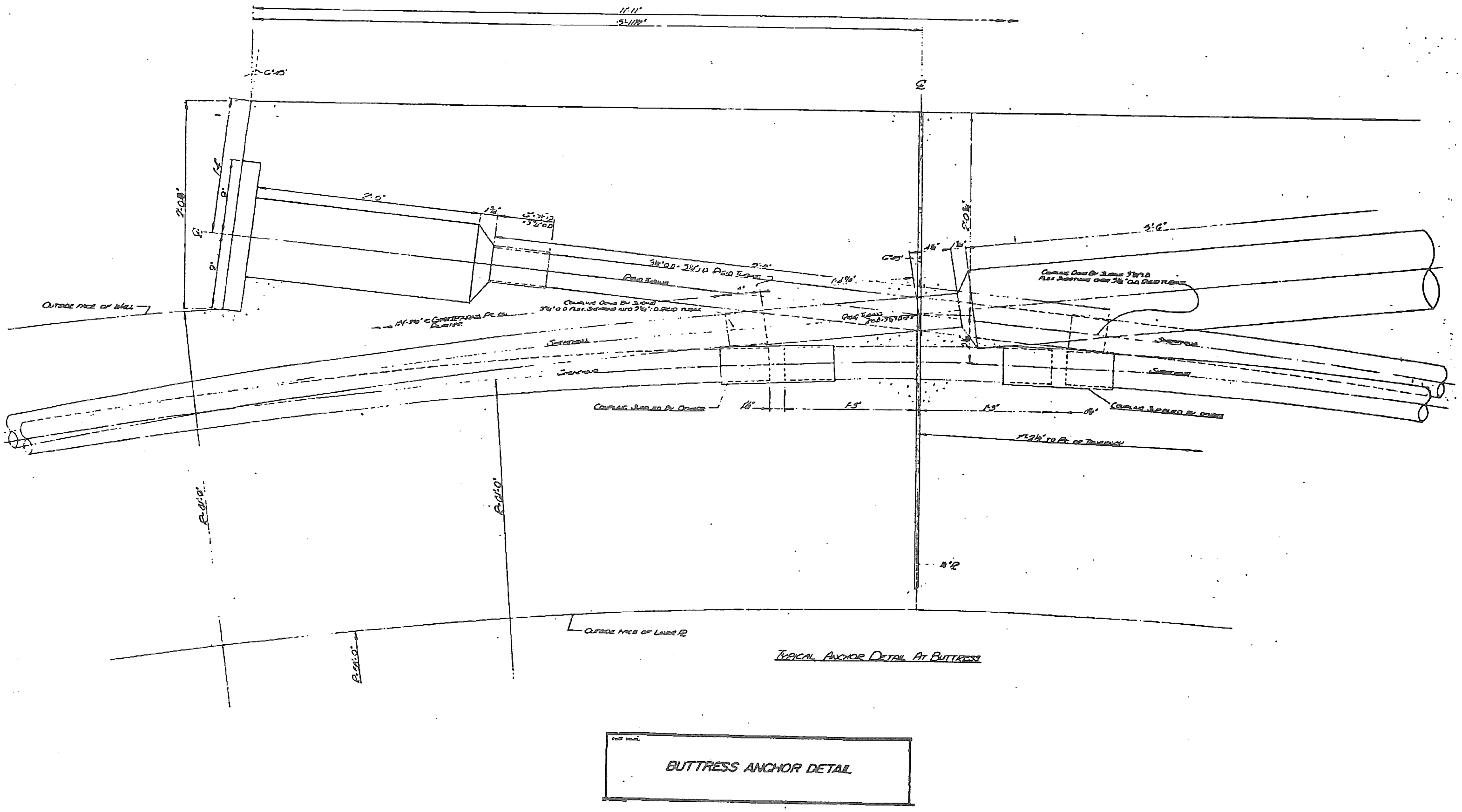
CONTAINMENT STRUCTURE
PRESTRESS TENDON HARDWARE ASSEMBLY
FIGURE 5.1-19
Sheet 2



HORIZONTAL ANCHOR DETAILS

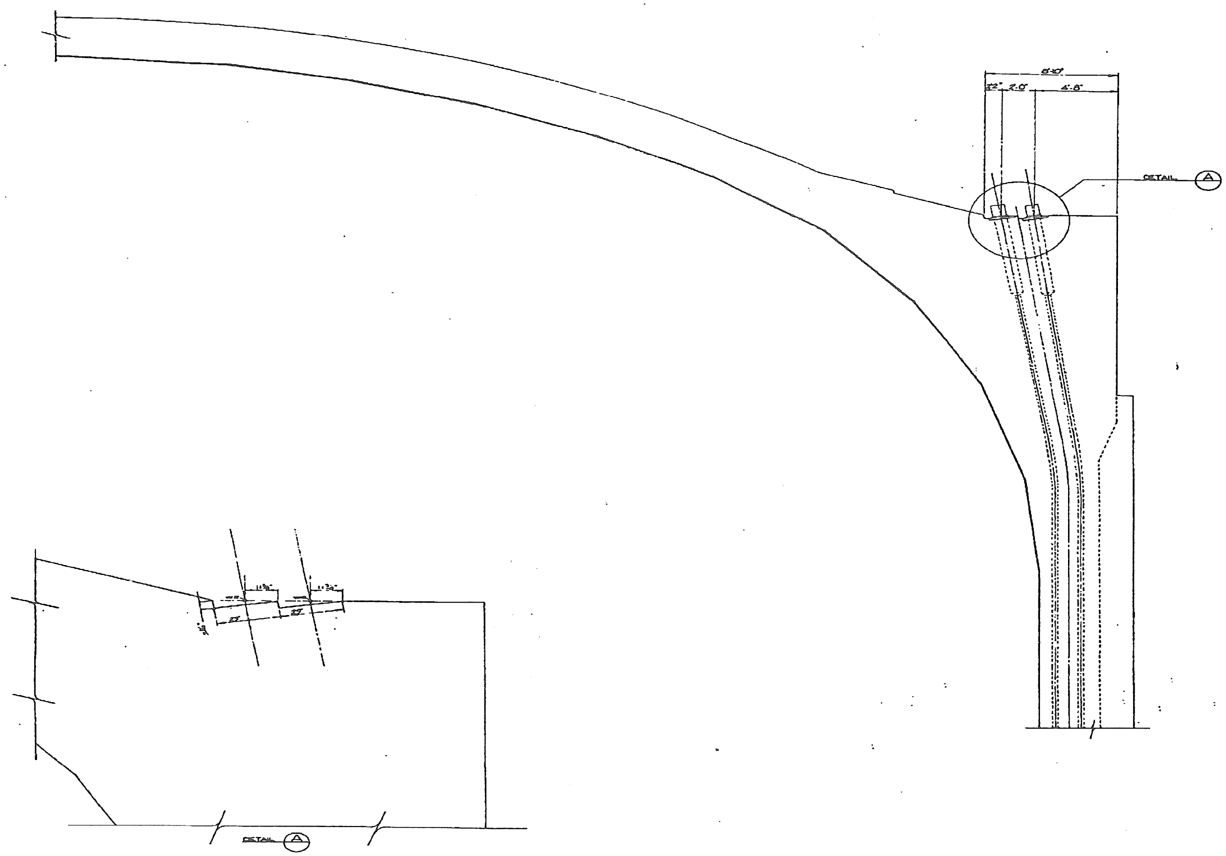
FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
PRESTRESS TENDON HARDWARE ASSEMBLY
FIGURE 5.1-19
Sheet 3



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
PRESTRESS TENDON HARDWARE ASSEMBLY
FIGURE 5.1-19
Sheet 4



POCKET DETAILS
FOR TOP VERTICAL
ANCHORS

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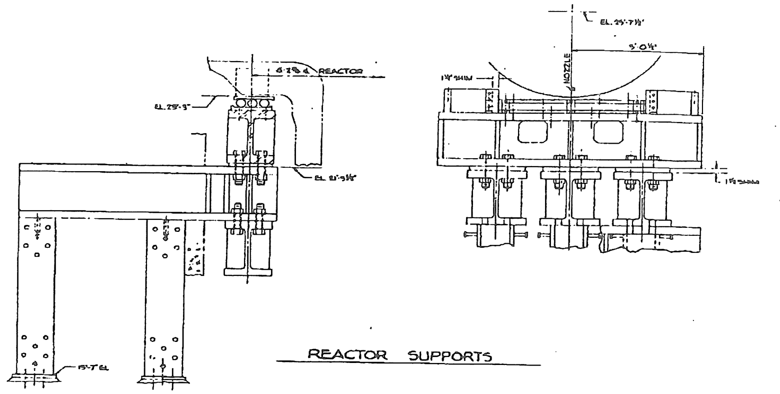
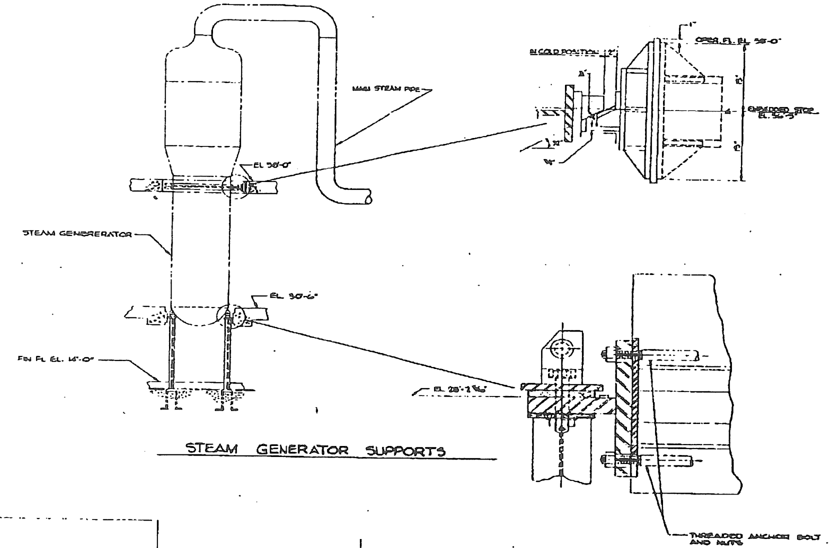
TURKEY POINT PLANT

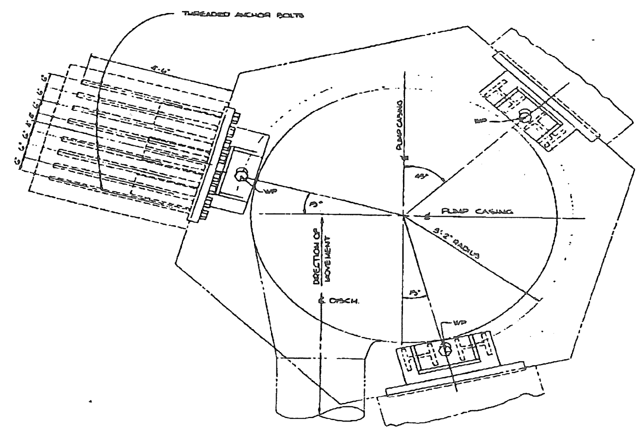
CONTAINMENT STRUCTURE
PRESTRESS TENDON HARDWARE ASSEMBLY
FIGURE 5.1-19
Sheet 5



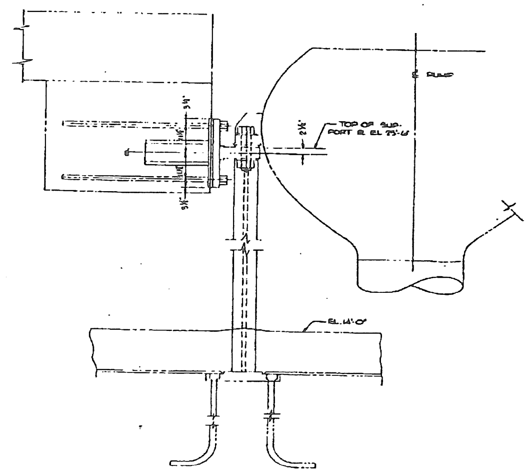
REACTOR SUPPORTS



STEAM GENERATOR SUPPORTS



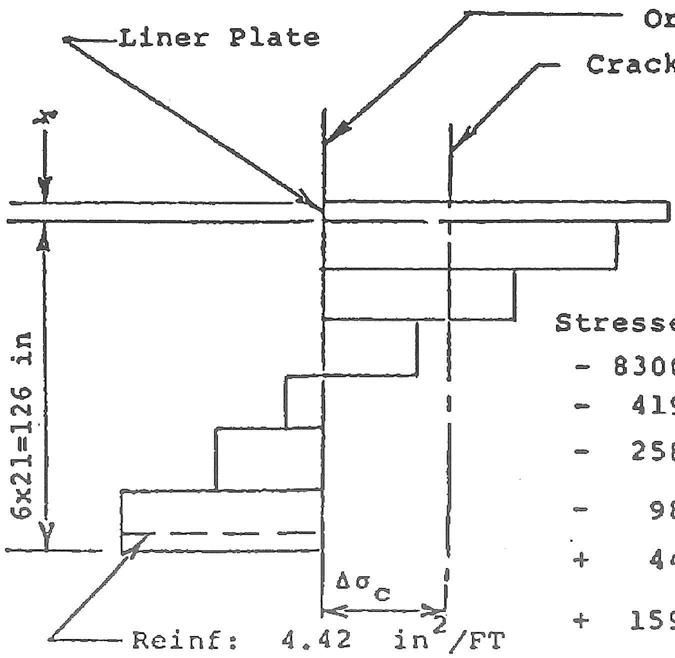
PLAN – PUMP SUPPORTS



SECTION

FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

CONTAINMENT STRUCTURE
MAJOR EQUIPMENT SUPPORTS
FIGURE 5.1-20



Stresses (psi)	Resultants (K/FT)
- 8300	- 24.0
- 419	- 105.7
- 258	- 65.0
- 98	- 24.3
+ 44	+ 11.1
+ 159	+ 40.0
+ 293	+ 73.8

NOTE :

THIS DIAGRAM IS ASSOCIATED WITH THE ORIGINAL CONTAINMENT ANALYSIS. THE NUMERICAL EXAMPLE GIVEN IN SECTION 5.1.3 USES THIS STRESS DIAGRAM TO DEMONSTRATE THE ANALYTICAL METHOD FOR THERMAL CRACK ANALYSIS.

FOR 1994 CONTAINMENT ANALYSIS METHODOLOGY AND RESULTS REFER TO APPENDIX 5H.

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FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

LINER PLATE
STRESS BLOCK

FIGURE 5.1-21

5.2 AUXILIARY BUILDING

5.2.1 General

The Class I systems and components which are located in the Auxiliary Building are shown in Appendix 5A.

5.2.2 Design Basis

The areas of the auxiliary building housing Class I systems have been designed and constructed to Class I requirements. The following loads and conditions have been considered:

1. All normal dead and live loads, external hydrostatic pressures, wind and earthquake loads. The loads are in accordance with the design criteria, Appendix 5A.

2. High velocity wind loads due to tornadoes and the effect of missiles generated from tornadoes in accordance with Appendix 5A.

3. Flooding in the room housing the engineered safeguards systems due to pipe rupture and the resulting hydrostatic load.

4. Vertical, lateral and steam jet loading resulting from rupture of high pressure piping.

5.2.3 Design Codes and Design Description

Design Codes

The building has been designed and constructed in accordance with "Building Code Requirements for Reinforced Concrete", ACI 318-63" and "Specification for the
Design, Fabrication and Erection of Structural Steel for Building", 1963 edition. Dead and live loads meet the requirements of the American Standard Building Code and the South Florida Building Code.

Wind and earthquake loadings, load factors and load combinations as specified in Appendix 5A have been used in the design of this building.

The major structural materials used in the construction of the Auxiliary Building are as follows:

Concrete	f'_c = 3000 psi at 28 days
Reinforcing bars	ASTM-A-15 (ASTM-A-615 Gr. 40)
Structural Steel	ASTM-A-36 (Original), A-992 (equiv), A-572 GR 50 (Equiv)
High Strength Bolts	ASTM-325

C26

Design Description

The building is constructed on a foundation mat with concrete bearing walls and slabs. Earthquake, wind and other appropriate lateral loads are resisted by diaphragm action of the walls and slabs. Ductile behavior of all the walls and slabs is maintained for better resistance of dynamic loads.

The new and spent fuel pit walls are designed to resist the effects of tornado, earthquakes, wind and missiles. The walls are also designed to withstand thermal stresses associated with the steady state thermal gradient of 150 F.

The building is designed to remain within the elastic limit under the action of a tornado wind of 225 mph acting simultaneously with a differential pressure of 1.5 psi. No-loss-of-function will be experienced under the action of a wind of 337 mph and a pressure of 2.25 psi.

In those cases where the engineered safeguards equipment is separated into two rooms, the partition wall is designed to withstand hydrostatic loading over the required height.

5.2.4 Fuel Storage Considerations

The Unit 3 and 4 spent fuel storage pit capacity has been increased by installing redesigned rigid high density stainless steel racks. The first redesign of the Unit 3 and 4 spent fuel storage pit capacity allowed a reduction in the center to center distance between the spent fuel assemblies from 21 to 13.659 inches. The current redesign has created two regions in the spent fuel pits. In region 1, the center to center distance between spent fuel assemblies has decreased from 13.659 inches to 10.6 inches except the cask pit rack, when installed. The cask pit rack Region I cells have a center-to-center distance of 10.1 inches E-W and 10.7 inches N-S. In region 2, the center to center spacing has decreased from 13.659 to 9.0 inches. The racks are submerged in borated water with the minimum boron concentration of 1950ppm. The provision for makeup water, rack spacing, presence of a sufficient soluble boron concentration, and the materials used are adequate to avoid criticality ($K_{eff} \leq 0.95$). The redesign enabled an increase in the capacity of the racks from 621 spent fuel assemblies (equivalent of approximately 4 cores) to 1404 assemblies (equivalent of 9 cores) at Units 3 and 4. When the cask area rack is installed, there is additional storage space for 131 fuel assemblies. To assure leak tightness of the fuel pool, the walls and floor are lined with a leak-tight stainless steel liner 1/4" thick.

Monitoring trenches are provided behind the liner for collecting and detecting any leaks. Any leakage is directed to the waste disposal drainage system, thus preventing uncontrolled leakage of fuel pool water.

The new fuel assemblies for a 1/3 core are stored in rigid racks. External flood protection for the storage area is discussed in Appendix 5G. Substructures are designed to resist flood tide buoyancy effects.

5.3 OTHER STRUCTURES

5.3.1 CONTROL BUILDING

The Control Building houses the following :

1. Reactor Control Rod Drive Equipment and 3B/4B Motor Control Centers

2. Cable Spreading Room and Battery Room

3. Control Room

4. Computer Room

The Control Building is a reinforced concrete structure, designed to Class I requirements similar to the Class I areas of the Auxiliary Building described in Section 5.2. Special ventilation and fire protection systems are provided as discussed in Sections 9.9 and 9.6, respectively.

5.3.2 INTAKE STRUCTURE, UNIT 3 EMERGENCY DIESEL ENCLOSURE, AND SWITCHGEAR ENCLOSURE

Intake cooling water pumps are Class I components supported by the Intake Structure. The structure is designed to Class I requirements. The external flood protection for intake cooling water pump motors is described in Appendix 5G. The pumps are designed for missile protection by separation and redundancy as described in Appendix 5E.

The Unit 3 emergency diesel generators and the switchgear equipment are located in separate reinforced concrete enclosures. The structures are designed to Class I requirements and to resist dead load, live load, hurricane and tornado winds, and the external missiles. Refer to Section 5.3.4 for the Unit 4 Emergency Diesel Generator Structures.

For a complete listing of Class I equipment, systems and structures, refer to Appendix 5A.

For a description of the external missiles protection, and a listing of equipment, systems and structures designed for the external missiles, refer to Appendix 5E.

5.3.3 RADWASTE SOLIDIFICATION BUILDING

The Radwaste Solidification Building houses the liquid and solid radwaste handling equipment.

The Radwaste Solidification Building is a reinforced concrete structure, designed to Class I requirements described in Appendix 5A.

5.3.4 UNIT 4 EMERGENCY DIESEL GENERATOR STRUCTURES

5.3.4.1 DESCRIPTION

The location of the Unit 4 Emergency Diesel Generator (EDG) Building is shown on the general building arrangement plan, Figure 1.2-1. The general arrangement of the Unit 4 EDG Building and diesel oil storage facility is shown on Figure 1.2-8.

The Unit 4 EDG Building, including the diesel oil storage facility, has been constructed as a free standing structure on a common foundation mat at a site located between Units 2 and 3, just west of the existing water treatment plant. It is not located adjacent to any other structure.

The Unit 4 EDG Building contains the 4A and 4B emergency diesel generators, air start skids, control panels, 4.16kV Swing Switchgear, 480V Motor Control Centers, and other auxiliary equipment. The diesel oil storage facility contains the two diesel oil storage tanks and diesel oil transfer pumps. The Unit 4 EDG Building and diesel oil storage facility is a Seismic Category I structure, designed for the effects of earthquakes, tornados, and hurricanes.

The Unit 4 EDG Building, which houses the emergency diesel generators, is partitioned into two halves, one for each emergency diesel generator set. The building consists of two stories: (1) the ground floor for the emergency diesel generators; and (2) the second floor for the balance of equipment noted above. The second floor is partially reinforced concrete and partially structural steel with steel grating. The grating areas will allow air flow between floors. In addition, the building houses diesel oil storage tanks which are steel lined concrete pools. There are two diesel oil storage tanks.

A chain-operated, underhung bridge crane above each emergency diesel generator set is provided for maintenance purposes. Each crane is able to traverse the full length of the emergency diesel generator.

The diesel radiators exhaust through the heavy steel missile shield grating on the south side. The two exhaust areas are separated by a reinforced concrete T-shaped fire wall to provide separation between the redundant trains.

Concrete block knock-out panels are provided for removal of the emergency diesel generators. The missile shield grating in this area is also removable. Equipment hatches are provided in the building roof for removal of smaller equipment.

The Unit 4 EDG Building includes the diesel oil storage facility which contains the diesel oil transfer pumps and storage tanks. The diesel oil tanks are designed as steel lined concrete pools. The steel liners have been fabricated in accordance with the requirements of ASME Code Section VIII and meet Seismic Category I requirements. Leak detection is provided behind the liner plate. The leak detection consists of a series of interconnected grooves in the concrete surface behind the liner plate that drain to a common sump. Isolation valves between the tank and pumps are located in the pump room. In the event of a pipe break between the isolation valve and fuel oil tank, the entire inventory of fuel oil will be contained in the pump room/oil tank area. Access to the diesel oil storage tanks is provided by roof hatches.

The Unit 4 EDG Building and diesel oil storage facility provides tornado missile protection for the safety related equipment inside. All critical exterior openings are covered by gratings, labyrinths, exhaust covers, etc., which are designed to resist the postulated missiles. The concrete interior walls of the structure protect each train from internal missiles that could damage the other train. The internally generated missiles considered in the design of the Unit 4 EDG structures are the EDG piston assembly, connecting rod, piston pin, fan blade, or an air receiver relief valve.

Diesel oil transfer piping between the diesel oil transfer pumps and the diesel oil day tanks are embedded in the building foundation mat.

To integrate the 4A and 4B EDGs into the plant system, an extensive underground ductbank system is provided.

On the north end of the Unit 4 EDG Building, ductbanks are designed to provide cable routes for the Unit 4 EDG main feeds, 4.16kV Switchgear 4D/3D feeders, MCC 4J and 4K feeders and most associated power, control, instrumentation, communication, and fire detection cables. These ductbanks originate in four manholes, 708 through 711, within the Unit 4 EDG Building (two for 4.16kV and two for 480V power and control). These ductbanks enter the plant exposed raceway system via conduit stub-ups.

The conduit stub-ups terminate in concrete pads located in an area extending from north of the Unit 3 4.16kV Switchgear Room to north of the Unit 3 EDG Building. These conduit stub-ups are connected to sections of exposed conduits and pullboxes. These exposed conduit sections and pullboxes provide a direct tie between the Unit 3 and 4 exposed raceways and the EDG underground ductbank system. These ductbanks have been provided with eight additional outdoor manholes (700 through 707) to accommodate cable pulling requirements. In addition, a ductbank extends to the west from manhole 701 and enters the plant via cable tray which connects directly to the ductbank through a pull box in the vicinity of the Unit 4 Start-up Transformer. This ductbank has been provided with an additional outdoor manhole (760) to accommodate cable pulling requirements.

On the south end of the Unit 4 EDG Building, manholes 712 and 713 are attached to the building. Manhole 712 is provided for future 3D/4D 4.16kV loads. Manhole 713 connects to a "C" (480V power and control) ductbank running south and terminating in conduit stub-ups in the vicinity of the CCW valve pit. In addition, ductbanks for 480V power and control cables enter the existing plant underground system in existing manholes 308 and 310 via manholes 714 and 715, which are attached to the Unit 4 EDG Building. Ductbanks connect existing manholes 308 and 310 with the existing plant system. Also, ductbanks are provided for the 4.16kV feeders to the "C" ICW and CCW pumps via manholes 716 and 717. All of the above six additional manholes (712 through 717) have been provided to accommodate cable pulling requirements.

5.3.4.2 APPLICABLE CODES, STANDARDS, AND SPECIFICATIONS

Codes, standards and specifications listed in Subsections 5.1.6, 5.2 and 5.3, as well as the following, are applicable to the Unit 4 EDG Seismic Category I structures described in this section.

USNRC Regulatory Guides	Title
1.29 (Rev. 3)	Seismic Design Classification
1.59 (Rev. 2)	Design Basis Floods for Nuclear Power Plants
1.60 (Rev. 1)	Design Response Spectra for Seismic Design of Nuclear Power Plants
1.61 (Rev. 0)	Damping Values for Seismic Design of Nuclear Power Plants
1.76 (Rev. 0)	Design Basis Tornado for Nuclear Power Plants
1.92 (Rev. 1)	Combining Modal Responses and Spatial Components in Seismic Response Analysis
1.94 (Rev. 1)	Quality Assurance Requirements for Installation and Testing of Structural Concrete and Structural Steel During the Construction Phase of Nuclear Power Plants
1.102 (Rev. 1)	Flood Protection
1.115 (Rev. 1)	Protection against Low-Trajectory Turbine Missiles
1.117 (Rev. 1)	Tornado Design Classification
1.122 (Rev. 0)	Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components
1.132 (Rev. 1)	Site Investigations for Foundations of Nuclear Power Plants

USNRC Regulatory Guides (Continued)	Title
1.138 (Rev. 0)	Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants
1.142 (Rev. 1)	Safety-Related Concrete Structures for Nuclear Power Plants (other than Reactor Vessels and Containment)

USNRC NUREG-0800 Standard Review Plan (SRP)	
Section 3.5.3 (Rev. 1-July 1981)	Barrier Design Procedures
Section 3.8.4 (Rev. 1-July 1981)	Other Seismic Category I Structures
Section 3.8.5 (Rev. 1-July 1981)	Foundations

ANSI Standards	
ANSI A58.1-82	Minimum Design Loads for Buildings and Other Structures
ANSI N45.2.5	Supplementary Quality Assurance Requirements for Installation, Inspection and Testing of Structural Concrete, Structural Steel, Soils, and Foundations during the Construction Phase of Nuclear Power Plants

ACI Codes	
ACI 349-85	Code Requirements for Nuclear Safety Related Concrete Structures
ACI 318-83	Building Code Requirements for Reinforced Concrete

Other Codes and Standards

South Florida Building Code, 1984 Edition

AISC Manual of Steel Construction, Eighth Edition

ASME Boiler & Pressure Vessel Code, 1983 Edition through Summer 1984

5.3.4.3 LOADS AND LOADING COMBINATIONS

The Seismic Category I Unit 4 EDG Building including the diesel oil storage facility and safety related ductbanks and manholes have been designed in accordance with the requirements for Class I structures, employing the loads, load combinations, and structural acceptance criteria specified in Appendix 5A as a minimum. In addition, more recent regulatory criteria for Seismic Category I structures has been employed as listed in Subsection 5.3.4.2. Structural design includes dead, live, thermal, seismic, wind and tornado loads.

5.3.4.3.1 DESIGN LOADS

Design loads for Class I structures are defined in Appendix 5A and are supplemented with the following for Seismic Category I structures.

Seismic loads are based upon an Operating Basis Earthquake (OBE) with a maximum ground acceleration of 0.05g and a Safe Shutdown Earthquake (SSE) with a maximum ground acceleration of 0.15g. The maximum vertical earthquake ground acceleration is equal to two-thirds of the maximum horizontal ground acceleration.

Wind loads as given in ANSI A58.1 were used as the design basis for the Unit 4 EDG Building and diesel oil storage facility. The South Florida Building Code wind loads do not govern the design.

The Unit 4 EDG Building and diesel oil storage facility have been designed for the Design Basis Tornado, using the parameters for Region I as given in Regulatory Guide 1.76.

The Unit 4 EDG Building and diesel oil storage facility have been designed for a Design Basis Flood up to elevation +20.0 feet MLW and wave run-up from the east up to elevation +22.0 feet MLW. Refer to Appendix 5G.

Internal missile loads have been considered for the interior design of the Unit 4 EDG Building and diesel oil storage facility, but there is no high-energy piping present within the building. The internally generated missiles considered in the design are the EDG piston assembly, connecting rod, piston pin, fan blade, or an air receiver relief valve.

5.3.4.3.2 LOAD COMBINATIONS

Load combinations presented in Appendix 5A for Class I structures, as supplemented by more recent criteria for Seismic Category I structures listed in Subsection 5.3.4.2 in accordance with NUREG-0800, SRP Section 3.8.4, have been used in the design of the Unit 4 EDG Seismic Category I structures.

In addition, building sliding, overturning and floatation stability have been checked under seismic, hurricane, tornado and flood conditions as specified in NUREG-0800, SRP Section 3.8.5, for foundations, with the result that the minimum factors of safety have been met.

Design and analysis procedures and structural acceptance criteria are presented in Subsections 5.3.4.4 and 5.3.4.5, respectively.

5.3.4.4 DESIGN AND ANALYSIS PROCEDURES

The Unit 4 EDG Building and diesel oil storage facility have been designed to meet the Turkey Point criteria for Class I structures, as supplemented by more recent regulatory criteria for Seismic Category I structures.

Building seismic loadings are based upon an Operating Basis Earthquake (OBE) with a maximum ground acceleration of 0.05g and a Safe Shutdown Earthquake (SSE) with a maximum ground acceleration of 0.15g. The maximum vertical earthquake ground acceleration is equal to two-thirds of the maximum horizontal ground acceleration.

Building seismic dynamic analysis was performed using an acceleration time-history with a ground response spectra which envelopes the spectra provided in Regulatory Guide 1.60. The artificially generated time-history was applied to a lumped mass cantilever model to generate maximum responses and floor response spectra. The computer program utilized for the dynamic analysis is Ebasco Program DYNAMIC 2037. Soil structure interaction has been accounted for in the dynamic model by the use of soil springs which link the time-history input motion with the cantilever model.

Structural dampings used in the dynamic analysis were taken from Regulatory Guide 1.61. Five percent (5%) and ten percent (10%) soil damping values were used for the OBE and SSE, respectively, which are based upon the dynamic properties of the underlying rock and crushed limerock fill. Dynamic properties used are discussed in Subsection 2.9.4.7.

Modal responses were combined in the maximum response dynamic analysis by Ebasco Program DYNAMIC 2037. The method of mode combination is in accordance with Regulatory Guide 1.92.

Orthogonal maximum response components were input into the three dimensional static finite element model. These components were combined using the "square-root-of-the-sum-of-the-squares" (SRSS) method in accordance with Regulatory Guide 1.92 using post-processor computer programs.

Building sliding, overturning and floatation stability have been checked under seismic, hurricane, tornado and flood conditions as specified in NUREG-0800, SRP Section 3.8.5 for foundations. The minimum factors of safety under these conditions as given in this reference have been met.

Reinforced concrete has been designed using the Strength Design Method. Load combinations and structural acceptance criteria are in accordance with NUREG-0800, SRP Section 3.8.4, for Seismic Category I structures. The design criteria meets the requirements of Appendix 5A as a minimum. Other concrete design requirements are as given in ACI 349 as modified by Regulatory Guide 1.142.

Structural steel, including the partial floor, platforms, embedded plates, missile shields and crane supports, has been designed according to the load combinations and structural acceptance criteria of NUREG-0800, SRP Section 3.8.4, and the Eighth Edition of the AISC Manual of Steel Construction. The design criteria meets the requirements of Appendix 5A as a minimum.

Soil properties for the site have been determined by a soil boring program as described in Subsection 2.9.4. The properties as given in this subsection have been used in the building foundation design.

The Unit 4 EDG Building and diesel oil storage facility have been designed for the Design Basis Tornado using the parameters for Region I as given in Regulatory Guide 1.76.

All exterior concrete walls and roofs of the Unit 4 EDG Building have been designed to protect safety-related equipment inside from the effects of tornado-generated missiles. All doors have been provided with missile resistant concrete labyrinths. Radiator, HVAC supply and exhaust openings have been provided with steel grating tornado missile barriers. Safety-related conduit outside the Unit 4 EDG Building, routed from the building to new manholes 700 through 707 and then onto stub-ups near the Unit 3 Turbine Building, from manhole 701 (through manhole 760) to the pull box in the vicinity of the Unit 4 Start-up Transformer, and from the Unit 4 EDG Building to manholes 712 through 717 and from manholes 714 and 715 to existing manholes 308 and 310, are protected by concrete missile shields designed for missile impact. All new manholes have also been designed for tornado missile impact. The covers on manholes and hatches on the roof of the Unit 4 EDG Building also constitute missile barriers. The design of exterior walls for missile loads takes into account embedded items such as boxes and conduit. The walls are adequate missile protection for most items, since the reinforcement is not interrupted and the overall wall thickness remains constant. In the remaining cases, steel plate is provided behind boxes to maintain adequate missile protection.

All missile barriers have been designed according to NUREG-0800, SRP Section 3.5.3 for barrier design. This design includes consideration of local penetration as well as the overall effect of impact and precludes spalling on the inside face of concrete barriers.

The Unit 4 EDG Building and diesel oil storage facility has been designed for a Design Basis Flood up to elevation +20.0 feet MLW and for wave run-up to elevation +22.0 feet MLW on the east. Building openings are protected by concrete walls up to the wave run-up elevation, except for the diesel compartment room doors. These doors are located above the flood level and are not exposed to wave run-up from the east. Reinforced concrete walls shield these openings from wave run-up on the north, east and south sides.

Building flooding through the plumbing system from the outside (i.e., through the oil collection sump) is prevented by the locked-shut valve located between the sump and the building. Conduit entering structures have been provided with water seals.

The radiator fan openings are protected from the Design Basis Flood by the concrete fire wall enclosure. Drains at the fire wall have been provided with scuppers which permit water to flow out of but not into the area between the wall and the building.

The Unit 4 EDG Building and diesel oil storage facility is designed for the hurricane wind loads as given in ANSI A58.1. The South Florida Building Code loadings did not govern the design.

The new Safety Related diesel oil transfer piping embedded in the Unit 4 EDG Building basemat is classified Seismic Category I in accordance with Regulatory Guide 1.29 and is designed to ASME Section III, Class 3 requirements. This criteria is more stringent than Appendix 5A criteria (i.e., ANSI B31.1).

The emergency diesel generators have been provided with an overspeed shutdown system and pressure systems contain relief valves, thus making the possibility of internally generated missiles remote. For conservatism, however, the interior walls which separate redundant equipment have been designed for the following internally generated missiles:

1. Piston Assembly
2. Connecting Rod
3. Piston Pin
4. Fan Blade
5. Air Receiver Relief Valve

Ductbanks, manholes, and handholes which are required to protect Safety Related conduit, are designated Seismic Category I structures in accordance with Regulatory Guide 1.29.

New ductbanks, manholes and handholes for safety related cables have been designed for the design basis events for Class I structures employing the loads and load combinations given in the Appendix 5A as a minimum. In addition, more recent regulatory criteria for the design of Seismic Category I structures have been used. The criteria employed are as follows. Seismic loadings (0.05g and 0.15g) have been applied. The tornado missile spectrum provided by SRP Section 3.5.3 has been used in lieu of that provided by Appendix 5E, since the former is more severe. Load combinations and structural acceptance criteria have been taken according to SRP Section 3.8.4 and ACI 349. These design criteria are at least as stringent as those required by Appendix 5A.

Safety Related ductbanks have been designed for the most limiting steam generator transport loadings, when located on the heavy haul route. Reinforced concrete shields protect the buried conduits from the surcharges where required. Ductbanks in other areas have been designed for H-20 truck loads, and normal soil surcharge.

Manholes and ductbanks in the laydown area between Unit 2 and Unit 3 have been evaluated for the turbine rotor transporter loads and were found capable of withstanding these loads. The manholes and ductbanks in the laydown area are only the "B" train. Therefore, a heavy load drop in this area is not a concern, since the other train is adequately separated.

Penetrations into existing manholes have been made without cutting reinforcing steel, such that, structural integrity is not adversely impacted.

5.3.4.5 STRUCTURAL ACCEPTANCE CRITERIA

The basis for the structural acceptance criteria is specified in Appendix 5A and the ACI 318, ACI 349 and AISC Codes as discussed in Subsection 5.3.4.

Reinforced concrete structures have been designed in accordance with the ACI 318 Strength Design Method. Structural acceptance criteria are in accordance with NUREG-0800, SRP 3.8.4 for Seismic Category I structures. Other concrete design requirements are as given in ACI 349 as modified by Regulatory Guide 1.142.

Structural and miscellaneous steels have been designed in accordance with the structural acceptance criteria of NUREG-0800, SRP Section 3.8.4 and the AISC Manual of Steel Construction.

The Seismic Category I EDG structures are proportioned to remain within the elastic limits under all design loading conditions described in Subsection 5.3.4.3.

Sliding, overturning and floatation stability of the Unit 4 EDG Building structure have been checked under seismic, hurricane, tornado and flood conditions as specified in NUREG-0800, SRP Section 3.8.5 for foundations, and the minimum factors of safety under these conditions as given in this reference have been met.

5.3.4.6 MATERIALS, QUALITY CONTROL AND SPECIAL CONSTRUCTION TECHNIQUES

The primary materials of construction are concrete, reinforcing steel and structural steel (rolled shapes and plates).

Basic quality control procedures are per the applicable effective codes.

No special construction techniques were implemented for the Seismic Category I structures described in this Subsection 5.3.4.

5.3.4.7 TESTING AND IN-SERVICE SURVEILLANCE REQUIREMENTS

Other than normal quality control testing required by applicable codes, no additional testing or in-service surveillance for the Seismic Category I structures described in this Subsection 5.3.4 was implemented.

5.4 PIPE WHIPPING RESTRAINTS

5.4.1 Design Basis

Pipe whipping restraints are designed and located to restrict the movement of ruptured pipes in order to prevent damage to adjacent components as stated herein. Protection from pipe rupture can be provided by installed restraints or physical barriers that are designed and located to prevent damage to adjacent components as stated herein. Protection against pipe rupture may also be in the form of redundancy and separation.

The whipping restraints are provided for the pipes which function continuously during normal operation as well as those which function only after a MHA, all in accordance with the Section 5.4.3. Most lines have been included without consideration to the probability of their failure (example: spray and containment cooler water lines do not carry pressures high enough to cause failure, but they have been included nevertheless). If it can be proven that once a line ruptures, it is incapable of damaging other critical lines (i.e., due to its low pressure service, or the ruptured line being of relatively small size or sufficient physical separation), that line need not be restrained or isolated to protect other critical lines.

Whipping restraints are generally designed for the loads resulting from an instantaneous double ended pipe rupture. "Slot" type failures are also considered in the restraint design where the ruptured pipe could cause subsequent failure of a shutdown or post-MHA cooling system.

Whipping restraints are designed such that they will perform their function and remain within their material elastic limit.

In response to USAEC's requirement for a study of high energy line breaks outside the containment, an analysis was performed to analyze high energy lines outside the containment for pipe failures.* The analysis revealed that breaks may occur on the main steam and feedwater lines. To mitigate the consequences of such a break, whipping restraints with jet impingement protection were installed. The potential for flooding safety related equipment does not exist with main steam and feedwater breaks since they are located outdoors.

* Contained in the James Coughlin of FPL letters to Angelo Giambusso of AEC, dated February 26, 1973, and June 21, 1973.

The requirements for selecting break locations established that breaks be postulated at terminal ends; at any intermediate location between terminal ends based on the longitudinal or circumferential stresses exceeding the prescribed threshold stress values for the combined loadings associated with seismic events and operating loads, or a specific limit for thermal expansion stresses; and , at arbitrary intermediate locations, even if the combined pipe stresses were below the threshold limits. These requirements were initially established post Operating License as a result of a request by the Atomic Emergency Commission (AEC) in 1972. This request was clarified later to provide changes and corrections to the guide entitled " General Information Required for Consideration of the Effect of a Piping System Break Outside Containment," (References 7 and 8). Following these clarifications and corrections, the HELB stress criteria for establishing the intermediate break locations is shown in the equations below.

Intermediate breaks should be postulated to occur at locations in each piping run where:

Stresses derived from loads P, Dw, OBE, T and Th > 0.8 (S_h + S_a)

Stresses derived from loads Th > 0.8 S_a C26

Where

P = Internal Pressure
Dw = Dead Weight
OBE = Operating Basis Earthquake
T = Transient
Th = Thermal Expansion range
S_h = Allowable Stress at Hot Condition
S_a = Allowable Thermal Expansion Stress

By inference obtained from Reference 7 and 8 communications, the allowable stress at hot condition, S_h, was determined in accordance with ASME Boiler and Pressure Vessel Code, Section III, Winter 1972 Addenda, NC-3600 for Class 2 and Class 3 components, respectively; and the allowable thermal expansion stress, S_a, was determined in accordance with ASME Boiler and Pressure Vessel Code, Section III, Winter 1972 Addenda, or per the USA Standard Code for Pressure Piping, ANSI B31.1.0-1967.

For the Main Steam lines outside containment, the two locations, designated as Postulated Intermediate Break Locations 4 and 5, were selected based on highest calculated stress, even though both were below the threshold stress values.

The NRC (in Generic Letter, GL 87-11) provided a relaxation for arbitrary intermediate pipe break requirements. GL 87-11 established that arbitrary locations need not be postulated, provided the threshold stress values are not exceeded. For the Extended Power Uprate condition, the Main Steam lines outside containment were re-analyzed to the pipe rupture criteria described above, including the GL 87-11 provisions. The analysis shows that for Main Steam lines, intermediate pipe breaks need not be postulated since the combined stress and the thermal expansion stresses at all intermediate locations between terminal ends are below the corresponding threshold values. Therefore, postulation of intermediate break locations are no longer applicable and the Main Steam line breaks are restricted to terminal ends only. Although the GL 87-11 provision is generally applicable for the potential elimination of arbitrary intermediate pipe breaks in all high energy lines, the elimination provision contained in GL 87-11 has only been applied to the Main Steam lines outside containment at the time. C26

The NRC documents in their letter of November 28, 1988 (Reference 1) that the leakage detection systems at Turkey Point Units 3 and 4 satisfy the requirements of Generic Letter 84-04, and that the primary loop piping complies with the criteria of general design criteria (GDC)from 10 CFR Part 50, Appendix A. GDC 4 allows the use of plant-specific Leak-Before-Break analysis to eliminate the dynamic effects of postulated pipe ruptures in high energy piping from the design basis of the plant. Plants with NRC approved Leak-Before-Break analysis may remove pipe whip restraints and jet impingement barriers. Turkey Point Units 3 and 4 received NRC approval (Reference 2) for elimination of the dynamic affects of postulated pipe ruptures on the reactor coolant primary loop piping from the design basis of the plant. The Turkey Point analysis for the Leak-Before-Break Methodology is documented in the Westinghouse report WCAP-14237 (Reference 3).

5.4.2 Basic Requirements

(a) The Reactor Coolant System is to be protected from any and all possible sources of damage. Loss of all safety injection, and reactor coolant piping is assumed for the ruptured reactor coolant loop after a MHA. All engineered safeguards piping associated with the given ruptured loop may, therefore, be damaged by that loop's reactor coolant pipe movement, without any loss of engineered safeguards capability.

(b) The first isolation valve or normally locked-closed valve between the reactor coolant system and the system under consideration, are assumed to function properly. Check valves are assumed to be capable of maintaining reactor coolant system integrity if downstream piping is damaged, and isolation valves are assumed to function if either upstream or downstream piping is damaged.

(c) Two low head safety injection lines will be available after a MHA until start of sump recirculation at which time one low head line is adequate. At least two high head injection lines are to be available.

5.4.3 Critical Systems Requiring Protection

The following categories are used in developing the piping protection criteria given in Table 5.4.3-1.

- Category A Lines that must be restrained from damaging the Reactor Coolant System.
- Category B Lines that must be restrained from damaging the containment liner plate.
- Category C Lines that must be protected from damage by ruptured reactor coolant system piping.
- Category D Lines that must be restrained from damaging the secondary system.
- Category E Lines that must be protected from damage by the secondary system.
- Category F Lines that must be protected from damage by or restrained from damaging their parallel redundant lines.

Based on Reference 4, pipe whip restraints on the low head safety injection (LHSI) piping outside of the secondary shield wall have been provided but are not required to satisfy the design requirement of this section.

5.4.4 References

1. NRC Letter from G. E. Edison (NRC) to W. F. Conway (FPL), "Turkey Point Units 3 and 4 - Generic Letter 84-04, Asymmetric LOCA Loads," dated November 28, 1988.

2. NRC Letter from R. P. Croteau (NRC) to J. H. Goldberg (FPL), "Turkey Point Units 3 and 4 - Approval to Utilize Leak-Before-Break Methodology for Reactor Coolant System Piping (TAC Nos. M91495 and M91495)," dated June 23, 1995.

3. Westinghouse WCAP-14237, "Technical Justification for Eliminating Large Primary Loop Pipe Rupture as the Structural Design Basis for the Turkey Point Units 3 and 4 Nuclear Power Plants," dated December 1994.

4. Safety Evaluation JPN-PTN-SENS-89-020, Rev. 0, "Safety Evaluation for Low Head Safety Injection Pipe Whip Restraints."

5. PTN-ENG-SEMS-96-049, Engineering Evaluation for Basis for HELB Inside Containment

6. NRC Generic Letter 87-11, " Relaxation in Arbitrary Intermediate Pipe Rupture Requirements."

7. AEC Letter from A. Giambusso to J. Coughlin, dated December 18, 1972, transmitting, " General Information Required for Consideration of the Effects of a Piping System Break Outside Containment." C26

8. AEC Letter from Karl Kniel to J. Coughlin, dated January 24, 1973, transmitting , "Errata sheet for General Information Required for Consideration of the Effects of a Piping System Break Outside Containment."

TABLE 5.4.3-1

PIPE RUPTURE
PROTECTION CRITERIA

LINES	A	B	C	D	E	F
High Head Safety Injection Lines	x	x	x	x	x	x
Low Head Safety Injection Lines	x	1	x	1		x
Charging Line	x	x		x	x	
Emergency Cooler Lines	x	x	x	x	x	x
Reactor Coolant Letdown Lines	x	x	x	x	x	
Decay Heat Removal Line	x	x		x	x	
Blowdown Line	x		x	x	x	x
Main Steam and Feedwater Lines			x		x	x
Reactor Coolant System Lines	x	x	x	x	x	x
Spray Headers	x	x	x	x	x	x

Note: 1. Pipe whip restraints on the LHSI piping outside the secondary shield wall have been provided but are not required (Reference 4)

5.5 PIPE SUPPORT EXPANSION ANCHORS

5.5.1 DESIGN BASIS

IE Bulletin 79-02 required all Licensees and permit holders for Nuclear Power Plants to review the design and installation procedures for concrete expansion anchors used in pipe support base plates in systems defined as Seismic Category I by Regulator Guide 1.29, "Seismic Design Classification," Revision 1, August 1973, or by the applicable SAR. FPL transmitted a revised final report summarizing previous responses and actions taken in response to IE Bulletin 79-02 to the NRC in Reference 1.

Expansion anchors for Seismic Category I pipe supports have been analyzed to account for baseplate flexibility. The concrete expansion bolts anchoring the plates have a minimum factor of safety between the bolt design load and the bolt ultimate capacity determined from static load tests of four for wedge type anchor bolts and five for shell type anchor bolts. The expansion anchors have been designed to withstand the design load of the piping system, consisting of deadweight, thermal, seismic and dynamic loads.

5.5.2 REFERENCES

1. Letter, L-87-383, C O Woody (FPL) to the U.S. Nuclear Regulatory Commission, dated October 22, 1987.

APPENDIX 5A

SEISMIC CLASSIFICATION & DESIGN BASIS
FOR
STRUCTURES, SYSTEMS AND EQUIPMENT
FOR
TURKEY POINT

5A-1.0 DESIGN BASES OF STRUCTURES, SYSTEMS AND EQUIPMENT

5A-1.1 Design Codes

The design bases for structures at normal operating conditions are governed by the applicable building design codes. The design bases for specific systems and equipment are stated in the appropriate FSAR section. The design bases for the containment structure are contained in Appendix 5B. The basic design criterion for the maximum hypothetical accident and earthquake conditions is that there be no loss of function if that function is related to public safety.

5A-1.2 Design Classification of Structures, Systems and Equipment

Class I structures, systems and equipment are those whose failure could cause uncontrolled release of radioactivity in excess of the established guidelines as prescribed in 10 CFR 50.67, those essential for immediate and long-term operation C26 following a loss-of-coolant accident to either cool the core or reduce the containment pressure, those required to function after a loss of power occurrence or steam line break to permit a controlled NSSS cool-down, or those required for a safe shutdown. Associated with Class I structures, systems and equipment are their supports, enclosures, piping, wiring, controls, power sources and switch-gear. They are designed to withstand the appropriate earthquake loads applied simultaneously with other applicable loads without loss of function. When a system as a whole is referred to as Class I the portions not associated with the loss of function of the system may be designated as Class III as appropriate. There are no components or structures designated as being Class II.

The following are classified as Class I structures, systems and equipment:

1. Reactor Coolant System
   - Reactor vessel
   - Reactor vessel internals
   - RCC assemblies and drive mechanisms
   - Steam generators
   - Reactor Coolant pumps
   - Pressurizer and relief tank
   - All reactor coolant piping, plus any other lines carrying reactor coolant under pressure.

2. Containment System
   - Containment structure
   - Containment penetrations
   - Containment purge valves
   - Equipment, personnel, and emergency hatches
   - All lines penetrating the containment, up to and including the first isolation valves.

3. Main Steam & Feedwater Lines within the Containment

4. Main Steam Outside of the Containment
   - Main steam safety valves
   - Main steam isolation valves (MSIV) and air accumulators
   - Main steam reverse check valves
   - Main steam atmospheric dump valves
   - Main steam piping to MSIVs

5. New Fuel Storage Facilities

6. Auxiliary Feedwater System
   - Auxiliary feedwater pumps and turbine drivers
   - Condensate storage tank
   - Steam, condensate and feedwater lines of auxiliary feedwater system.

7. Emergency Diesel Generators
- Engine, generator, fuel skid
- Fuel day tanks
- Fuel storage tanks
- Fuel transfer pumps
- Air start receivers
- Associated piping

NOTE: Load combinations for Class I structures, as supplemented by more recent criteria for Seismic Category I structures listed in Section 5.3.4.2, were used in the design of the Unit 4 EDG Seismic Category I structures. See Section 5.3.4.3 for specific design criteria.

8.a Containment Polar Crane and Rail Support
The containment polar crane and associated rails are seismically qualified Class I structures in the unloaded configuration. These structures are also seismically qualified in all plant operating modes for a maximum load lift of 1,760 lbs by either hoist of the polar crane.

8.b Cask Handling Crane Support Structure
The cask handling crane support structure will maintain integrity under all seismic loading conditions and is qualified as Class I structure in the cask handling configuration. C26

9. Refueling Water Storage Tanks

10. Emergency Containment Cooling Units C26

11. Intake Cooling Water Systems
- Intake structure and crane supports
- Intake cooling water pumps and motors
- Intake cooling water piping, from pumps to component cooling water heat exchanger inlets
- Basket strainers
- Intake cooling water piping up to the point where the piping enters the ground downstream of the turbine plant cooling water heat exchangers is seismically qualified to ensure the pressure integrity of the intake cooling water system. C26

12. Component Cooling System
- Component cooling heat exchangers
- Component cooling pumps and motors
- Component cooling surge tanks
- Component cooling head tank

13. Spent Fuel Storage Facilities
- Spent fuel pit and racks
- Spent fuel pit cooling water pumps and motors
- Spent fuel pit heat exchangers C26
- Spent fuel pit demineralizer

14. Safety Injection System
- Containment spray pumps and motors
- Residual heat removal pumps and motors (low-head safety injections pumps)
- Residual heat removal heat exchangers
- High-head safety injection pumps and motors
- Containment spray headers
- Accumulator tanks

15. Chemical and Volume Control System
- Charging pumps
- Volume control tank
- Boric acid blender
- Boric acid tanks
- Boric acid transfer pumps
- Boric acid filters
- Regenerative Heat Exchanger

16. Fuel Transfer Tube

17. Post Accident Containment Venting System
- Piping within containment and to at least the second valve outside containment

18. Waste Handling Facilities Building

19. Turbine Plant Cooling Water System
- Turbine plant cooling water piping from the pump discharge to the turbine plant cooling water heat exchanger nozzles is seismically qualified to prevent adverse interaction with the pressure integrity of the intake cooling water system. C26

5A-1.3 Class I Structures, Systems and Equipment Design Requirements

5A-1.3.1 Class I Structure Design Requirements

5A-1.3.1.1 Normal Operation

For loads to be encountered during normal operation, Class I structures are designed in accordance with design methods of accepted standards and codes insofar as they are applicable.

5A-1.3.1.2 Hypothetical Accident, Wind and Earthquake Conditions

The Class I structures are proportioned to maintain elastic behavior when subjected to various combinations of dead loads, accident loads, thermal loads, and wind or seismic loads. The upper limit of elastic behavior is considered to be the yield strength of the effective load-carrying structural materials. The yield strength for steel (including reinforcing steel) is considered to be the minimum as given in the appropriate ASTM Specification. Concrete structures are designed for ductile behavior whenever possible; that is, with steel stress controlling the design. The values for concrete, as given in the ultimate strength design portion of the ACI 318-63 Code, are used in determining "Y", the required yield strength of the material. Limited yielding is allowable provided the deflection is checked to ensure that the affected Class I systems and equipment (except reactor vessel internals under MHA loadings) are not stressed beyond the values given below. The Unit 4 Emergency Diesel Generator structure is designed as described in Section 5.3.4.

The structure design loads are increased by load factors based on the probability and conservatism of the predicted normal design loads.

The Class I structures outside the containment structure satisfy the most severe of the following:

$$Y = 1/\emptyset\ (1.25D + 1.25E)$$
$$Y = 1/\emptyset\ (1.25D + 1.0R)$$
$$Y = 1/\emptyset\ (1.25D + 1.25H + 1.25E)$$
$$Y = 1/\emptyset\ (1.0D + 1.0E')$$

where;

Y = required yield strength of the material.

D = dead load of structure and equipment plus any other permanent loads contributing stress, such as soil or hydrostatic loads. In addition, a portion of "live load" is added when such load is expected to be present when the unit is operating. An allowance is also made for future permanent loads.

R = force or pressure on structure due to rupture of any one pipe.

H = force on structure due to restrained thermal expansion of pipes under operating conditions.

E = design earthquake load.

E' = maximum earthquake load.

W = wind load (to replace E in the above load equations whenever it produces higher stresses then E does).

Ø = 0.90 for reinforced concrete in flexure.

Ø = 0.85 for tension, shear, bond, and anchorage in reinforced concrete.

Ø = 0.75 for spirally reinforced concrete compression members.

Ø = 0.70 for tied compression members.

Ø = 0.90 for fabricated structural steel.

5A-1.3.2 <u>Class I Systems and Equipment Design Requirements</u>

All Class I systems and equipment are designed to the standards of the applicable Code. The loading combinations which are employed in the design of Class I systems and equipment are given in Table 5A-1.

Table 5A-1 also indicates the stress limits which are used in the design of the listed equipment for the various loading combinations.

To perform their function, i.e., allow core shutdown and cooling, the reactor vessel internals must satisfy deformation limits which are more restrictive than the stress limits shown on Table 5A-1. For this reason the reactor vessel internals are treated separately.

5A-1.3.2.1 <u>Piping and Vessels</u>

The reasoning for selection of the load combinations and stress limits given in Table 5A-1 is as follows: For the design earthquake, the nuclear steam supply system is designed to be capable of continued safe operation, i.e., for the combination of normal loads and design earthquake loading. Critical equipment needed for this purpose is required to operate within normal design limits.

In the case of the maximum hypothetical earthquake, it is only necessary to ensure that critical components do not lose their capability to perform their safety function, i.e., shut the unit down and maintain it in a safe condition. This capability is ensured by maintaining the stress limits as shown in Table 5A-1. No rupture of a Class I pipe is caused by the occurrence of the maximum hypothetical earthquake.

Careful design and thorough quality control during manufacture and construction and inspection during unit life, ensures that the independent occurrence of a reactor coolant pipe rupture is extremely remote. Leak-Before-Break (LBB) criteria has been applied to the reactor coolant system piping based on fracture mechanics technology and material toughness. That evaluation, together with the leak detection system, demonstrates that the dynamic effects of postulated primary loop pipe ruptures may be eliminated from the design basis (Reference 5A-2). This Leak-Before-Break evaluation was approved by the NRC for use at Turkey Point (Reference 5A-5). This evaluation has been revised for the period of extended operation, as discussed in Subsection 16.3.8. This evaluation has been revised for the subsequent period of extended operation, as discussed in Subsections 17.3.8.3, 17.3.8.4, and 17.3.8.5. C31

5A-1.3.2.2 <u>Reactor Vessel Internals</u>

5A-1.3.2.2.1 <u>Reactor Vessel Internals Design Criteria</u>

The internals and core are designed for normal operating conditions and subjected to load of mechanical, hydraulic, and thermal origin. The response of the structure under the design earthquake is included in this category.

The stress criteria established in the ASME Boiler and Pressure Vessel Code, Section III, have been adopted as a guide for the design of the internals and core with the exception of those fabrication techniques and materials which are not covered by the Code. Earthquake stresses are combined in the most conservative way and are considered primary stresses.

The members are designed under the basic principles of: (1) maintaining distortions within acceptable limits; (2) keeping the stress levels within acceptable limits; and (3) prevention of fatigue failures.

5A-1.3.2.2.2 Reactor Vessel Internals Design Analysis

A mathematical model of the reactor pressure vessel using three-dimensional nonlinear finite elements was used to evaluate the reactor internals as part of the thermal uprate project. The model consists of three submodels interconnected by nonlinear impact elements. The first submodel consists of the reactor vessel, shell, and associated components. The second submodel consists of the reactor core barrel, thermal shield, lower support plate, tie plates, and secondary core support components. The third submodel represents the upper support plate, guide tubes, support columns, upper and lower core plates, and the fuel.

Loading applied to the analytical model includes: (a) deadweight of the components and contents; (b) pressure differentials due to coolant flow; (c) seismic excitation; (d) loss of coolant accident loads; (e) vibrational loading; (f) thermal expansion; and (g) preloads on certain components.

Global element matrices and arrays are assembled into the global structural matrices and arrays and used for dynamic solution of the differential equation of motion for the structure:

$$[M]\{\ddot{U}\} + [D]\{U\} + [K]\{U\} = F$$

All resulting stresses and deflections are less than the respective criteria. Fatigue usage factors are in accordance with ASME acceptance limits (Reference 5A-1).

5A-1.3.3 Class I Structures, Systems and Equipment Seismic Loading (Seismic Loads E and E')

AEC Publication TID 7024, "Nuclear Reactors and Earthquakes," as amplified in this Appendix is used as the basic design guide for seismic analysis.

Seismic loading on structures, systems and equipment is determined by realistic evaluation of dynamic properties and the accelerations from the attached acceleration spectrum curves. These spectrum curves are corrected for the design ground accelerations. Damping factors are listed in the table below.

Seismic forces are combined by absolute summation of the vertical and highest horizontal direction. The vertical component of acceleration at any level is taken as two-thirds of the horizontal ground acceleration.

DAMPING FACTORS FOR VARIOUS TYPES OF CONSTRUCTION

STRUCTURAL COMPONENT	% CRITICAL DAMPING	
	Design Earthquake (E) (0.05g Ground Surface Acceleration)	Maximum Earthquake (E') (0.15g Ground Surface Acceleration)
Welded Steel Plate Assemblies	1	1
Welded Steel Framed Structures	2	2
Bolted Steel Framed Structures	2	2
Concrete Equipment Supports on Another Structure	2	2
Prestressed Concrete Containment Structure	2	5
Soil	5	10
Prestressed Containment Including Interior Concrete and Soil Composite	3.5	7.5
Reinforced Concrete Frames and Buildings	3	5
Composite with Soil	5	7.5
Steel Piping	0.5	0.5

5A-1.3.4 Class I Structures, Systems and Equipment Wind Loading (Wind Load W)

The wind loads are determined from the fastest mile of wind for a 100-year occurrence as shown in Figure 1(b) of Reference 5A-4. This is 122 mph at the Turkey Point site. The Class I structures are designed, however, to withstand a wind velocity at 145 mph.

The forces due to the wind are calculated in accordance with methods described in Reference 5A-4. Applicable pressure and shape coefficients are used. There is no variation with height or gust factor.

5A-1.3.5 Class I Structures, Systems and Equipment Tornado Wind Loading

Class I structures are designed to resist the effects of a tornado. Design loadings due to tornado winds used in the design of tornado resistant structures are as follows, the loads to be applied simultaneously:

1. Differential pressure between inside and outside of enclosed areas - 1.5 psi (bursting).

2. External forces resulting from a tornado wind velocity of 225 mph.

3. Missiles as defined in Appendix 5E.

The forces resulting from a tornado are combined with dead loads only. Dead loads include piping and all other permanently attached or located items. There will be sufficient time after sighting a tornado to remove significant live loads such as loads on cranes.

When considering tornado wind loading, allowable stresses are limited to yield strength for structural steel and reinforced concrete. Local crushing of concrete is permitted at the missile impact zone. In addition, all Class I structures are reviewed to assure no loss of function for tornado wind of 337 MPH combined with a pressure differential of 2.25 psi.

5A-1.4 Class III Structures, Systems and Equipment

5A-1.4.1 Design Requirements

Class III Systems and equipment including pipe are generally not designed to withstand any seismic loads. However, for the “Generic Letter 87-0/2Unresolved Safety Issue (USI) A-46” effort, the plant was evaluated to review the seismic adequacy of certain Turkey Point equipment, including the potential interaction between Class III and Class I structures, systems and components (Reference 5A-6). Modifications were made to resolve seismic concerns identified by the review. Subsequent to the Generic Letter 87-02/USI A-46 seismic review effort, Class III structures, systems and equipment in the power block are now reviewed for earthquake loads if the potential for interaction with Safety Related structures, systems and components exists.

The wind loads used for design prior to 1994 were as per South Florida Building Code which has a basic design pressure of 37 psf. Alternatively, for newer structures, wind loads are as required by the edition of the South Florida Building Code applicable at the time of design. Shape Factors are applied in accordance with Reference 5A-4, or as required by the South Florida Building Code applicable at the time of design. No tornado loads are considered.

5A-1.4.2 Turkey Point Fossil Unit 1 Chimney Design Requirements

The Fossil Unit 1 chimney, located directly north of Unit 3, does not perform any safety related functions, or directly protect safety related equipment. However, failure of this structure has the potential of adversely affecting safety related systems. Accordingly, this structure has been designed to not fail and cause an adverse interaction with any safety related systems, when subjected to the Class I seismic loads (0.15 g) and wind loads (145 mph hurricane and 225 mph tornado) described in Sections 5A-1.3.4 and 5A-1.3.5 of this appendix. C28

5A-1.5 Miscellaneous Loads for Structures, Systems and Equipment

The units are designed for an outdoor temperature range of +30°F to +95°F. No ice or snow loads are considered in the design of the various structures and equipment.

External flood protection is described in Appendix 5G.

5A-2.0 METHOD OF SEISMIC ANALYSIS

5A-2.1 Structures

The methods for seismic analysis of the containment structures described in Section 5.1.3.2.

5A-2.2 Response Spectra

Response spectra curves for floors at grade and for the containment basemat were developed based on the El Centro, California, earthquake. These curves are shown in Figures 5A-1 for the design basis earthquake event (E), and Figure 5A-2 for the maximum earthquake event (E'). For class I piping, floor response spectra for the connecting points are developed. Additionally, response spectra curves are also generated for the control building. The analysis methodology is similar to the technique described in Section
5.1.3.2(b). (Reference 5A-3)

5A-2.3 Seismic Class I Piping Analysis

Seismic Class I piping systems are typically analyzed as mathematical models consisting of lumped masses connected by elastic members. The distance from the pipe axis to the center of gravity of the valve and operator is considered, with the mass of the valve and operator, for all motor, air, or gear operated valves. When necessary for the integrity of the piping, valve, or operation, the valve structure is externally supported. The stiffness matrix for the pipe is developed to include the effects of torsional, bending, shear and axial deformations as well as change in flexibility due to curved members and internal pressure. Flexibility factors are calculated in accordance with USAS B31.1. System natural frequencies and mode shapes for all significant modes of vibration are then determined using equations of motion, and spectral accelerations as determined from the response spectra applied.

The following equations are successively used to determine the response for each mode, maximum displacement for each mode, and the total displacement for each mass point:

(1) $$Y_n(\max) = \frac{R_n Sa_n D}{M_n \omega_n^2}$$

(2) $$V_{in} = \varphi_{in} Y_n(\max)$$

(3) $$V_i = \sqrt{\sum V_{in}^{\ 2}}$$

where:

$Y_n(\max)$ = response of the n^{th} mode

R_n = participation factor for the n^{th} mode = $\Sigma M_i \phi_{in}$

M_i = mass i

ϕ_{in} = mode shape i for n^{th} mode

Sa_n = spectral acceleration for the n^{th} mode

D = earthquake direction matrix

M_n = generalized mass matrix for the n^{th} mode = $\Sigma M_i \phi^2_{in}$

ω_n = angular frequency of the n^{th} mode

V_{in} = maximum displacement of mass i for mode n

V_i = maximum displacement of mass i due to all modes calculated

The inertial forces for each direction of earthquake for each mode are then determined from:

$$Q_n = KV$$

where;

Q_n = inertia force matrix for mode n
V = displacement matrix corresponding to Q_n
K = stiffness matrix

Each mode's contribution to the total displacements, internal forces, moments and reactions in the pipe can be determined from standard structural analysis methods using the inertia forces for each mode as an external loading condition. The total combined results are obtained by taking the square root of the sum of the squares of each parameter under consideration, in a manner similar to that done for displacements.

A representative number of critical piping runs have been analyzed by this method. Balance of the pipe runs have been evaluated by:

(i) Closeness of similarity to the runs fully analyzed,

(ii) Simplicity of layout lending to a visual examination for location of seismic restraints to remove the fundamental frequency away from the resonance range, and

(iii) Static analysis based on a uniform static load equal to the peak of the pertinent response spectrum curve.

5A-3.0 METHOD OF SEISMIC ANALYSIS AND RESULTS FOR REACTOR COOLANT LOOP

The reactor coolant loop (RCL), which consists of the reactor vessel (RV), steam generator (SG), reactor coolant pump (RCP), the pipe connecting these components, and the large component supports, has been analyzed for seismic loads. The components and piping are modeled as a system of lumped masses connected by springs whose values are computed from elastic properties that are input. A simplified support model was arrived at by representing the structural support system as equivalent springs rather than as member beams and columns.

The analysis was performed by using a proprietary computer code called WESTDYN. The code uses as input, system geometry, inertia values, member sectional properties, elastic characteristics, support and restraint characteristics, and the appropriate seismic floor response spectrum for 0.5% critical damping. The floor response spectrum curves were generated at the appropriate support locations of the equipment by a time history technique described in Section 5.1.3(b). Both horizontal and vertical components of the seismic response spectrum are applied simultaneously. Two directions, namely X and Z axes, were chosen for application of the horizontal component of the seismic response spectrum. The results of the two cases were combined to determine the most severe loading condition.

With this input data, the overall stiffness matrix [K] of the three dimensional piping system is generated (including translational and rotational stiffnesses). Zero rows and columns representing restraints are deleted, and the stiffness matrix is inverted to give the flexibility matrix [F] of the system.

$$[F] = [K]^{-1}$$

A product matrix is formed by the multiplication of the flexibility and mass matrices. This product matrix forms the dynamic matrix, [D], from which the modal matrix is computed.

$$[D] = [F]\,[M]$$

The eigenvalues and eigenvectors representing the frequency and associated mode shape for each mode are generated using a modified Jacobi method.

$$(\omega^2[M] - [K])\,\{X\} = 0$$

From this information, the modal participation factor is combined with the mode shapes and the appropriate seismic response spectrum values to give the structural response for each mode. Then the forces, moments, deflections, rotations, constraint reactions, and stresses are calculated for each significant mode. The maximum response of the system is obtained by combining the modal contributions using the root mean square method.

The restraints, supports, and other constraints assumed for input into the seismic computer model are given below (see Figure 5A-4 for axes orientation.)

Reactor Vessel	The RV is rigid.
Steam Generator	The SG at the upper support point is permitted to translate along and rotate about the X, Y, and Z axes, but translations along X and Z are resisted by the springs representing the upper support. The SG at the lower support point is permitted to translate along and rotate about the X, Y, and Z axes, but all movements are resisted by springs representing the lower supports stiffness.
Reactor Coolant Pump	The RCP is permitted to translate alone and rotate about the X, Y and Z axes, but all movements are resisted by springs representing the supports stiffness.

A summary of maximum pipe stresses is given in Table 5A-3.

5A-4.0 METHOD OF SEISMIC ANALYSIS FOR MISCELLANEOUS COMPONENTS

Electrical cable trays and D.C. battery racks have been checked for `g' loading obtained from the spectrum curves of the supporting floors. Motor Control Centers and Load Centers have been shaker-table tested to demonstrate no-loss-of-function capability under the maximum hypothetical earthquake. For additional information on instrumentation, see page B-37 in response to Request No. 7.3.

Mechanical and electrical equipment has been purchased under specifications that include a description of the seismic design criteria for the plant. Hydrodynamic analysis of the Refueling Water Storage Tank has been performed using the methods of chapter 6 of the U.S. Atomic Energy Commission - TID 7024.

Various tanks, switchgear cabinets and motor control centers are retrofitted to meet seismic anchorage requirements as part of USI A-46 efforts. (Reference 5A-6).

5A-5.0 SEISMIC INSTRUMENTATION

The requirements of AEC Safety Guide 12 and subsequent Regulatory Guide 1.12, "Instrumentation for Earthquakes" were developed after issuance of the C23 construction permit for Turkey Point 3 and 4. Nevertheless, a seismograph was installed and a comprehensive seismic instrumentation program has been provided to record any seismic disturbance at the site.

A three component seismograph is installed in the Unit 3 electrical penetration room south for recording vibrations caused by strong local earthquakes. A Triaxial Motion Accelerograph is utilized. C24

These components do not perform a safety function. Nevertheless, they have been seismically analyzed and meet the intent of Regulatory Guide 1.12.

5A-6.0 REFERENCES

5A-1 Turkey Point Units 3 and 4 Fuel Transition to 15x15 Upgrade Reload Transition Safety Report, April 2010.

5A-2 Westinghouse WCAP-14237, Technical Justification for Eliminating Large Primary Loop Pipe Rupture as the Structural Design Basis for the Turkey Point Units 3 and 4 Nuclear Power Plants," dated December 1994.

5A-3 Shock and Vibration Handbook, edited by Harris and Crede, Volume 3 Chapter 50: "Vibration of Structures Induced by Seismic Waves," by George W. Housner.

5A-4 ASCE Paper No. 3269, "Wind Forces on Structures." C26

5A-5 NRC letter, from R. P. Croteau (NRC) to J. H. Goldberg (FPL),"Turkey Point Units 3 and 4 - Approval to Utilize Leak-Before-Break Methodology for Reactor Coolant System Piping (TAC Nos. M91494 and M91495)," dated June 23, 1995.

5A-6 "Plant Specific Seismic Adequacy Evaluation of Turkey Point Units 3 and 4 to Resolve Unresolved Safety Issue (USI) A-46 and Generic Letter (GL)87-02", Stevenson & Associates, April 30, 1993 (number 90C1585D).

5A-7 Letter, J.R. Bensen to W.H. Rodgers/J.R. Bensen and C.D Miller W/A Turkey Point Units 3 & 4 Seismic Instrumentation 5610-C-36, dated November 7, 1972.

5A-8 Westinghouse Technical Report WCAP-17152-P Rev.0, "Turkey Point Units 3 & 4 Extended Power Uprate Engineering Report", August 2012. C26

TABLE 5A-1

LOADING COMBINATIONS AND STRESS LIMITS

LOADING COMBINATIONS	VESSELS REACTOR COOLANT SYSTEMS	PIPING REACTOR COOLANT SYSTEM	PIPING OTHER CLASS 1 PIPING (3)
Normal Loads	$P_m \leq S_m$ P_m (or P_L) + $P_B \leq 1.5\ S_m$	$P_m \leq S$ $P_L + P_B \leq S$	$\sigma p + \sigma g \leq S$
Normal + Design Earthquake Loads	$P_m \leq S_m$ P_m (or P_L) + $P_B \leq 1.5\ S_m$	$P_m \leq 1.2S$ $P_L + P_B \leq 1.2\ S$	$\sigma p + \sigma g + \sigma sd \leq 1.2S$
Normal + Maximum Potential Earthquake Loads	$P_m \leq 1.2\ S_m$ P_m (or P_L) + $P_B \leq 1.2\ (1.5\ S_m)$	$P_m \leq 1.2\ S$ $P_L + P_B \leq 1.2\ S$	(1) $\sigma p + \sigma g + \sigma sm \leq Sy$
Normal + Pipe Rupture Loads	$P_m \leq 1.2\ S_m$ P_m (or P_L) + $P_B \leq 1.2$ (1.5 S_m or 2004 ASME Code, Section III, Appendix F limits)	Not Applicable	Not applicable - See Pipe Restraint Criteria.
Normal + Maximum Potential Earthquake Pipe Rupture Loads (2)	Not Applicable	$P_m \leq 1.8\ Sy$ $P_L + P_B \leq 1.8\ Sy$	Not Applicable

Where:
- P_m = primary general membrane stress; or stress intensity
- P_L = primary local membrane stress; or stress intensity
- P_B = primary bending stress; or stress intensity
- S_m = stress intensity value from ASME B & PV Code, Section III
- S = allowable stress from USAS B31.1 Code for Pressure Piping
- σp = longitudinal pressure stress
- σg = gravity-caused stress
- σsd = seismic stress due to design earthquake
- σsm = seismic stress due to maximum potential earthquake
- Sy = Minimum yield strength at operating temperature

Notes:

(1) - This equation satisfies no loss of function criteria.

(2) - Earthquake and pipe rupture are combine by the square-root-sum-of-the squares method. Sy may be taken from the certified material test report for the piping.

(3) - The thermal stratification analysis of the pressurizer surge line uses fatigue stress limits from the 1986 ASME Code.

C26

C26

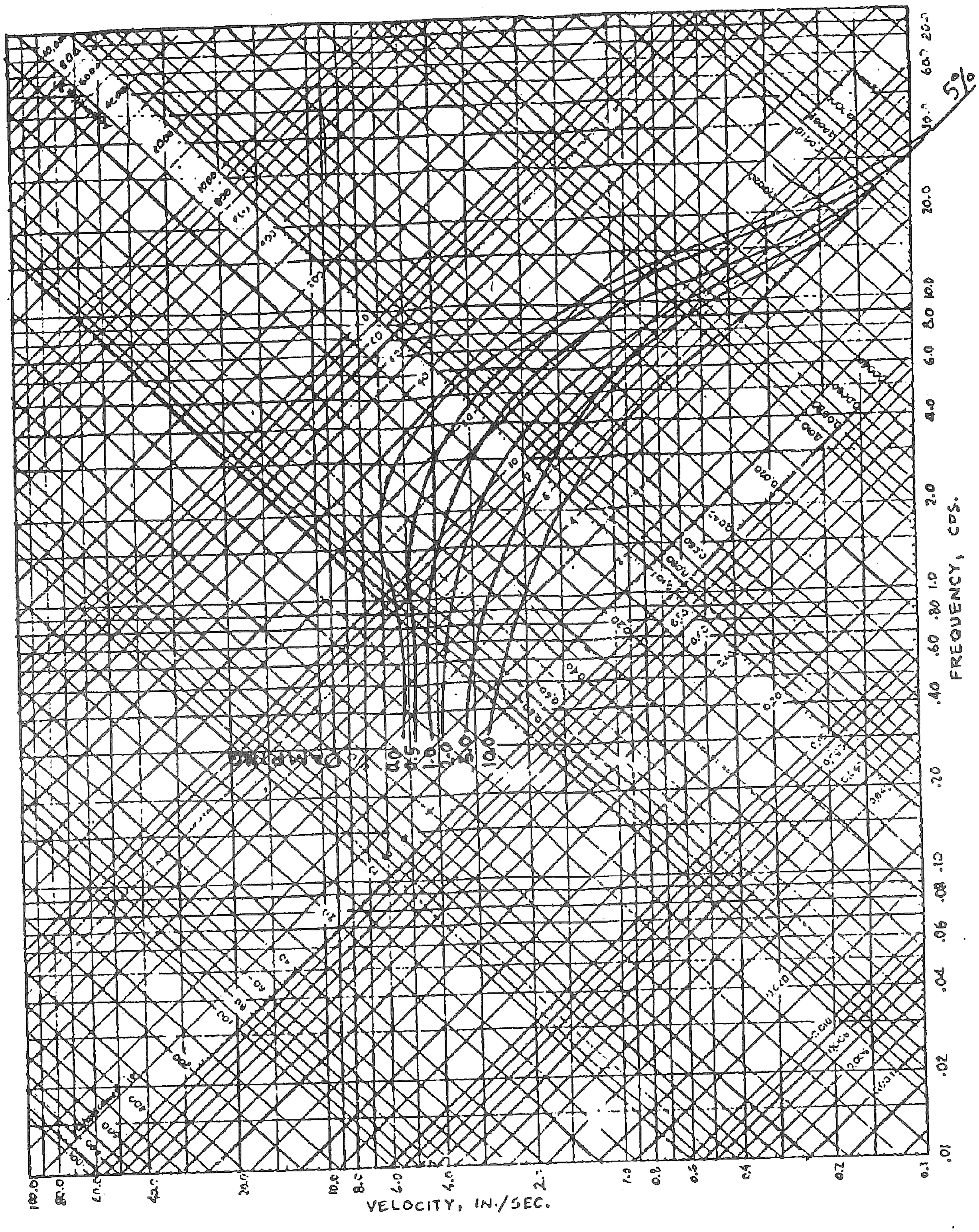
TABLE 5A-2 INTENTIONALLY DELETED

TABLE 5A-3

MAXIMUM STRESSES EXPECTED IN REACTOR COOLANT SYSTEM PIPING
DUE TO THE OPERATING (.05g) EARTHQUAKE

Location	Maximum Stress (psi)
Reactor Coolant Pump Inlet	4085
Reactor Coolant Pump Outlet	3616
10 Inch Accumulator Line	3201
Steam Generator Outlet	2274
Reactor Vessel Inlet	1289
Reactor Vessel Outlet	182
Pressurizer Surge Line Connection	78
Steam Generator Inlet	71

Maximum Allowable Seismic Stress = 13,125 psi (This value is the result, after deadweight and pressure stresses have been subtracted from 1.2 times the material allowable stress.)



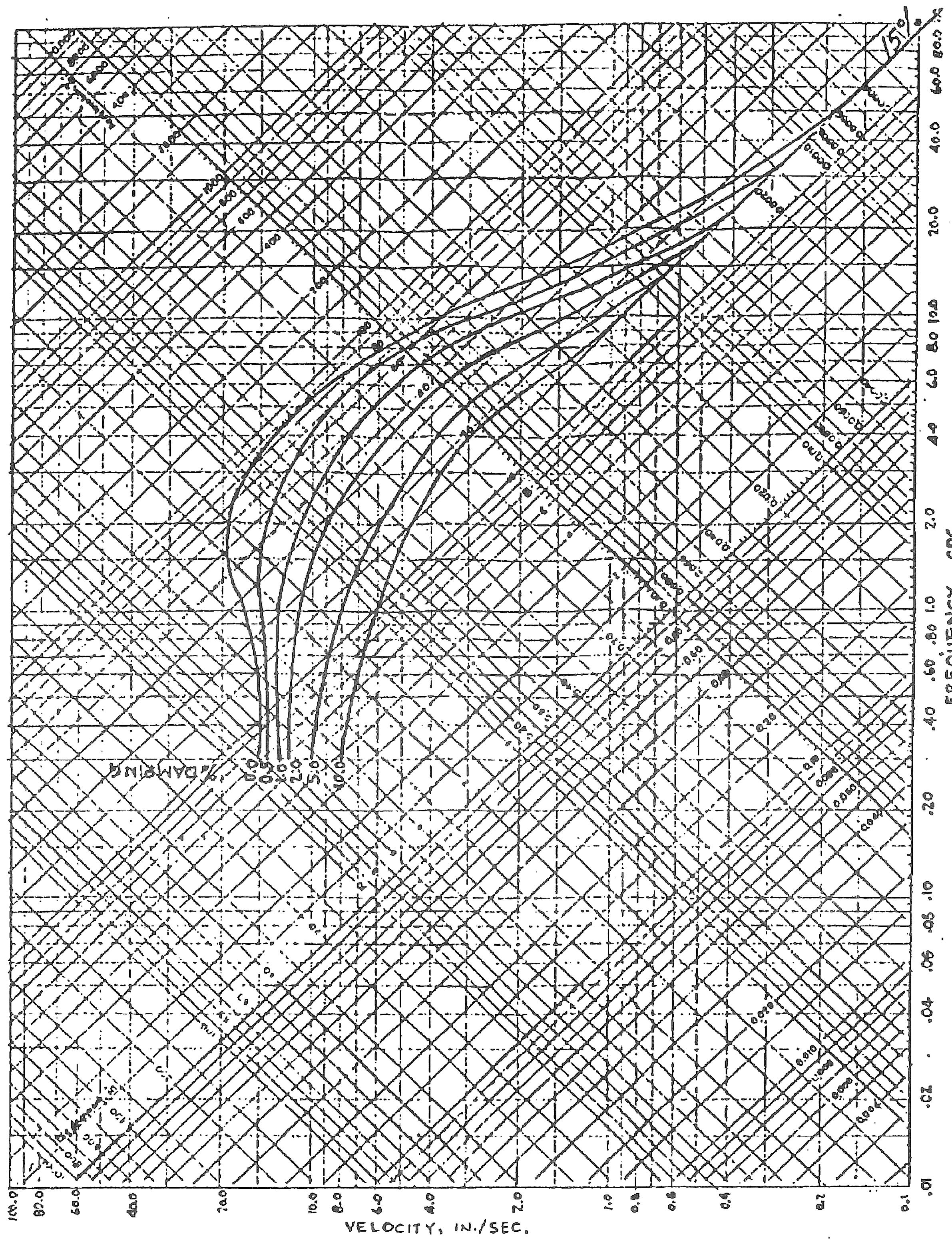
FLORIDA POWER & LIGHT COMPANY

TURKEY POINT PLANT

DAMPED RESPONSE SPECTRA

5% ACCELERATION

FIGURE 5A-1



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT

DAMPED RESPONSE SPECTRA
15% ACCELERATION
FIGURE 5A-2

FINAL SAFETY ANALYSIS REPORT

FIGURE 5A-3 HAS BEEN DELETED

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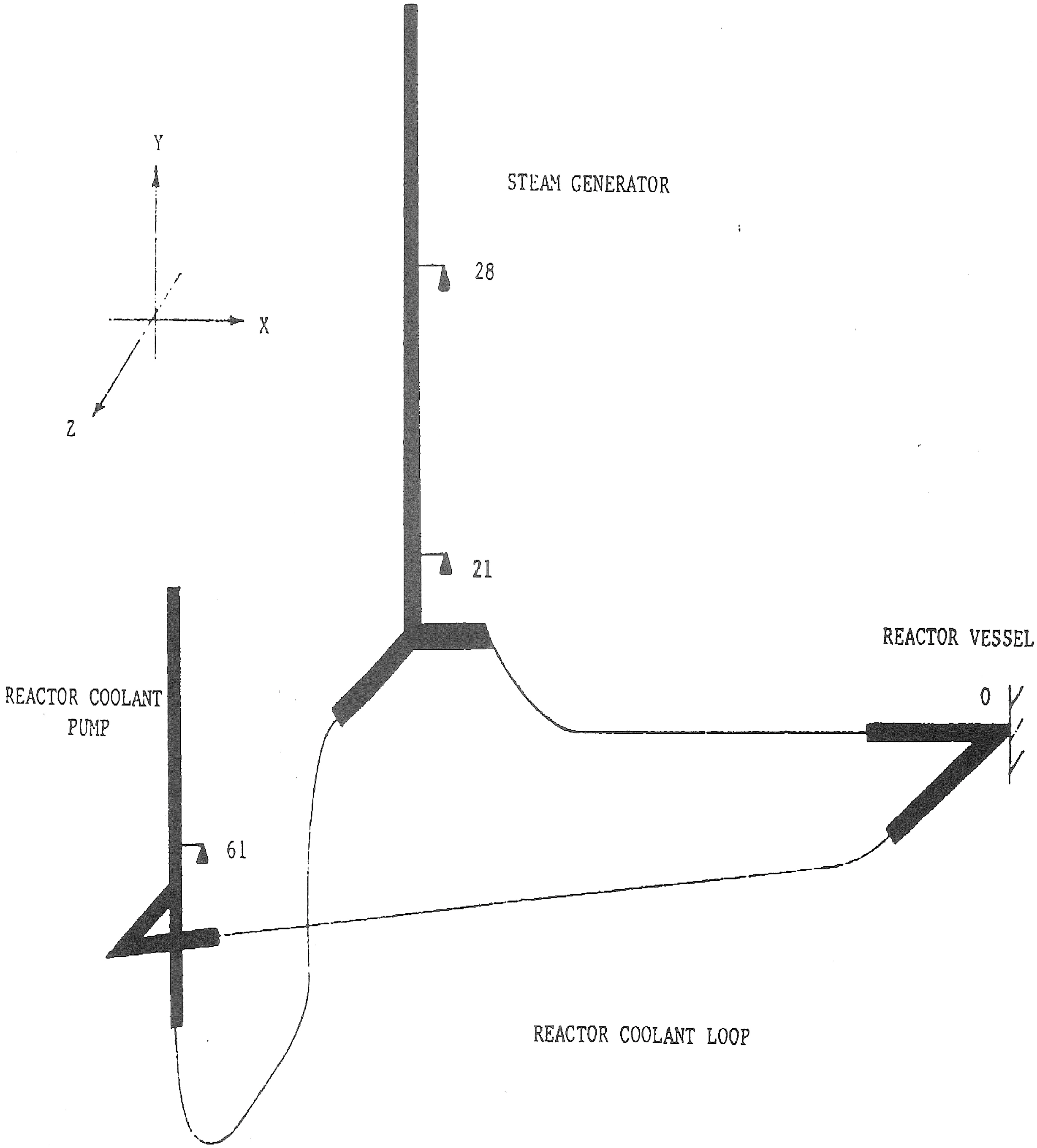
TURKEY POINT PLANT UNITS 3 & 4

MATHEMATICAL MODEL FOR REACTOR VESSEL INTERNAL ANALYSIS (HORIZONTAL EXCITATION)

FIGURE 5A-3



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MODEL FOR SEISMIC ANALYSIS
OF
REACTOR COOLANT LOOP
FIGURE 5A-4

APPENDIX 5B
CONTAINMENT STRUCTURE DESIGN CRITERIA

B.1 Integrity of the containment structure under extraordinary circumstances and its performance at various loading stages are the main considerations in establishing the containment structural design criteria:

The two basic criteria are:

a) The integrity of the liner plate shall be maintained under all loading conditions, and,

b) The structure shall have a low-strain elastic response such that its behavior will be predictable under all design loadings. The strength of the containment structure at working stresses and over-all yielding is compared with various loading combinations to ensure safety. The containment structure is examined with respect to strength, the nature and the magnitude of cracking, the magnitude of deformation, and the extent of corrosion to ensure proper performance. The structure is designed to meet the performance and strength requirements under the following conditions:

   a) Prior to prestressing
   b) At transfer of prestress
   c) Under sustained prestress
   d) At design loads
   e) At yield loads

   Deviations in allowable stresses for the design loading conditions in the working stress method are permitted if the yield capacity criteria are fully satisfied. All design is in accordance with the ACI Code 318-63 unless otherwise stated.

   No special design bases are required for the design and checking of the base slab. It develops primarily bending rather than membrane stresses. This condition is covered by ACT-318-63. The loads and stresses in the cylinder and dome are determined as described below.

B.1.1 Design Method

The structure is analyzed using a finite element computer program for individual and various combinations of loading cases of dead load, live load, prestress, temperature and pressure loads. The computer output includes direct stresses, shear stresses, principal stresses, and displacements of each nodal point.

Stress plots which show the total stresses from appropriate combinations of loading cases are made, and areas of high stresses are identified. The modulus of elasticity is modified as needed to account for the nonlinear stress-strain relationship at high compression. Stresses are then recomputed if there are sufficient areas which require attention.

In order to consider creep deformation, the modulus of elasticity of concrete under sustained loads, such as dead load and prestress, is differentiated from the modulus of elasticity of concrete under instantaneous loads such as internal pressure and earthquake loads.

The forces and shears are added over the cross-section and the total moment, axial force, and shear are determined. From these values, the straight-line elastic stresses are computed and compared with the allowable values. The ACI-318-63 design methods and allowable stresses are used for concrete and prestressed and non-prestressed reinforcing steel except as noted in these criteria.

B.1.2 Loads prior to prestressing

Under this condition the structure is designed as a conventionally reinforced concrete structure. It is designed for dead load, live loads (including construction loads), and wind loads.

Allowable stresses are according at ACI 318-63.

B.1.3 Loads at transfer of prestress

The containment structure is checked for prestress loads and the stresses compared with those allowed by ACI 318-63 with the following exceptions: ACI 318-63, chapter 26, allows a concrete compressive stress of $0.60f'_{ci}$ at initial transfer. In order to limit creep deformations, the membrane compression stresses are limited to $0.30f'_{ci}$, whereas in combination with flexural compression the maximum allowable stress will be limited to $0.60'_{ci}$ as per ACI 318-63.

For local stress concentrations with nonlinear stress distribution as predicted by the finite element analysis, $0.75f'_{ci}$ is permitted when local reinforcing is used to distribute and control such localized strains. These high local stresses, though present in every structure, are seldom identified because of simplifications made in the design analysis. these high stresses are allowed because they occur only in a very small portion of the cross-section, and are confined by material at lower stress and would have to be considerably greater than the values allowed before significant local plastic yielding could occur. Bonded reinforcing is added to distribute and control these local strains.

Membrane tension and flexural tension are permitted provided they do not jeopardize the integrity of the liner plate. Membrane tension is permitted to occur during the post tensioning sequence, but will be limited to $1.0\ f'_{ci}$. When there is flexural tension, but no membrane tension, the section is designed in accordance with section 2605(a) of the ACI 318-63. The stress in the liner plate due to combined membrane tension and flexural tension, is limited to $0.5\ f_y$.

Shear criteria are in accordance with ACI 318-63 Code, Chapter 26, as modified by the equations shown in paragraph B.1.6 using a load factor of 1.5 for shear loads.

B.1.4 <u>Loads under sustained prestress</u>

The conditions for design and allowable stresses for this case are the same as above except that the allowable tensile stress in non- prestressed reinforcing is limited to 0.5 f_y. ACI 318-63 limits the concrete compression to $0.45f'_c$ for a sustained prestress load. Values of $0.30f'_c$ and 0.60 f'_c are used as described above, which bracket the ACI allowable value. However, with these same limits for the concrete stress at transfer of prestress, the stresses under sustained load are reduced due to creep.

B.1.5 <u>At Design Loads</u>

The following loading cases are used in basic "working stress" design of the containment structure:

(a) $D+F+L+T_o$
(b) $D+F+L+F+T_A+E$ (or W)
(c) $D+F+L+P'$

Where:

D= Dead Load
L= Appropriate Live Load
F= Appropriate Prestressing Load
P= Pressure Load (varies with time from design pressure to zero pressure)
T_o= Thermal Load due to the operating temperature
T_A= Thermal Load due to the temperature corresponding to a pressure P.
P'= Test Pressure = 1.15P
W= Wind Load
E= Design Earthquake Load

Sufficient prestressing is provided in the cylinder and dome portions of the containment structure to eliminate membrane tensile cracking under design loads. Flexural tensile cracking is permitted but is controlled by bonded reinforcing steel.

Under the design loads the same performance limits stated in B.1.3 apply with the following exceptions:

(a) Membrane compression below 100 psi is neglected, and a cracked section is assumed in the computations for flexural bonded reinforcing steel. The allowable tensile stress in bonded reinforcing steel is 0.5 f_y.

(b) When the maximum flexural tensile stress does not exceed 6 $\sqrt{f'_c}$ and the extent of the tension zone is not more than 1/3 the depth of the section, bonded reinforcing steel is provided to carry the entire tension in the tension block. Otherwise, the bonded reinforcing steel is designed assuming a cracked section. When the bending moment tension is additive to the thermal tension, the allowable tensile stress in the bonded reinforcing steel is $0.5f_y$ minus the stress in reinforcing due to the thermal gradient determined in accordance with the method of ACI-505.

(c) Shear and diagonal tension in the structure are considered in two parts: with membrane principal tension, and with flexural principal tension. Since sufficient prestressing is used to eliminate membrane tensile stress, membrane principal tension is not critical at design loads. Membrane principal tension due to combined membrane tension and membrane shear is considered under B.1.6.

Flexural principal tension is the tension associated with bending in planes perpendicular to the surface of the shell, and shear stress normal to the shell (radial shear stress). The present ACI 318-63 provisions of Chapter 26 for shear are adequate for design purposes with proper modifications as discussed under B.1.6, using a load factor of 1.5 for shear loads.

Crack control in the concrete is accomplished by adhering to the ACI-ASCE Code Committee standards for the use of reinforcing steel. These criteria are based upon a recommendation of the Prestressed Concrete Institute, and are as follows:

0.25 percent reinforcing shall be provided at the tension face for small members
0.20 percent for medium size members
0.15 percent for large members

A minimum of 0.15 percent bonded steel reinforcing is provided in two perpendicular directions on the exterior faces of the wall and dome for proper crack control.

The liner plate is anchored in the inside faces of the shell. Since, in general, there is no tensile stress due to temperature on the inside faces, bonded reinforcing steel is not necessary at the inside face.

B.1.6 Loads necessary to cause structural yielding

The structure is checked for the factored loads and load combinations given below.

The load factors are the ratio by which loads will be multiplied for design purposes to assure that the load/deformation behavior of the structure is one of elastic, low-strain behavior. The load factor approach is being 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. It also places minimum emphasis on the fixed gravity loads and maximum emphasis on accident and earthquake or wind loads.

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

(a) $Y = 1/\emptyset\ (1.05D+1.5P+1.0T_A +1.0F)$
(b) $Y = 1/\emptyset\ (1.05D+1.25P+1.0T_A+1.25E+1.0F)$
(c) $Y = 1/\emptyset\ (1.05D+1.25H+1.0R+1.0F+1.25E+1.0T_o)$
(d) $Y = 1/\emptyset\ (1.0D+1.0P+1.0T_A+1.0H+1.0E'+1.0F)$
(e) $Y = 1/\emptyset\ (1.0D+1.0H+1.0R+1.0E'+1.0F+1.0T_o)$

Where Y = required yield capacity strength of the structure as defined below.

Ø = capacity reduction factor (defined in B.1.7)
D = dead loads of structures and equipment plus any other permanent loading contributing stress, such as hydrostatic or soil. In addition, a portion of the live load is added when it includes items such as piping, cable and trays suspended from floors. An allowance is made for future additional permanent loads.
P = design accident pressure load
F = effective prestress loads
R = force or pressure on structure due to rupture of any one pipe
H = force on structure due to operational thermal expansion of restrained pipes
T_o = thermal loads due to the temperature gradient through the wall during operating conditions.
T_A = thermal loads due to the temperature gradient through the wall and expansion of the liner. It is based on a temperature corresponding to the factored design accident pressure.
E = design earthquake load
E'= maximum earthquake load See Appendix 5A

W = Wind load, shall be substituted in the above equations in lieu of E if it produces higher-stresses. E' shall be similarly replaced by the higher tornado (337 mph).

Equation (a) assures that the containment will have the capacity to withstand pressure loadings at least 50 percent greater than those calculated for the MHA alone.

Equation (b) assures that the containment will have the capacity to withstand loadings at least 25 percent greater than those calculated for the MHA with a coincident design earthquake or design wind.

Equation (c) assures that the containment will have the capacity to withstand earthquake loadings, 25 percent greater than those calculated for the design earthquake, coincident with the rupture of any attached piping due to that earthquake.

Equations (d) and (e) assure that the containment will have the capacity to withstand the maximum hypothetical earthquake, concurrent with an MHA or with the rupture of any attached piping.

The loads obtained from the combinations and load factors given above, are less than the yield strength of the structure. The yield strength of the structure is defined as the upper limit of elastic behavior of the effective load carrying structural materials. For steels, both prestress and non-prestress, this limit is the minimum yield strength as given by the appropriate ASTM specification. For concrete, it is the ultimate values of shear (as a measure of diagonal tension) and bond per ACI 318-63 and the 28 day ultimate compressive strength for concrete in flexure (f'_c). The ultimate strength assumptions of the ACI Code for concrete stress are not allowed, i.e., the concrete stress is not allowed to go beyond yield and redistribute at a strain of 3 or 4 times that which causes yielding.

The maximum strain due to secondary moments, membrane loads and local loads, exclusive of thermal loads,is limited to that obtained by dividing the ultimate stress by the modulus of elasticity (f'_c / E_c), assuming a straight line stress distribution to the neutral axis. For the above loads combined with thermal loads the peak strain is limited to 0.003 inch/inch. For concrete membrane compression, the yield strength is assumed to be $0.85f'_c$ to allow for local irregularities, in accordance with the ACI approach. The stress in the reinforcing steel forming part of the load carrying system, is allowed to reach, but not to exceed, yield as is allowed by the ACI ultimate strength design.

A further definition of yielding is the deformation of the structure causing strains in the steel liner plate to exceed 0.0025 inch/inch. The yielding of non-prestress reinforcing steel is allowed, either in tension or compression, if the above restrictions are not violated. Yielding of the prestress tendons is not allowed under any circumstances.

Principal concrete tension due to combined membrane tension and membrane shear, excluding flexural tension due to bending moments or thermal gradients, is limited to $3\sqrt{f'_c}$. Principal concrete tension due to combined membrane tension, membrane shear, and flexural tension due to bending moments or thermal gradients is limited to $6\sqrt{f'_c}$. When the principal concrete tension exceeds the limit of $6\sqrt{f'_c}$, bonded reinforcing steel is provided in the following manner:

(a) Thermal flexural tension - Bonded reinforcing steel is provided in accordance with methods of ACI-505. The minimum area of steel provided is 0.15 percent in each direction.

(b) Bending moment tension - Sufficient bonded reinforcing steel is provided to resist the moment on the basis of cracked section theory using the yield stresses stated above with the following exception: When the bending moment tension is additive to the thermal tension, the allowable tensile stress in the reinforcing steel is 'f_y' minus the stress in reinforcing due to the thermal gradient as determined by the methods of ACI-505.

Shear stress limits and shear reinforcing for the radial shear are in accordance with Chapter 26 of the ACI 318-63 with the following exceptions:

Formula 26-12 of the Code shall be replaced by:

$$V_{ci} = Kb'd\sqrt{f'_c} + M_{cr}\frac{(V)}{(M')} + V_i$$

where:

$$K = [1.75 - \frac{0.036}{np'} + 4.0np']$$

but not less than 0.6 for $p' \geq 0.003$.

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

$$M_{cr} = \frac{I}{Y}\left[6\sqrt{f'_c} + f_{pc} + f_n + f_i\right]$$

f_{pc} = Compressive stress in concrete due to prestress applied normal to the cross-section, after all losses, (including the stress due to any secondary moment) at the extreme fiber of the section at which tension stresses are caused by live loads.

f_n = Stress due to axial applied loads. (f_n shall be negative for tension stress and positive for compression stress.)

f_i = Stress due to initial loads at the extreme fiber of a section at which tension stresses are caused by applied loads (including the stress due to any secondary moment. Therefore, f_i shall be negative for tension stress and positive for compression stress.

$n = \frac{505}{f'_c}$

$p' = \frac{A'_s}{bd}$

V = Shear at the section under consideration due to the applied loads.

M'= Moment at a distance d/2 from the section under consideration, measured in the direction of decreasing moment, due to applied loads.

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

The lower limit placed by ACI 318-63 on V_{ci} as $1.7b'd\sqrt{f'_c}$ is not applied.

Formula 26-13 of the Code shall be replaced by

$$V_{cw} + 3.5b'd\sqrt{f'_c}\left[\frac{\sqrt{1+F_{pc}+f_n}}{3.5\sqrt{f'_c}}\right]$$

The term f_n is as defined above. All other notations are in accordance with Chapter 26, ACI 318-63.

A. (1) This formula is based on the recent tests and work done by Dr. H. Mattock of the University of Washington.

(2) This formula is based on the commentary for Proposal Redraft of Section 2610 ACI-318 by Dr. A. H. Mattock, dated December, 1962.

When the above mentioned equations show that allowable shear in concrete is zero, radial horizontal shear ties are provided to resist all the calculated shear.

B.1.7 Yield capacity reduction factors

The yield capacities of all load carrying structural elements are reduced by a yield capacity reduction factor (Ø) as given below. This factor provides for "the possibility that small adverse variations in material strengths, workmanship, dimensions, control and degree of supervision while individually within required tolerance and the limits of good practice; occasionally may combine to result in under-capacity" (refer to footnote on page 66 of ACI 318-63 Code.).

Yield Capacity Reduction Factors:

Ø = 0.90 for concrete in flexure
Ø = 0.85 for tension, shear, bond and anchorage, in concrete
Ø = 0.75 for spirally reinforced concrete compression members
Ø = 0.70 for tied reinforced concrete compression members
Ø = 0.90 for reinforcing steel in direct tension
Ø = 0.90 for welded or mechanical splices of reinforcing steel
Ø = 0.85 for lap splices of reinforcing steel
Ø = 0.95 for prestressed tendons in direct tension

The Ø 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 material 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 Code provides for these variables by using theseØ factors:

Ø = 0.90 for concrete in flexure
Ø = 0.85 for diagonal tension, bond, and anchorage
Ø = 0.75 for spirally reinforces concrete compression members
Ø = 0.70 for tied compression members

Additional Ø factors have been selected on the basis of the material quality in relation to the ACIØ factors and represent the best judgment for each material and condition not covered directly by the ACI Code.

Conventional concrete design of beams requires that the design be controlled by yielding of the tensile reinforcing steel. This steel is generally spliced by lapping in an area of reduced tension. For members in flexure, ACI specifiedØ = 0.90. The same reasoning has been applied in assigning a value ofØ = 0.90 to reinforcing steel in tension, which now includes axial tension. However, the code recognizes the pos-sibility of reduced bond of bars at the laps by specifying aØ = 0.85. Mechanical and welded 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.

The only significantly new value introduced is ϕ = 0.95 for prestressed tendons in direct tension. A higher ϕ value than for conventional reinforcing has been allowed because (1) during installation the tendons are each jacked to about 94 percent of their yield strength, so in effect, each tendon has been proof tested, and (2) the method of manufacturing prestressing steel (cold drawing and stress relieving) ensures a higher quality product than conventional reinforcing steel.

B.1.8 Prestress Losses

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

(a) Seating of anchorage
(b) Elastic shortening of concrete
(c) Creep of concrete
(d) Shrinkage of concrete
(e) Relaxation of prestressing steel stress
(f) Frictional loss due to intended or unintended curvature in tendons.

All of the above losses can be predicted with a reasonable degree of accuracy.

The environment of the prestress system and concrete is not appreciably different, in this case, from that found in numerous bridge and building applications. Considerable research has been done to evaluate the above items and is available to designers. Building code authorities consider it an acceptable practice to develop permanent designs based on these values.

B.2.1 Liner Plate Criteria

The design criteria which are applied to the containment liner to ensure that the specified leak rate is not exceeded under accident conditions are as follows:

(a) That the liner be protected against damage by missiles that are coincident with the accident. (See paragraph B.3.1).
(b) That the liner plate strains be limited to allowable values for pressure piping.
(c) That the liner plate be prevented from developing significant distortion.
(d) That all discontinuities and openings be well anchored

to accommodate the forces exerted by the restrained liner plate, and that careful attention be paid to details at corners and connections to minimize the effects of discontinuities.

The most appropriate basis for establishing allowable liner plate strains is considered to be that portion of the ASME Boiler and Pressure Vessel Code, Section III, Nuclear Vessels, Article 4. Specifically the following sections are adopted as guides in establishing the allowable strain limits:

Paragraph N 412 (m)	Thermal Stress
Paragraph N414.5	Peak Stress Intensity
	Table N 413
	Figure N 414, N 415 (A)
Paragraph N 412 (n)	
Paragraph N 415.1	

Implementation of the ASME Code requires that the liner material be prevented from experiencing significant distortion due to thermal load and that the stresses be considered from a fatigue standpoint. (Paragraph N412 (m) (2)).

The following fatigue loads are considered in the 80-year design analysis of the liner plate (See Subsection 17.3.7 for additional details): C31

(a) Thermal cycling due to annual outdoor temperature variations. The number of cycles for this loading is 80 cycles for the unit life of 80 years. C31

(b) Thermal cycling due to the containment interior temperature variation during the startup and shutdown of the reactor system. The number of cycles for this loading is assumed to be 500 cycles.

(c) Thermal cycling due to the MHA will be assumed to be one cycle.

(d) Thermal load cycles in the piping systems are somewhat isolated from the liner plate penetrations by the concentric sleeves between the pipe and the liner plate. The attachment sleeve is designed in accordance with ASME Section III fatigue considerations. All penetrations are reviewed for a conservative number of cycles to be expected during the 80-year unit life. C31

The thermal stresses in the liner plate fall into the categories considered in Article 4, Section III, of the ASME Boiler and Pressure Vessel Code. The allowable stresses in Figure N-415 (A) are for alternating stress intensity for carbon steel and temperatures not exceeding 700°F.

In accordance with ASME Code Paragraph N412 (m) 2, the liner plate is restrained against significant distortion by continuous angle anchors and never exceeds the temperature limitation of 700°F and also satisfies the criteria for limiting strains on the basis of fatigue consideration. Paragraph N412 (n) Figure N-415 (A) of the ASME Code has been developed as a result of research, industry experience, and the proven performance of code vessels, and it is a part of recognized design code. Figure N-415 (A) and its appropriate limitations have been used as a basis for establishing allowable liner plate strains. Since the graph in Figure N-415 (A) does not extend below 10 cycles, 10 cycles is being used for MHA instead of one cycle.

The maximum compressive strains are caused by accident pressure, thermal loading prestress, shrinkage and creep. The maximum strains do not exceed .0025 in/in and the liner plate always remains in a stable condition.

At all penetrations the liner plate is thickened to reduce stress concentrations in accordance with the ASME Boiler and Pressure Vessel Code Section III.

B.2.2 Penetration Criteria

Penetrations conform to the applicable sections of ASA N6.2-1965, "Safety Standard for the Design, Fabrication, and Maintenance of Steel Containment Structures for Stationary Nuclear Power Reactors." All personnel locks and any portion of the equipment access door extending beyond the concrete shell conform in all respects to the requirements of ASME Section III.

The basis for limiting strains in the penetration steel is the ASME Boiler and Pressure Vessel Code Section III, Article 4, and therefore, the penetration structural and leak tightness integrity are maintained. Local heating of the concrete immediately around the penetration will develop compressive stress in the concrete adjacent to the penetration and a negligible amount of tensile stress over a large area. The mild steel reinforcing added around penetrations distributes local compressive stresses for overall structural integrity.

B.3 B.3.1 Missile Protection Criteria

High pressure reactor coolant system equipment which could be the source of missiles is suitably screened either by the concrete shield wall enclosing the reactor coolant loops, by the concrete operation floor or by special missile shields to block any passage of missiles to the containment walls. Potential missile sources are oriented so that the missile will be intercepted by the shields and structures provided. A steel missile shield is provided as part of the integrated Head Assembly. This shield is located above the control rod drive mechanisms to block any missiles generated from fracture of the mechanisms.

Missile protection is provided to comply with the following criteria:

(a) The containment and liner will be protected from loss of function due to damage by such missiles as might be generated in an MHA for break sizes up to and including the double-ended severance of a main coolant pipe.

(b) The engineered safeguards system and components required to maintain containment integrity will be protected against loss of function due to damage by the missiles defined below.

During the detailed design, the missile protection necessary to meet the above criteria are developed and implemented using the following methods:

(a) Components of the reactor coolant system are examined to identify and to classify missiles according to size, shape, and kinetic energy for purposes of analyzing their effects.

(b) Missile velocities are calculated considering both fluid and mechanical driving forces which can act during missile generation.

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

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

A detailed listing and description of the credible missiles is given in Appendix 5E.

APPENDIX 5-C
CONTAINMENT STRUCTURE STRAIN INSTRUMENTATION

5C-1 Scope

The purpose of the instrumentation is to measure the structural response of the vessel, during and after prestressing and during proof-testing, so that it can be compared to the theoretical analysis. The instrumentation was only necessary for original construction and testing. The instrumentation is no longer in use and is abandoned or removed.

5C-2 Number and Type of Instruments

The strains and deformations are recorded by a combination of electric resistance strain gages, load cells, and taut wire gages. The measuring devices are located in the base slab, reactor pit, cylindrical shell, and the dome of the Unit 3 containment. For Unit 4, the resistance strain gages are provided in the base slab, but none in the cylindrical shell or the dome, while the taut wire gages for deformation measurements are similar to Unit 3.

The following number of instruments are used in the two containments: The numbers in parenthesis indicate the gages damaged during construction.

	Number	
	UNIT 3	UNIT 4
Encapsulated electric resistance strain gauges, Budd Company designation C6-121-R2TC, attached to reinforcing bars.	222 (25)	32
Electric resistance strain gauges, encapsulated in a brass envelope embedded in concrete.	18	

	UNIT 3	UNIT 4
Carlson strain meters, Type SA-10, embedded in concrete.	6	
Taut wire gages for measuring deflections	30	30
Compression load cells, each with a maximum capacity of one million pounds, to measure loads acting on prestressing tendons.	10	
Three element rosette, electric resistance strain gauges, Budd Company designation C6-121B-R3T, attached to the outside face (concrete side) of the liner and penetration nozzles.	11 (6)	
Electric resistance strain gauges, Budd Company designation C6-141B, attached to the inside and outside faces of the liner and penetration nozzles.	173 (26)	8

Sufficient redundancy is provided in the type and number of instruments so that the damages to gages during installation does not impair the ability to monitor strain behavior of the structure to the desired degree.

5C-3 Layout of Instruments

The instrument layout is shown in Figure 5C-1.

The types and locations of the gauges are also described in the legend shown in the figure.

5C-4 Preparation and Installation of Electric Resistance Strain Gauges

Because of their vulnerability to moisture, special care is taken in bonding and waterproofing the electric resistance strain gauges. To reduce the possibility of faulty mounting of strain gauges on the reinforcing steel in the field, the gauges are attached to 3'-0" long reinforcing bars, encapsulated, and the wires soldered to the gauge leads and waterproofed in the shop.

Bonding and water proofing materials are GA-1 or GA-4 bonding cements, and GW-1 and GW-2 waterproofing coats, both supplied by the Budd Company.

5C-5 Zeroing of Electric Resistance Strain Gauges

The gauges are set at zero readings during installation. No subsequent adjustments are made.

5C-6 Effect of Concrete Properties on Recorded Strains

In order to determine the strains that are induced in the structure by the test loads, an evaluation has been made of the strains due to creep, autogenous growth, thermal diffusivity and coefficient of thermal expansion of the concrete. This is described in Appendix 5D.

5C-7 Procedure

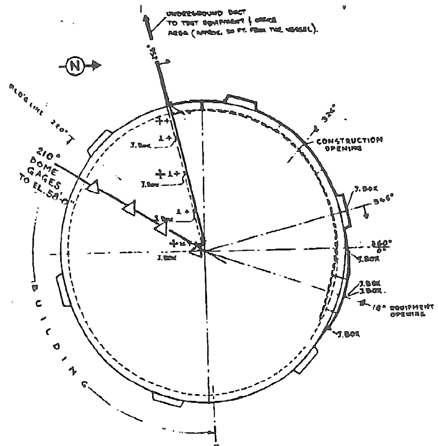
a. Test strain gauges immediately after installation.
b. Test strain gauges immediately after placing concrete and periodically thereafter.
c. Record strains and observe cracking at intervals suitably spaced during prestressing and immediately after all prestressing is completed.

d. After prestressing and before testing, take a number of readings to determine the effects of creep and shrinkage.

e. During pressurization, record strain and deformation measurements at the pressure increments stated in Section 5.1.7.2.

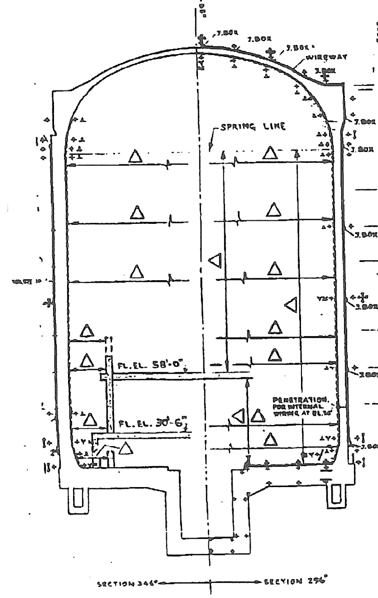
f. During depressurization, record strain and deformation measurements at the pressure increments stated in Section 5.1.7.2.

g. Observe the development of cracks during load application and take measurements.

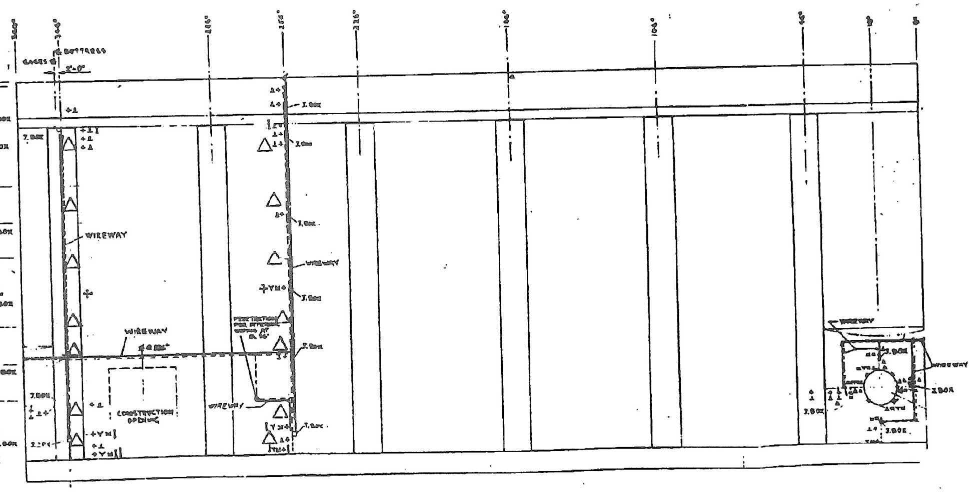


LEGEND

GAGE SYMBOL	GAGE TAG NO.	GAGE DESIGNATION	REMARKS
— + ✳	SGL-1	CG-121-R2TC	MOUNTED ON 3'-0" RE-BAR – P.O. 5610-C-48
⊸ ✣	SGE-2	A59-1	ENCAPSULATED IN BRASS ENVELOPE – P.O. 5610-C-48
Y	SGR-4	CG-121B-R3T	3-ELEMENT ROSETTE – EPOXY ENCAPSULATED – P.O. 5610-C-48
⊥ ✕	SGL-5	CG-141B	LINEAR GAGE – EPOXY ENCAPSULATED – P.O. 5610-C-48
—	SA-10	SA-10	CARLSON STRAIN METER – P.O. 5610-C-48
Δ	—	—	— — — TAUT WIRE
	—	—	



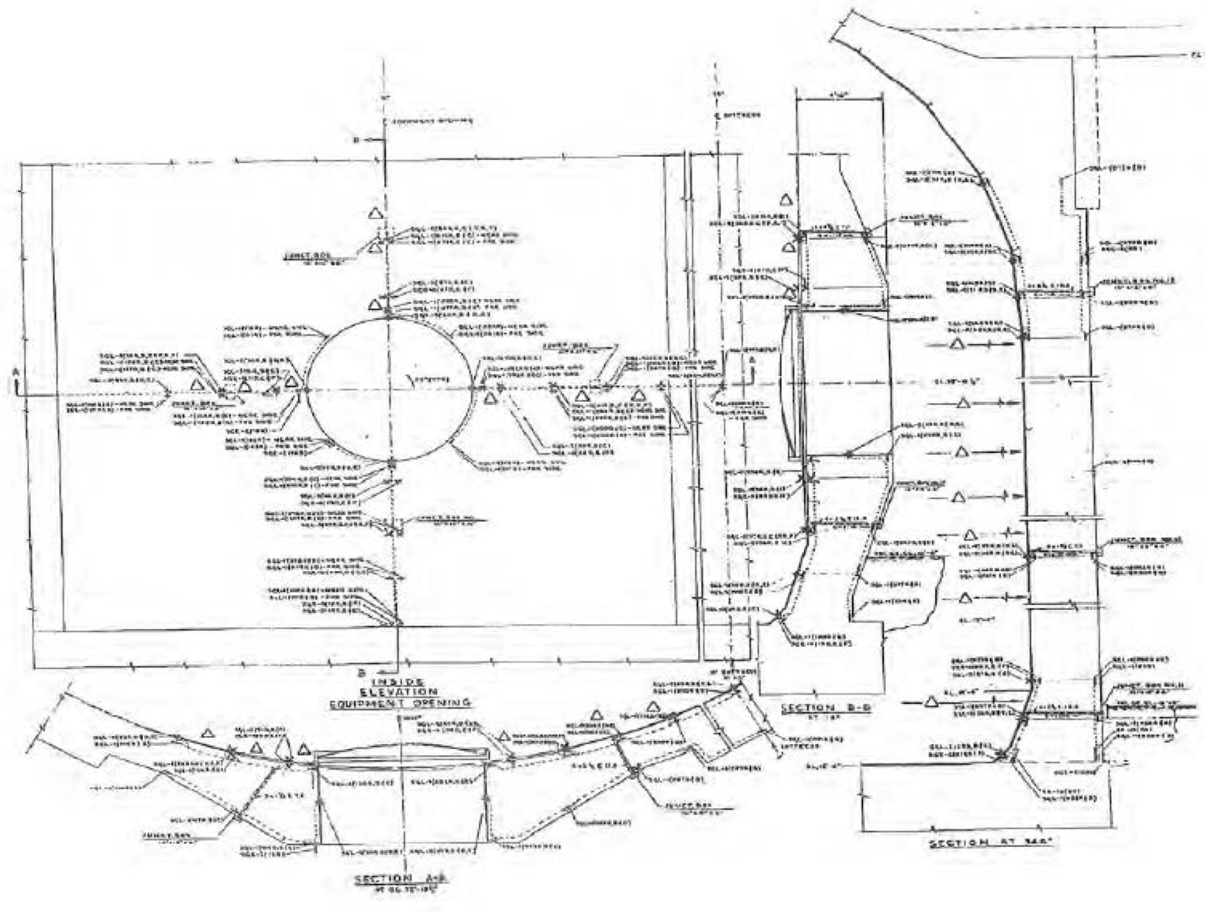
SECTIONS AT 256° & 346°



DEVELOPED ELEVATION

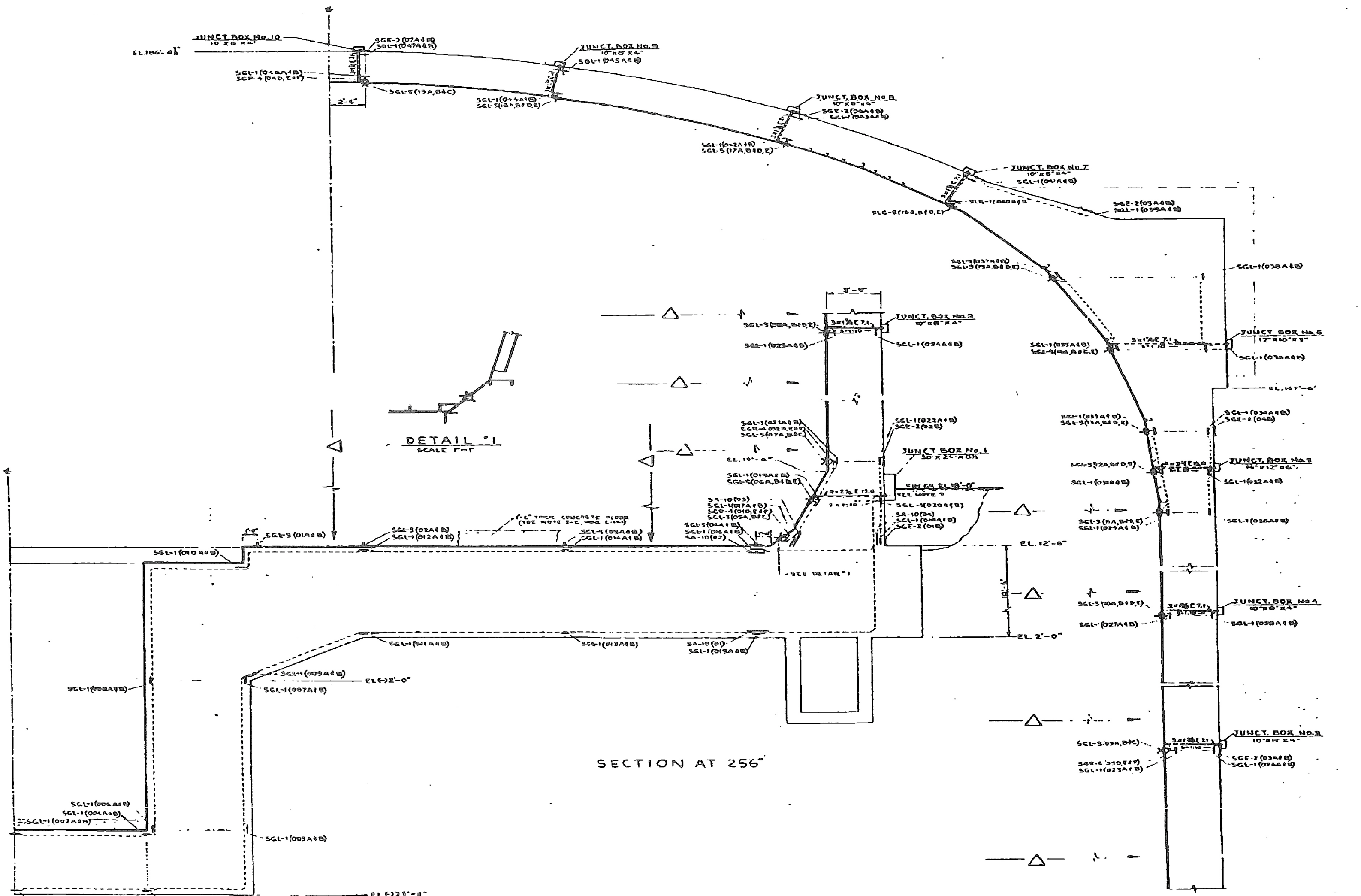
FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

CONTAINMENT STRUCTURE STRAIN GAGE INSTRUMENTATION
FIGURE 5C-1
Sheet 1



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

CONTAINMENT STRUCTURE STRAIN GAGE INSTRUMENTATION
FIGURE 5C-1
Sheet 2



FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

CONTAINMENT STRUCTURE STRAIN GAGE INSTRUMENTATION
FIGURE 5C-1
Sheet 3

APPENDIX 5D

Report

to

Bechtel

on

STUDIES OF CONCRETE FOR TURKEY POINT

NUCLEAR CONTAINMENT VESSELS

by

David Pirtz

University of California
Davis Hall
Berkeley, California
June 24, 1968

UNIVERSITY OF CALIFORNIA, BERKELEY

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COLLEGE OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING
DIVISION OF STRUCTURAL ENGINEERING
AND STRUCTURAL MECHANICS

BERKELEY, CALIFORNIA 94720

March 18, 1969

Mr. R. H. Stone
Bechtel Corporation
P. O. Box 607
Gaithersburg, Maryland 20760

Dear Mr. Stone:

For the two reports dated June 24, 1968 and August 12, 1968 on "Concrete Properties of Turkey Point Nuclear Containment Vessel", there is a typographical error on page 5, paragraph 2, second sentence. The second sentence should read as follows:

"The ~~Palisades~~ Turkey Point mix contained 6.1 scy and the results indicated a 28-day specific heat of 0.268 Btu. per lb. per °F for a mean temperature of 80.5°F, and a thermal diffusivity, h^2, of 0.0340 ft^2/hr."

Sorry to have caused you this inconvenience.

Sincerely yours,

David Pirtz

David Pirtz
Professor of Civil
Engineering

UNIVERSITY OF CALIFORNIA, BERKELEY

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AND STRUCTURAL MECHANICS

BERKELEY, CALIFORNIA 94720

June 24, 1968

Bechtel Corporation
190 Shady Grove Road
P.O. Box 607
Gaithersburg, Maryland 20760

Attention: Mr. R. E. Stade

Gentlemen:

RE: Concrete Properties of Turkey Point Nuclear Containment Vessels.

Transmitted herewith is a report entitled "Studies of Concrete for Turkey Point Nuclear Containment Vessels," which gives the results obtained to date from this study. Included are the results obtained from creep studies and from the thermal properties studies.

Sincerely yours,

David Pirtz

David Pirtz
Professor of Civil Engineering

DP:rp

cc: D. Graham

STUDIES OF CONCRETE FOR TURKEY POINT

NUCLEAR CONTAINMENT VESSELS

Manufacture of Specimens

The 6-in. by 18-in. molds for the creep specimens consisted of a 21-in. by 1/8 in. thick fabric-reinforced seamless butyl rubber sleeve which was bonded to a 2-in. thick stainless steel base plate. An 8-in. Carlson Meter was centered on the axis of the cylinder with its lead wire being brought out through a hole in the center of the base plate. A 16-gage sheet metal mold was then slipped over the rubber sleeve and banded snugly around the sleeve. The purpose of the outer steel mold is to assure that the outer surface of the creep specimens will be kept uniform during casting. A 1/8-in. by 8-in. metal rod was placed diametrically across the top of this mold to serve as support for a wire which held the meter in an axial position during casting. After casting, the wire was cut off and the rod removed.

Compressive strength specimens were case in 6-in. by 12-in. metal cans provided with lids which were soldered shut (one day after casting) to internally seal the specimen.

Specimens for specific heat and thermal diffusivity tests were case in 8-in. by 16-in. tin-plate cans. Specimens for specific heat tests were cast with a 1/2-in. hole centered on the axis of the specimen, and specimens for thermal diffusivity texts were cast solid except for a 1/2-in. diameter by 8-in. deep thermometer well centered on the axis of the specimen. The external metal container was left on the cylinders throughout the duration of the test.

Specimens for the linear coefficient of thermal expansion test were case in 6-in. by 12-in. tin-plate cans, with an 8-in. Carlson Strain Meter positioned midway along the axis of the mold.

One day after casting, the lids of all specimens for the specific heat, diffusivity and coefficient of thermal expansion tests were soldered to completely seal the specimens. The sealed specimens were then cured for 14 days at 100°F and at 70°F thereafter.

Casting of Specimens

The mix design and casting data for the Turkey Point nuclear containment vessel concrete is shown in Tables B and D and the physical properties of the aggregates are shown in Table C.

The aggregates used in the casting of the Turkey Point specimens were shipped in steel drums to Davis Hall on the University of California campus at Berkeley where the batching then took place.

The casting of the Turkey Point specimens took place on July 27, 1967, starting at 10:00 am and completed at 12:00 pm. The concrete was mixed in a 6-cu. ft. capacity pan-type mixer.

Hand-held internal vibrators were used in the casting of the Turkey Point specimens to insure proper compaction of the concrete.

After casting, lids were placed on the specific heat, diffusivity, thermal expansion and compressive strength cylinders, and all of them were placed in the 70° fog room.

The creep specimens were allowed about 3 hours time for the bleeding water to rise to the surface, and then a conical-shaped layer of mortar made from the original mix was formed on the top of each cylinder. The 2-in. thick stainless steel top-plates were then worked back and forth into position until the mortar appeared to be spread uniformly between the plate and the specimen. Finally, a square was used to assure that each top-plate was level, and the creep specimens were then moved to the 70°F fog room.

Sheet metal molds were stripped from the creep specimen at the age of one day, and the top 2 inches of the butyl rubber sleeve was then bonded and banded to the top plate to assure that the specimens would be internally sealed.

A certain number of the compressive specimens which are shown in Table A with parentheses, ( ), were then stripped and placed back in the 70°F fog room. The rest of the specimens were soldered shut in their cans, and placed along with the creep specimens in the 100°F room, where they were to remain for 14 days being moved to their respective temperature rooms.

RESULTS

Compressive Strength and Elastic Properties

Compressive strength and modulus of elasticity was determined for the Turkey Point concrete was the ages and temperatures indicated on the schedule. Compression tests were carried out in accordance with ASTM Standard C39-64 on "The Compressive Strength of Molded Concrete Cylinders", and the Static Young's Modulus of Elasticity and Poisson's Ratio were determined in accordance with ASTM Designation C469-65. The results of the compression tests are shown in the graphs at the end of the report and are summarized in Table E.

Creep Tests

Creep characteristics for the Turkey Point concrete were determined on sealed 6-in. by 18-in. cylinders initially loaded at the ages of 28, 180 and 365 days and at the temperatures of 70° and 100°F. Two specimens were tested at each temperature as noted in the schedule and the remaining unloaded specimens were used to determine autogenous volume change.

A stress level of 1500 psi was applied to all creep specimens by a hydraulic system with an automatic controller which was used to maintain a constant stress level.

Creep strains per psi, which have been corrected for autogenous volume change, are shown plotted on the graphs in the back of the report.

TESTS OF THERMAL PROPERTIES

The various thermal properties of the Turkey Point concrete tested in this investigation included specific heat of concrete, thermal diffusivity of concrete and linear coefficient of thermal expansion of concrete.

Specific Heat and Thermal Diffusivity

The mix design and casting data for the Turkey Point concrete are shown in Tables B and D. The Palisades mix contained 6.1 scy and the results indicated a 28-day specific heat of 0.268 Btu, per lb. per °F for a mean temperature of 80.5°F, and a thermal diffusivity, h^2, of 0.0340 ft^2/hr.

Thermal Coefficient of Expansion

The two 6-in. by 12-in. thermal-coefficient specimens for the Turkey Point concrete were cycled from 70°F to 40°F, 70°F, 100°F and then back to 70°F. The cycle was started at the age of 26 days and was completed at the age of 30 days. The specimens were then kept at 70°F awaiting the next cycling which took place at the age of 180 days. The average 28-day linear coefficient of thermal expansion of two specimens was 5.2 millionths per °F.

TABLE A

Scope: To outline purpose, procedures and extent of tests, and results to be obtained.

Summary: Six different tests are required for the Turkey Point containment vessel concrete. These are:

TEST	TEMP	7-DAY LOAD 8-3-67	28-DAY LOAD 8-24-67	180-DAY LOAD 1-23-68	365-DAY LOAD 7-27-68
Uniaxial Creep 6" x 18"	70°F 100	- -	2 2	2 2	2 2
Modulus of Elasticity 6" x 12"	70°F 100	(1) -	2 + (2) 2	2 + (2) 2	2 + (2) 2
Autogenous Control Cylinder 6" x 18"	70°F 100	- -	2 2	same same	same same
Coefficient of Thermal Expansion 6" x 12"	70 40 70 100 70	-	2	same	same
Diffusivity and Specific Heat 8" x 16"		-	2	-	-
Compressive Strength 6" x 12"	70°F 100	(3) -	3 + (3) 3	3 + (3) 3	3 + (3) 3

TABLE B

MIX DESIGN FOR TURKEY POINT CONCRETE

MATERIAL		SOURCE
Cement:	Florida, Type II	Shipped 5-1-67 from Oolite Industries Inc., Miami, Fla.
Sand:	Oolite screenings	
Course Aggregate:	1" x #4 Oolite Rock	
WRA Admixture:	Retardwell	Received 5-19-67 from Union Carbide

SPECIFICATIONS

5,000 psi. at 28 days
3 fl. oz. Retardwell WRA per sack of cement
2" slump
65°F maximum casting temperature

ONE CUBIC YARD BATCH, SATURATED SURFACE-DRY WEIGHTS

Cement	588 lbs.
Sand	1290
1" x #4	1660
Water	279
WRA	555 ml.

TABLE C

PHYSICAL PROPERTIES OF THE AGGREGATES

U.S. STANDARD SIEVE SIZE	% PASSING U.S. STANDARD SIEVE SIZE	
	OOLITE SCREENINGS	OOLITE 1" BY NO. 4
1 1/2"	-	100
1"	-	84
3/4"	-	37
1/2"	-	13
3/8"	-	4
#4	100	4
#8	92	4
#16	72	4
#30	54	4
#50	27	4
#100	4	4
FINENESS MODULUS	2.1	6.80
MATERIAL FINER THAN #200	1.5%	2.1%
BULK SP. GR. (SSD BASIS)	2.52	2.44
ABSORPTION CAPACITY (OD TO SSD)	3.4	5.1

TABLE D

CASTING DATA

DATE	7-29-67	7-27-67	
SPECIMENS CAST	1-8x16" sp. heat	12-6x18" creep 2-6x12" thermal expansion 30-6x12" compressive strength Young's modulus and Poisson's ratio 1-8x16" diffusivity	
BATCH NO.	1	1	2
UNIT WT. PCF	142.4	143.1	143.2
SLUMP, IN.	1 1/2	3	2 1/2
AIR, % BY VOL.	3.2	2.3	2.2
CONC. TEMP. ºF	63	59	62
AIR TEMP. ºF	-	70	70
CEM.FACTOR, SCY	6.1	6.4	6.4
WATER, PCY	256	263	260

TABLE E

COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES

Age, Days	Temp °F	Compressive Strength psi[a]	"E"[b] psi x 10^6	Poisson's Ratio[b]
7	(70)	6,490	4.1	0.23
28	(70)	7,660	4.6	0.22
	70	7,780	4.7	0.23
	100	7,480	4.4	--
180	(70)	8,000	4.6	0.25
	70	7,760	4.6	0.24
	100	7,660	4.6	--

Note

All specimens with ( ) were stored unsealed in 70° fog room

a - Average of three 6 by 12-in. cylinders

b - Average of two 6 by 12-in. cylinders

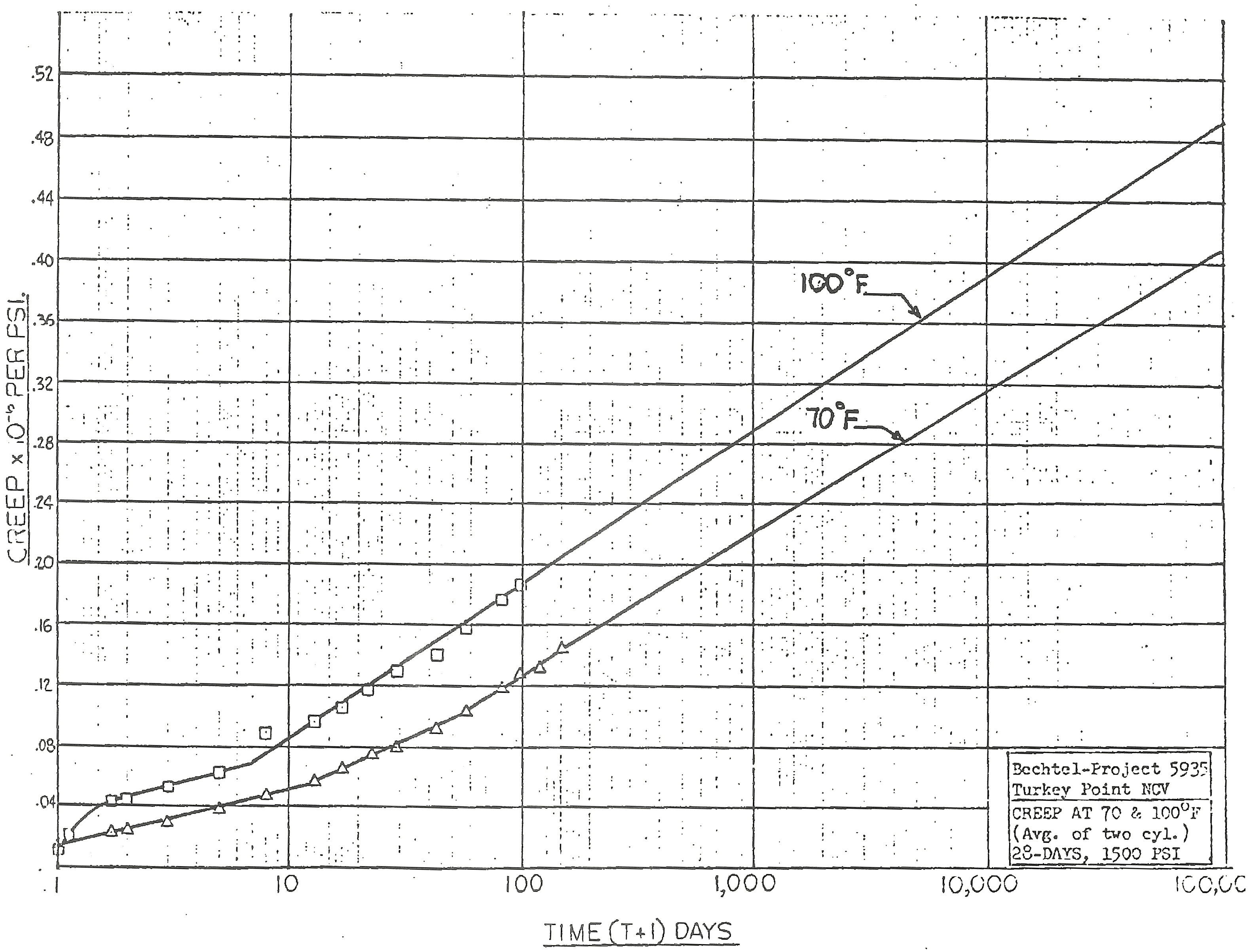
TABLE F

TURKEY POINT NCV CONCRETE CREEP AND CORRESPONDING FACTORS

CYCL. NOS.	STORAGE TEMP. °F	AGE CYL. LOAD	CREEP: DAY AFTER LOADING	CREEP: CREEP/PSI X10^{-6}	CREEP: CREEP X10^{-6}	POISSON RATIO	"E" X 106: INSTANT-ANEOUS	"E" X 106: SUS-TAINED	AUTOGENEOUS VOLUME X 10^{-6}	COEF.OF THERMAL EXPX10^{-6}	AVERAGE COMPRESSIVE STRENGTH PSI	"E"X10^6 STATIC (6"X12"CYL)
A21,A23	70	28	1	0.220	330		5.1	--				
			180	0.349	523	0.23	--	2.9		5.2	7,780	4.7
			14,600	0.528	792	--	1.9					
A22,A24	70	180	1	0.181	272		6.2	--				
			180	0.259	388	0.24	--	3.9	-2	5.1	7,760	4.6
			14,600	0.362	543	--		2.8				
A25,A26	70	365							-4			
A27,A28	100	28	1	0.252	378		4.8		--			
			180	0.461	691	--	--	2.2		5.2	7,480	4.4
			14,600	0.617	925	--		1.6				
A30,A31	100	180	1	0.219	328		5.5		--			
			180	0.334	500	--	--	3.0	-5	5.1	7,660	4.6
			14,600	0.488	733	--		2.0				
A29,A32	100	365							-8			

NOTE

Creep for 40 years (14,600 Days) is projected



CREEP x 10⁻⁶ PER PSI.
.52
.48
.44
.40
.36
.32
.28
.24
.20
.16
.12
.08
.04
100°F
70°F
Bechtel-Project 5935
Turkey Point NCV
CREEP AT 70 & 100°F
(Avg. of two cyl.)
28-DAYS, 1500 PSI
1
10
100
1,000
10,000
100,000
TIME (T+1) DAYS

TABLE G

APPENDIX 5E

MISSILE PROTECTION CRITERIA

5E-1 General

Hypothetical missiles that could be generated either from various components of the unit or by hurricanes and tornadoes, are considered in the design. Hypothetical missiles may be generated either inside or outside containment. Components and systems that are essential for the safety of the public and are required to function immediately after a MHA, are protected by either concrete or steel barriers designed to resist missile impact, or by redundancy and spacing to maintain their integrity with no-loss-of-function, when subjected to internal missiles.

5E-2 External Missiles

The following external missiles are considered in the design of the units:

1) Tornado-generated missiles

Missile	Size	Velocity (mph)	Weight (lbs.)
Corrugated Sheet of Siding	4' x 8'	225	100
Bolted wood decking	12' x 4' x 4"	200	450
Passenger Car (on ground)	----	50	4000

These tornado-generated missiles are considered concurrently with a loss-of-offsite-power but not concurrently with a maximum hypothetical accident (MHA).

2) Turbine missiles are not considered a potential threat to plant vital systems, structures or components and need not be considered in new designs or modifications. The originally installed built-up Low Pressure turbine rotors were replaced with new fully integral rotors. This replacement resulted in a reduction of the probability of unacceptable damage due to a turbine missile to within NRC accepted guidelines. (Refs. 5E-1 thru 5E-3). C31

Following the EPU, the Unit 3 & 4 LP Turbines were upgraded to improve efficiency. The OEM provided a revised turbine missile analysis which resulted in the probability of a turbine missile based on a 6 month turbine valve test interval of 2.9E-06 for the upgraded Unit 3 & 4 LP Turbine system (Ref 5E-9). C31

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Unit components and systems, vital to a safe shutdown, are protected from the postulated tornado missiles. Protection of such components and systems from the missiles is provided by either of the following:

1. Enclosure by either concrete or steel structures.

2. Redundancy and spacing of the components and equipment.

Of the vital systems and components listed in Appendix 5A, those not fully enclosed by concrete structures are listed below:

Item 4. Main Steam Safety, Isolation and Atmospheric Dump Valves.
Item 5. New Fuel Storage Facilities
Item 6. Auxiliary Feedwater System (Condensate Storage Tanks)
Item 7. Emergency Diesel Generators (Unit 3 Diesel Fuel Oil Storage Tank, Solenoid Valves, Fuel Oil Transfer Pumps and Exhaust Silencers)
Item 9. Refuel Water Storage Tanks
Item 11. Intake Cooling Water System
Item 12. Component Cooling System
Item 13. Spent Fuel Storage Facilities

These systems and equipment are protected from wind driven missiles by redundancy and separation, or analyzed to show that the failure of the non-vital portion would not prevent safe shutdown or cause an uncontrolled release in excess of the established guidelines.

The following list represents an evaluation of the protective measures that are taken against missile impact, for each of the vital systems, components, and structures listed in Appendix 5A, based on a concurrent loss-of-power condition:

1. <u>Reactor Coolant System:</u>

   The reactor vessel and all reactor coolant loop components are enclosed by the containment structure which is designed to withstand impact of all missiles.

2. <u>Containment System:</u>

   The containment structure is designed to withstand impact of all missiles.

All containment electrical penetration areas are protected from the missiles by the electrical penetration rooms which are designed to withstand the missile impact.

Most containment pipe penetrations and valves are enclosed by the pipe and valve room which is designed to withstand the impact of missiles. Some penetrations are exposed to tornado missiles; however, the design of the containment structure in conjunction with the redundancy provided by the interior penetration isolation barrier permits the loss of the exterior penetration isolation barrier due to missile impact without the loss of containment integrity.

There are two containment purge supply valves and two containment purge exhaust valves. One each of these valves is located on the outside of containment and the second valve is located at the corresponding location on the inside of containment. The redundancy provided by the interior valve permits the loss of the exterior valve due to a missile, which is permissible without affecting shutdown capability.

3. Main Steam & Feedwater Lines Within the Containment:

These lines are enclosed by the containment structure which is designed to withstand impact of all missiles.

4. Main Steam Outside of Containment:

Spacing of the three main steam lines and associated valves and equipment outside the containment allows the loss of only one steam line due to a missile, which is permissible without affecting shutdown capability.

5. New Fuel Storage Facilities:

The New Fuel Room is protected from missile impact by a concrete roof and concrete walls. The entry into the new fuel storage facility consists of a roll-up metal door which is not missile resistant. However, the only safety related components which are exposed to missile impact through the door opening are new fuel assemblies. Since these assemblies are not irradiated, they do not present a potential for uncontrolled release of radioactivity in excess of 10CFR100 guidelines.

6. Auxiliary Feedwater System:

For a loss-of-power situation, the auxiliary feedwater pumps and turbine drivers are provided with missile protection by their location under platforms designed to withstand the effects of a missile. The associated piping is protected by physical separation and other structures.

The valve stations and associated nitrogen back-up stations are protected from tornado missiles by redundancy and separation.

The Auxiliary Feedwater System is supplied by the Condensate Storage Tanks. Redundancy and spacing of the Condensate Storage Tanks provide the required system capability in the event of damage to one component by a tornado missile. If one tank is lost due to missile impact with a coincidental loss of power, an adequate supply of water is available from the remaining tank to achieve hot standby for a period of time since the tanks are cross-tied. If water inventory decreases below an adequate volume in the remaining tank, non-safety related sources of make-up water to the tank are available.

7. Emergency Diesel Generators:

The Unit 3 emergency diesel generators located to the north of Unit 3 containment are protected from tornado missiles by a concrete enclosure. Associated air cooled radiators are protected by the overhanging projection of the structures roof and steel grating.

The Unit 3 EDG diesel fuel oil storage tank and associated solenoid valves which supply the transfer pumps and ultimately the day tanks are not missile protected. Other equipment not missile protected includes portions of inlet supply/cross-tie piping to a Unit 3 Day Tank which is routed from a non-safety related underground cross tie piping and Class I, missile protected Unit 4 Fuel Oil Storage Tanks. This is one of numerous methods of supplying fuel oil to the Unit 3 EDGs. In addition to this method of supplying fuel oil to the Unit 3 Day Tank, an alternate fill line is available on the inlet side of the Day Tank. This alternate fill line may be used to connect a temporary hose from the Unit 4 Fuel Oil Storage Tank source along with connecting to a mobile transfueler managed by the FLEX support program and guidelines. Each day tank is connected to a skid-mounted tank. The fuel oil day tank and C30 skid-mounted tank contain sufficient oil inventory to allow operation of the diesel generator until a mobile fuel oil tank or other temporary means could supply additional fuel oil.

Two diesel fuel transfer pumps are used to transfer diesel oil from the diesel fuel oil storage tank to the day tanks. The pumps are not missile protected. However, if the pumps become non-functional due to external missile damage, the fuel oil day tanks contain sufficient fuel oil inventory to allow operation of the diesel generator until a mobile fuel oil tank could supply additional fuel oil to the diesel generators. This emergent supply of fuel oil from multiple mobile sources is capable within the allotted timeframe considering a tech spec minimum 2000 gallons total within the day tank and skid C30 mounted fuel tank. The 2000 gallon minimum fuel supply will enable the highest loaded EDG (i.e., 3B EDG) to operate at a minimum of 15 hours which is based on the 1965 Northeast USA blackout, which lasted approximately 13 hours. The presence of an alternate fill line on the inlet side of the Unit 3 Day Tank allows for the connecting of a temporary hose from the Unit 4 Fuel Oil Storage Tank source along with connecting to a mobile transfueler managed by the FLEX support program and guidelines. The FLEX guidelines, procedures and off-normal operating procedures demonstrate the equipment availability, means and methods to supply an alternate source and volume of fuel oil within 15 hours. The pipe from the EDG fuel oil transfer pumps to the fuel oil day tanks discharge into the top of the tanks. If the EDG fuel oil transfer pumps or connected pipe rupture due to an external missile impact, the fuel oil in the fuel oil day tanks will not leak because a siphon will not be created.

The Unit 3 diesel exhaust silencers located on top of the Unit 3 EDG Building roof are exposed to potential damage from external missiles. The maximum size hypothetical missile could potentially damage one of the two silencers. The other silencer would not be affected since the separation distance between the silencers exceeds the maximum missile size. Damage to its exhaust system may force the shut down of the affected diesel generator. However, this scenario does not adversely affect the capability of the remaining Unit 3 Diesel Generator and the two Unit 4 Diesel Generators to bring the units to a safe shutdown. C30

The Unit 4 emergency diesel generators and their associated diesel oil storage tanks, located in the Unit 4 Emergency Diesel Generator (EDG) Building are protected from the Design Basis Tornado for Region I as given in Regulatory Guide 1.76, Revision 0, in accordance with the protective barrier design of NUREG - 0800, Standard Review Plan (SRP) Section 3.5.3 (Rev. 1 - July 1981).

8. Containment Polar Crane and Rail Support:

These components are enclosed by the containment structure which is designed to withstand impact of all missiles.

9. Refueling Water Storage Tanks:

The refueling water tanks (RWSTs) are carbon steel, epoxy lined tanks located in the yard east of the Auxiliary Building. A loss of either RWST due to a missile impact will not affect shutdown capability because:

a) The safe shutdown condition for Turkey Point is defined as Hot Standby with the reactor coolant system temperature greater than or equal to 540°F. This condition does not require RWST inventory to achieve or maintain reactor subcriticality.

b) The RWST is required to provide borated water to the safety injection system, RHR and containment spray systems during maximum hypothetical accident (MHA) conditions. However, a MHA accident coincident with a missile impact is not a design basis for the plant.

10. <u>Emergency Containment Cooling Units:</u>

These components and equipment are enclosed by the containment structure which is designed to withstand impact of all missiles.

11. <u>Intake Cooling Water System:</u>

Protection is provided for the pumps and motors by separation. The basket strainers are located within the CCW Rooms which are designed to withstand the impact of tornado missiles. The loss of one pump, motor or basket strainer due to a missile leaves adequate intake cooling water capability for shutdown. The pipe and valve pits adjacent to the intake structure are protected by a reinforced concrete barrier. The piping from the pumps to the component cooling water heat exchanger inlets are below grade and, therefore, protected from missile impact. The TPCW isolation valves are protected by separation. The loss of one TPCW isolation valve or rupture of associated piping will not prevent the ICW System from performing its safety related functions.

The Intake Structure is designed to withstand impact of all missiles.

12. <u>Component Cooling System:</u>

The steel grating located over the Units 3 & 4 Component Cooling Water Rooms and the concrete walls are designed to withstand impact of all tornado missiles. Based on the location of the CCW room entrance openings, the presence of intervening structures, and the location of components inside the CCW pump rooms, a tornado missile resulting in a loss of the CCW function is not considered credible.

The Supplemental Cooling System (SCS) is part of the CCW system pressure boundary when SR AOV, CV-2216 is open and connected to the CCW system. The following engineering features are included in the SCS design to mitigate potential inventory loss on tornado missile impact: C29

- SR check valve on SCS discharge

- SR AOV, CV-2216 on SCS suction (fail close on loss of power or air supply)

- 2 SR control circuits to close AOV and trip pump

The check valve and AOV are located in the Auxiliary Building which provides a qualified barrier against a tornado missile. The AOV closes on low level in the CCW head tank via one of the SR control circuits. The other SR circuit trips the SR SCS pump concurrent with closure of the AOV. The cooling function of SCS is non-vital and its pressure boundary is no longer vital when it is isolated from the CCW system. C29

The SCS is a shared system (See UFSAR Table A-1). The SCS can be aligned to Unit 3 or Unit 4 when Unit 3 or Unit 4 is in any MODE of operation.

13. <u>Spent Fuel Storage Facilities:</u>

The Spent Fuel Storage Facilities consist of the spent fuel pit and racks, the spent fuel pit pumps, motors and heat exchangers. The spent fuel pit is designed to protect the spent fuel from tornado missiles. The spent fuel pit cooling loop consists of pumps, heat exchangers, and piping. As stated in UFSAR 9.3 and SER 9.5, redundancy of this equipment is not required because of the large heat capacity of the pit and its correspondingly slow heatup rate. The piping is designed such that its loss would not drain the pit to a level lower than six feet above the top of the fuel racks (this assumes valve 797 is lock closed per Administrative Controls). The off-site doses are not changed from those calculated for loss of cooling. Therefore, it is not necessary for the spent fuel pit cooling system to be totally protected from tornado missiles. However, the system has been upgraded to remain functional after a safe shutdown earthquake.

14. Safety Injection System:

The High Pressure Safety Injection Pumps and Containment Spray Pumps are protected from the missiles by concrete enclosures designed to withstand the missile impact. Portions of the High head Safety injection and Residual Heat Removal recirculation piping are exposed to missile impact. However, these portions of piping are not required to support plant safe shutdown. A tornado missile impact coincident with a MHA is not a design for the plant.

The Residual Heat Removal pumps are protected from a missile because of their location in the lower portions of the Auxiliary Building.

The Residual Heat Removal heat exchangers are protected by a segment of the Auxiliary Building roof directly overhead, which is designed to withstand the effects of all missiles.

The Accumulator Tanks are enclosed by the containment structure which is designed to withstand impact of all missiles.

15. Chemical and Volume Control System:

The Charging Pumps are protected from missiles by concrete enclosures designed to withstand missile impact.

The Volume Control Tank is enclosed within a concrete structure designed to withstand a missile impact.

The three Boric Acid Tanks are shared between both units with sufficient boric acid inventory to conduct an orderly shutdown and cooldown of both units to cold shutdown conditions. Because the three tanks are located close to one another, missile protection is provided by concrete barriers. The boric acid pumps, filters and blender (batch tank), which are located adjacent to the tanks, are also missile protected by the concrete barriers.

The Regenerative Heat Exchanger is enclosed by the containment structure which is designed to withstand impact of all missiles.

16. Fuel Transfer Tube:

The fuel transfer tube is protected from a missile because of its location in the SFP Building and the Containment Building. The short section of the tube between the buildings is protected by a concrete enclosure.

17. Post Accident Containment Venting System:

The piping within containment is protected by the containment building which is designed to withstand impact of all missiles. The piping outside of containment is protected from a missile because of its location within the Auxiliary Building.

18. Waste Handling Facilities Building:

The waste handling facilities are located within the Radwaste Building which is designed to withstand tornado missile impact. The entrance doors into the Radwaste building are not missile resistant. However, the waste handling facilities are located within the interior portions of the building and are not in the path of a postulated missile entering through the doors.

19. Switchgear:

The 4160 and 480 volt switchgear located at the generator end of each turbine pedestal deck are protected from missiles by a concrete enclosure.

20. Control Room Equipment:

The control room walls and roof are designed to withstand the missile effects. Missile protection is also provided for the motor control centers, located at grade elevation within the Control Building.

5E-3 Internal Missiles

High pressure reactor coolant system equipment which could be the source of missiles is suitably enclosed either by the shield walls enclosing the reactor coolant loops or by the concrete operating floor to block any passage of missiles to the containment walls. A steel missile shield is provided as part of the Integrated Head Assembly. This shield is located above the control rod drive mechanisms to block any missiles generated from fracture of the mechanisms.

Systems containing hot pressurized fluids and which might affect the engineered safeguards components have been carefully checked against the possibility of being sources of missiles. The general criterion adopted has been to make provision, when necessary, against the generation of missiles rather than allow missile formation and try to contain their effects.

Once the design requirement that the above systems are not to be sources of missiles has been set forth, identification of potential deficiencies and generation of adequate fixes is done through the quality assurance program.

The following examples illustrate how this approach has been implemented.

Valves

All the valves installed in the Nuclear Steam Supply System have stems with a back seat. This rules out the probability of ejecting a valve stem. Even if it were assumed that the stem threads fail, analysis shows that the back seats or the upset end cannot penetrate the bonnet and thereby become a missile. Additional interferences are encountered with air and motor operated valves.

Valves with nominal diameter larger than 2" have been designed against bonnet-body connection failure and subsequent bonnet ejection by means of:

a. Using the design practice of ASME B&PV Code Section VIII which limits the allowable stress of bolting material to less than 20% of its yield strength.

   Original manufacture and maintenance practices complied with this design feature. Industry experience has shown that the former practice of torquing to low percentages of yield stress is not as desirable as higher values. Torquing to higher values provides more resistance to self-loosening due to vibration without compromising the structural adequacy of the joint. During assembly, the studs/nuts are tightened to higher percentages of yield stress in accordance with the recommendations of EPRI/NMAC good bolting practices.

b. Using the design practice of ASME Section VIII for flange design.

c. And by controlling the load during the bonnet body connection stud tightening process.

The pressure containing parts except the flange and studs are designed per criteria established by the USAS B16.5. Materials of construction for these parts are procured per ASTM A182, F316, or A351, GR CF8M.

Stud and nut material is ASTM A193-B7 and A194-2H or corrosion resistant alloys where boric acid residue is a concern. The proper stud torquing procedures and the use of torque wrench, with indication of the applied torque, limit the stress of the studs to the allowable limits established in the ASME Code, i.e., 20,000 psi. This stress level is far below the material yield, i.e., about 105,000 psi. The complete valves are hydrotested per USAS B16.5. (1500# USAS valves are hydro to 5400 psi). The cast stainless steel bodies and bonnets are radiographed and dye penetrant tested to verify soundness.

Valves with nominal diameters of 2" or smaller are forged and have screwed bonnets with canopy seals. The canopy seal is the pressure boundary while the bonnet threads are designed to withstand the hydrostatic end force. The pressure containing parts are designed per criteria established by the USAS B16.5 Specification.

Valve Replacements:

Use of the ASME Section III Code for procurement of replacement valves is acceptable in lieu of the above design requirements. The ASME Section III Code is a well recognized nuclear design code meeting the design and quality requirements of the Nuclear industry. Additionally, substitution of the original construction code for ASME Section III is permitted via the NRC adopted Code Case N-406.

<u>Reactor Coolant Pump Flywheel</u>

The reactor coolant pump flywheel is not considered to be a credible source of missiles because of conservative design and care in manufacture and inspection. The flywheel material is ASTM A-533 having an NDTT less than 10°F. The design results in a primary stress less than 50% of the material yield strength at operating speed. The flywheel is inspected at least once every 20 years(Reference 5E-5 and 5E-6), by either conducting C26
an in-place ultrasonic examination over the volume from the inner bore of the flywheel to the circle of one-half the outer radius, or by conducting a surface examination

(magnetic particle and or liquid penetrant) of exposed surfaces of the disassembled flywheel. The design overspeed of the pump is 125% of rated speed. The maximum pump speed on loss of external load is 112% of rated speed.

5E-4 References

5E-1 WCAP 11525, “Probabilistic Evaluation of Reduction in Turbine Valve Test Frequency”, June 1987.

5E-2 NRC Safety evaluation Report, ”Approval for Referencing of Licensing Reports WSTG-1-P, WSTG-2-P and WSTG-3-P”, dated 02/02/87.

5E-3 Westinghouse Document CT-27081, “Results of the Analysis of the Probability of the Generation of Missiles from fully Integral Nuclear Low Pressure Rotors”, condensed from WSTG-4-P.

5E-4 WCAP-16054-P, “Probabilistic Analysis of Reduction in Turbine Valve Test Frequency for Nuclear Plants with Siemens-Westinghouse BB-96/96 Turbines,” April 2003.

5E-5 WCAP-15666-A, Revision 1, “Extension of Reactor Coolant Pump Motor Flywheel Examination,” October 2003.

5E-6 License Amendments 242 and 238 issued on February 23, 2010.

5E-7 Westinghouse LTR-RAM-I-11-036,Rev.1, “Turkey Point Failure Analysis and Missile Probability Calculation for the Turkey Point Units 3 & 4 Uprate” (Transmitted to FPL via Letter SEI-FPL-EHC-0097 and accepted by FPL via Letter EPU-PT-11-1212).

5E-8 EC 284975, Rev 1, “UFSAR Change to Allow Extension of the Six Month Turbine Valve Test Frequency to Twelve Months.”

5E-9 Siemens letter SEI-FPL-PTN-16-127, CT27546 Revision 1 "Missile Probability Analysis" May 31, 2016 (EC285668 and EC285699). C31

APPENDIX 5F

INTERNAL PLANT FLOODING

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5F.1 PTN INTERNAL FLOODING PROTECTION

As a result of Atomic Energy Commission (AEC) requests for review of the susceptibility of safety-related systems to flooding from failure of non-Category I (seismic) systems (References 3, 6, 9, and 10), FPL performed reviews of the Turkey Point facility, documenting the results of these reviews through a series of submittals to the AEC/NRC commencing in November 1972 (References 1, 4, 5, and 7). These reviews considered failure of non-Category I (seismic) systems or components simultaneous with a loss of offsite power. The culmination of these submittals resulted in the NRC issuing a Safety Evaluation Report (SER) on September 4, 1979, which concluded that a sufficient level of protection from internal flooding due to failure of non-Category I (seismic) systems for equipment important to safety is provided. A summary of the Turkey Point internal flooding reviews is provided in the sections below.

5F.1.1 FLOODING SOURCES AND SUSCEPTIBLE PLANT AREAS

During the reviews of the Turkey Point nuclear facility, the following were determined to be sources of potential flooding:

1. Circulating Water Systems

2. Fire Protection Systems

3. Drainage System

4. Chemical and Volume Control Systems (holdup tanks)

5. Primary and Service Water Tanks

The following systems, equipment, or locations were considered to require protection from flooding:

1. Unit 3 Diesel Generator Room

2. Residual Heat Removal (RHR) Pump Rooms

3. Switchgear Rooms

4. Safety Injection Pumps

5. Motor Control Centers

6. Charging Pumps, Containment Spray Pump Rooms, and Boric Acid Transfer Pump Rooms

7. Component Cooling Water Pumps

8. Auxiliary Feedwater Pumps

9. Control Room, Reactor Protection Equipment Rooms, and Battery Rooms

5F.1.2 GENERAL INTERNAL FLOODING CONSIDERATIONS

Protection from internal flooding for equipment important to safety is generally provided by the arrangement of that equipment. All equipment important to safety, with the exception of the RHR system, is located in rooms that are at or above grade level. The equipment that could possibly be damaged by flooding is also located well above floor level in these rooms. The flooding potential of these rooms is small because either the water can flow outside onto the ground or the entire Auxiliary Building below grade level would have to flood first.

Flood mitigating features for which credit is taken at Turkey Point include the following:

1. Locating equipment above grade level.

2. Installing safety related equipment on pedestals or providing curbs.

3. Use of floor drainage systems including sumps and sump pumps.

4. Water level alarms.

5. Operator presence and actions.

6. Use of closed doors with water-tight sills. The doors are maintained closed by administrative procedures.

7. Blocked pipe trenches.

8. Encasement of piping.

5F.1.3 SPECIFIC PLANT AREAS

5F.1.3.1 DIESEL GENERATOR ROOMS

All potential flooding sources previously identified in the Unit 3 Diesel Generator Rooms (1-1/2 inch service water piping) meet the seismic design requirements of FSAR Appendix 5A for Category I structures, systems and components. Since no other non-Category I (seismic) flooding sources were identified for these EDG rooms, flooding of these rooms due to failure of non-Category I (seismic) systems is precluded.

The Unit 4 EDG rooms contain 3/4-inch diameter demineralized water lines which fall outside the pipe rupture criteria for NUREG-0800, Standard Review Plan (SRP) Section 3.6.1, for piping failures in fluid systems. The automatic fire suppression systems in these rooms are preaction sprinkler systems in which no water enters the piping network until the fire detectors cause the preaction valve at the fire main to open. There are no other major sources of water in the rooms. Internal flooding in the Unit 4 EDG Building is only postulated for the diesel oil transfer pump rooms which have wet pipe fire suppression systems. In case either room is flooded, credit is taken for the redundant pump in the other room.

5F.1.3.2 RESIDUAL HEAT REMOVAL (RHR) PUMP ROOMS

The RHR pump rooms, located below grade elevation in the Auxiliary Building, could be subject to flooding should a fire protection system pipe break occur. The pump rooms contain sump level alarms, which are powered from a vital source (Reference 12) and annunciate in the control room to notify the operator of an abnormal condition in the room. In addition, each pump room is equipped with a sump and automatic pumping system. The motors of the pumps and valves are positioned at least 30 inches above the floor. Water entering the rooms from a rupture of piping would be pumped out, or the alarm would be received in time for the operator to take action before serious flooding could occur.

5F.1.3.3 SWITCHGEAR ROOMS

The 4160-volt switchgear rooms are located at grade elevation in the Turbine Building. These rooms are subject to several potential sources of flooding.

The first possible source of flooding is rainwater backing up through floor drain pipes and entering the switchgear rooms by seeping under the doors during a heavy rain storm. The floor drain piping has been blocked off and two sumps were installed in each of the switchgear rooms. Each sump is equipped with a high-water level alarm which is powered from a vital source and a sump pump which would automatically begin to pump out any water flooding the rooms. In addition, grating-covered drains have been installed outside the switchgear rooms in front of the main door to preclude rainwater from entering under the doors.

The second possible source of flooding is a circulating water system piping rupture which would flood the condenser pit and possibly overflow into the switchgear area. The water from this overflow would flow into the grating-covered drains in front of the main doors and run off to area storm drains.

The third possible source of flooding is steam and water pipes which pass (or passed) through the switchgear rooms. Several small, low-energy cooling water pipes for the generator exciters and priming air ejector coolers are encased in sheet steel boxes so that any leakage will run outside the switchgear rooms through the drain pipes located at the bottom of the sheet metal boxes. The Units 3 and 4 Priming Piping has been abandoned in place and capped at each end outside the switchgear room. C28

The Units 3 and 4 swing 4160 volt switchgear rooms and Unit 4 EDG control room, located on the second floor of the Unit 4 EDG Building, are not subject to internal flooding.

5F.1.3.4 SAFETY INJECTION PUMPS

The high pressure safety injection pumps are located at grade elevation in a separate compartment which does not contain any non-Category I (seismic) systems which could flood the pumps. Pipe trenches leading to or from the room are blocked to prevent water flow into the room.

5F.1.3.5 MOTOR CONTROL CENTERS

All safety-related Motor Control Centers (MCCs) are located at grade elevation, are physically separated from each other, and are mounted on pedestals above floor elevation.

5F.1.3.6 CHARGING PUMP ROOMS, CONTAINMENT SPRAY PUMP ROOMS, AND BORIC ACID TRANSFER PUMP ROOMS

The charging pump rooms, containment spray pump rooms, and the boric acid transfer pump room are located at grade elevation. For flooding to occur in any of these rooms, it would be necessary for the entire Auxiliary Building below grade elevation to be flooded.

Such flooding would be detected by the plant equipment operator in the Auxiliary Building sufficiently early to isolate the source of flooding. In addition, the charging pump motors and the containment spray and boric acid transfer pump motors are mounted on pedestals above the floor elevation.

5F.1.3.7 COMPONENT COOLING WATER (CCW) PUMPS

The CCW pumps are located at grade elevation in an outdoor area. Any piping failure in this area would result in the water flowing to yard storm drains. Pipe trenches leading from the CCW system rooms are blocked to prevent water flow into the Auxiliary Building.

5F.1.3.8 AUXILIARY FEEDWATER (AFW) PUMPS

The AFW pumps are located outside the Auxiliary Building at grade elevation. The pumps and controls are elevated above the grade and any water in this area will run off on the ground to the discharge or intake canal.

5F.1.3.9 CONTROL ROOM, REACTOR PROTECTION EQUIPMENT ROOMS, ELECTRICAL EQUIPMENT ROOM AND BATTERY ROOMS

The control room, the reactor protection equipment rooms, and the battery rooms are located above grade level and have service water piping which passes through the rooms or through adjoining non-water-tight rooms. The reactor protection equipment rooms and the battery rooms have floor drains which connect to the yard storm drain system. Because the service water lines are small (less than 1 inch in diameter), the drainage system and operator action to isolate ruptures in the piping would be sufficient to protect the rooms from flooding should a rupture in one of these lines occur.

There are no internal flooding concerns for the Spare Battery Room which is located on the east side of the new Electrical Equipment Room, or for the new Electrical Equipment Room. These rooms are located in the Auxiliary Building. The chilled water system in the area does not contain a sufficient volume of liquid to have any significant impact on the internal flooding analysis of the Auxiliary Building. The fire protection system has been seismically analyzed and designed, while piping associated with an eyewash station has been seismically supported.

5F.2 PC/M REVIEW FOR INTERNAL FLOODING

In response to INPO Significant Event Report 50-84 (and later INPO Significant Operating Experience Report 85-5), which described a number of incidents at operating plants involving internal plant flooding, a review of selected modifications was initiated to determine whether they resulted in any adverse affect on any previous flooding evaluations (i.e. SER). The scope of this review included a determination of whether these selected modifications involved any changes to sources of potential flooding, flood protection and mitigating features, and location of safety-related equipment. The results of this review determined that one of the modifications deleted a flood mitigating feature in the Unit 3 Emergency Diesel Generator (EDG) rooms, for which credit was taken in the SER.

To address this concern, a pipe stress analysis and associated support evaluation was performed for the potential flooding source identified in the SER for the Unit 3 EDG rooms (i.e. 1-1/2 inch service water piping). The results of the stress analysis and support evaluation indicate that the subject piping system meets the seismic design requirements of UFSAR Appendix 5A for Class I systems, with the exception of one support. The pipe stress analysis also determined that the 1-1/2 inch service water pipe stresses were within FSAR allowables when failure of this support is assumed. Therefore, potential internal flooding concerns for the Unit 3 EDG rooms are precluded.

5F.3 REFERENCES

1. Letter, FPL (R. Uhrig) to AEC (G Lear), January 7, 1975.
2. AEC, Proposed General Design Criteria, July 10, 1967.
3. Letter, AEC (R. DeYoung) to FPL (J. Coughlin), September 26, 1972.
4. Letter, FPL (J. Coughlin) to AEC (R. DeYoung), November 6, 1972.
5. Letter, FPL (J. Coughlin) to AEC (R. DeYoung), January 5, 1973.
6. Letter, AEC (G. Lear) to FPL (R. Uhrig), December 5, 1974.
7. Letter, FPL (R. Uhrig) to NRC (G. Lear), August 18, 1975.
8. Letter, NRC (A. Schwencer) to FPL (R. Uhrig), September 4, 1979.
9. Letter, AEC (A. Giambusso) to FPL (J. Coughlin), December 18, 1972.
10. Letter, AEC (K. Kniel) to FPL (J. Coughlin), January 24, 1973.
11. Letter, FPL (J. Coughlin) to AEC (A. Giambusso), June 21, 1973.
12. Turkey Point Flooding Review, telecopy response to NRC questions, G.D. Whittier of FPL to M. Grotenhuis, NRC (March 6, 1979).

APPENDIX 5G

EXTERNAL FLOOD PROTECTION
FOR
TURKEY POINT

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5G.1 GENERAL

This Appendix describes the design basis and flood protection features provided at Turkey Point Units 3 and 4 for protection against the effects of an external flood.

5G.2 BACKGROUND

The Turkey Point initial licensee application was based on a design basis of +20 feet above Mean Low Water (MLW) for hurricane wave run-up and +8 feet above MLW for buoyancy based upon the flood protection for Turkey Point Units 1 and 2. This flood criteria was considered by the Army Corps of Engineers to be adequate for a 100 year hurricane flood tide (Reference 1). References 2 and 3 required additional evaluation for the hurricane flood protection requirements for the Turkey Point Site. Additional analysis and model testing were performed. A summary of the results of the analysis and testing was presented to the AEC in Supplement No. 13 of the PSAR (Reference 1).

5G.3 MAXIMUM DESIGN WATER STAGE

The predicted maximum flood stage resulting from the maximum probable hurricane has been calculated to be 18.3 feet above MLW. This was based on postulating that the maximum probable hurricane hovers at the most critical position in proximity to the site long enough to establish steady state conditions (Reference 1). Model tests were done at the University of California (Reference 1) to obtain information on possible flooding of the cooling pumps on the intake structure as a result of 8.7 foot high waves with periods of 6.8 and 8.5 seconds occurring with a water stage of +18.3 feet above MLW.

Based on the conclusions derived from the analysis and model testing, the following actions were taken:

1. A 4 ft. high concrete wall was provided at the seaward extremety (east side) of the intake structure deck. The wall provides flood protection to +20 feet above MLW.

2. A 2 ft. high opening was provided along the east wall of the intake structure between elevations +11 and +13 ft.

3. The intake cooling water pump motor bases were raised from +20 feet MLW to +22.5 feet MLW, and are therefore protected by their elevation.

4. The concrete intake structure deck has been designed for an uplift pressure of 500 lbs/sq. ft., and the overhanging lip of the intake for an uplift pressure of 1000 lbs/sq. ft. These pressures are created by wave surge.

5. External flood protection has been provided to +20 ft. above MLW to the west of the facility and to +22ft. above MLW to the north and south of the facility (Reference 5) by a continuous barrier consisting of building exterior walls, flood walls, a flood embankment, and stop logs for the door openings as shown in Figure 5G-1. C32

6. External flood protection has been provided to +22 ft. above MLW to the east of the facility by a continuous barrier consisting of building exterior walls and stop logs for the door openings as shown in Figure 5G-1. Elevation +22 ft. provides protection from maximum wave run-up.

5G.4 FLOODING FROM RAIN

Flooding from rain water is prevented by an elaborate system of storm drains, catch basins, and sump pumps. Design changes to the Unit 3 Diesel Oil Storage Tank secondary containment area (Reference 4) created a catch basin that could flood during a design basis rain. Manual reach rod bypass valves were added to ensure capability of fuel oil transfer during a design basis rain event should the normal method with solenoid operated valves be incapacitated due to immersion in the rain water. All outdoor equipment is designed for such service.

5G.5 SUMMARY

The two nuclear units have been constructed on compacted limerock fill to elevation +18 ft. MLW, and flood protected to an elevation of +20 ft. MLW, west of the facility, which is well above any experienced or predictable flood tides. Components vital to safety, with the exception of the ICW pumps, are protected against flood tides, and wave runup, to +22 ft. MLW on the north, south, and east side of the units. The ICW pump motors are protected to +22.5 ft. MLW. C32 C32

Post-Fukushima flood protection improvements were implemented to ensure adequate Available Physical Margin (APM) and reliability of flood protection features credited for levels during a Probable Maximum Storm Surge (PMSS). (References 5, 6, 7, 8). C32

5G.6 REFERENCES

1. Supplement No. 13 to application for licenses of the Turkey Point PSAR, December 11, 1967, Turkey Point Units 3 and 4 Hurricane Flood Protection Criteria Additional Information.

2. Letter, ACRS to FPL, January 18, 1967.

3. Safety Evaluation Report by the AEC Division of Reactor Licensing, February 8, 1967.

4. PC/M 04-036, "Oil Spill Prevention and Control Compliance for 40CFR112"

5. EC294356, Flood Protection Improvements C32

6. Letter L-2019-125, Turkey Point Unit 3 and Unit 4, Flood Protection Features Planned Changes Implementing Plan Commitment Update Notification, FPL to NRC, June 27, 2019, ADAMS Accession Number ML19204A179 C32

7. FPL Letter, L-2017-124, Flooding Focused Evaluation Summary, dated June 29, 2017, ADAMS Accession Number ML17212B180 C32

8. FPL Letter L-2020-152, Flooding Focused Evaluation-Summary Commitment Revisions for Flooding Protection Features Planned Changes, dated October 26, 2020, ADAMS Accession Number ML20301A899 C32

FINAL SAFETY ANALYSIS REPORT

FIGURE 5G-1

REFER TO ENGINEERING DRAWING

5610-C-1695 , SHEET 1

REV. 13 (10/96)

FLORIDA POWER & LIGHT COMPANY

TURKEY POINT PLANT UNITS 3 & 4

EXTERNAL FLOOD PROTECTION
FLOOD PROTECTION BARRIERS
PLANT ARRANGEMENT

FIGURE 5G-1

FINAL SAFETY ANALYSIS REPORT

FIGURE 5G-2

REFER TO ENGINEERING DRAWING

5610-C-1695 , SHEET 2

REV. 13 (10/96)

FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

EXTERNAL FLOOD PROTECTION
PERIMETER FLOOD WALL DETAILS

FIGURE 5G-2

APPENDIX 5H

TURKEY POINT PLANT UNITS 3 AND 4

1994 CONTAINMENT STRUCTURE RE-ANALYSIS

FLORIDA POWER AND LIGHT COMPANY

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APPENDIX 5H

1994 CONTAINMENT STRUCTURE RE-ANALYSIS

5H.1 GENERAL

This appendix documents the results of the containment re-analysis relative to the determination of the minimum prestressing requirements for each tendon group. The containment re-analysis was completed in 1994.

5H.2 BACKGROUND

The tendon surveillance programs for the Turkey Point Units 3 and 4 containment structure post-tensioning systems have been performed at one, three, and five years after the containment Initial Structural Integrity Test (ISIT), and every five years thereafter. During the performance of the 20th year tendon surveillance, the measured normalized lift-off forces for a number of randomly selected surveillance tendons (two of five hoop tendons, one of three dome tendons, and one of four vertical tendons in Unit 4, and four of five hoop tendons in Unit 3) were below the predicted lower limit (PLL). In accordance with the recommendations of the Turkey Point Plant Technical Specifications and to further investigate the extent and probable cause of the low lift-off conditions, additional lift-off measurements on adjacent tendons were taken. The measured lift-off force in the adjacent tendons (with the exception of one dome tendon and two vertical tendons in Unit 4) were also found to be below the PLL. Consequently, in accordance with the Turkey Point Plant Technical Specifications, engineering evaluations (References 1 through 6) were prepared to address the low lift-off conditions.

References 3 and 6 evaluated the low lift-off forces and concluded that the most probable cause for the low lift-off forces measured during the 20th year tendon surveillances were due to an increased tendon wire steel relaxation loss caused by average tendon temperatures higher than originally considered. Considering this higher steel relaxation loss rate, these evaluations also concluded that the Units 3 and 4 containment post-tensioning systems will provide sufficient prestress force to maintain Turkey Point licensing basis requirements at least through the 25th year tendon surveillance. These evaluations also recommended a structural re-analysis of the containment structure and the post-tensioning system to determine a new minimum required prestress force and to establish the time period that the containment post-tensioning system will provide sufficient prestress force to maintain the Turkey Point licensing basis requirements.

5H.2.1 ORIGINAL CONTAINMENT ANALYSIS

The original containment structural analysis results are documented in Updated Final Safety Analysis Report (UFSAR), Section 5.0.

In the original containment analysis/design, the containment base slab was designed as a conventional reinforced concrete structure. The containment re-analysis as described in this appendix does not include a new evaluation of the base slab since the base slab is not affected by the post-tensioning system. However, the base slab was included in the containment re-analysis model to provide a realistic boundary condition for the model. Therefore, the original base slab design/analysis, as summarized in UFSAR, Sections 5.1.3, 5.1.4 and Table 5.1.4-1, Sheet 6 remains unchanged. In addition, certain load conditions (e.g., initial prestressing and initial structural integrity test condition) and evaluations (e.g., buttress anchorage zone stress evaluation) were not included in the 1994 re-analysis. The UFSAR, Section 5.0 has been annotated in all areas where the 1994 re-analysis has modified the original analysis.

5H.3 CONTAINMENT RE-ANALYSIS

5H.3.1 MODEL DESCRIPTION

A three dimensional (3-D) finite element model using Bechtel's Structural Analysis Program (BSAP), is used for the re-analysis of the containment structure. The 3-D model consists of the cylindrical wall (including buttresses), ring girder, dome, base slab, and the major penetrations (equipment hatch and the personnel hatch). A plate element is used in the 3-D model to represent the shell (including buttresses and major penetrations), dome, ring girder, and the base slab. This element is a thin quadrilateral and/or triangular element that has both membrane and bending properties. The formulation of this element is based on the thin shell and small deflection theory. The base slab is modeled as a circular foundation including a central hole with appropriate boundary conditions representing the centerline of the reactor pit walls. The soil-structure interaction is accounted for by introducing the soil springs at each node of the base slab. Refer to Figures 5H-1 through 5H-4 for the geometric plots of the 3-D model. The development of the 3-D model is documented in Reference 7.

5H.3.2 MATERIAL PROPERTIES

The material properties used in the 3-D model are as follows:

1. Modulus of Elasticity of Concrete (Ec) = 1.5 x 10^6 psi
2. Concrete Poisson's Ratio = 0.17
3. Coefficient of Thermal Expansion (α_c)=5.0 x 10^{-6} per °F

These values are consistent with the information included on page 5.1.3-2 which were used in the original design basis analysis of the Turkey Point containment structure.

The soil properties are based on the 1988 seismic survey conducted at the Turkey Point site for the EDG enhancement project (Reference 19). The properties for each soil layer used in the re-analysis are as follows:

SOIL LAYER	POISSON's RATIO	SHEAR MODULUS
Limerock Fill	0.256	7,380 ksf
Miami Oolite	0.253	18,620 ksf

For the Fort Thompson formation, consistent with the original analysis, 0.22 and 4 x 10^6 psi was used for the Poisson's ratio and the Young's modulus, respectively.

Detailed explanation of the 3-D model material properties is documented in Reference 7.

5H.3.3 DESIGN LOADS AND LOAD COMBINATIONS

The design loads and the load combinations used in the re-analysis of the containment structure are in accordance with the requirements of Appendix 5B "Containment Structure Design Criteria". All load combinations included in Appendix 5B for the design load and the yield conditions have been evaluated in the re-analysis of the containment structure. Reference 12 documents the load conditions and the load combinations that have been considered in the analysis.

5H.3.4 METHOD OF ANALYSIS & STRESS ALLOWABLES

The working stress method (elastic analysis) is applied to the load combinations for design load, as well as yield load conditions. The design assumption of straight line variation of stresses is maintained under yield conditions. This method of analysis is consistent with the original design basis for the Unit 3 and Unit 4 containment structures as outlined in Appendix 5B.

The stress allowables used for evaluation of the critical sections of the containment structures are in accordance with Appendix 5B. This is documented in Reference 15.

5H.3.5 BASELINE ANALYSIS

A baseline analysis was performed to demonstrate correlation between the results of the 1994 3-D BSAP finite element analysis and the original Turkey Point containment axisymmetric analysis (Refer to Page 5.1.3-1). The results of the baseline analysis demonstrate good correlation between the 1994 BSAP 3-D analysis and the original axisymmetric analysis specified in the isostress plots in the UFSAR, Section 5.0. In addition, the baseline analysis for the pressure load case was compared to classical closed form solutions with good correlation. It was concluded that the 3-D finite element model accurately predicts the state of stress in the containment structures. The baseline analysis has been documented in Reference 10.

5H.3.6 THERMAL CRACK ANALYSIS

As stated in Section 5.1.3.1, the thermal loading used in the original design was based on Figure 5.1-8 "Design Thermal Gradient Across Containment Wall". Also, as stated in Page 5.1.3-3, a temperature of 283°F was used for liner plate in the original design. The thermal loading used in the re-analysis of the containment structure is consistent with the original criteria. In addition, the occurrence of a higher containment bulk temperature (i.e., from 120°F to 125°F) as stated in Pages 14.3.4-16 and 14.3.4-22 has been considered in the re-analysis. The thermal loading for the 3-D model is documented in Reference 8.

Consistent with the original analysis, the thermal crack analysis outlined on Pages 5.1.3-7 through 5.1.3-9 has been used to determine the stresses in reinforcing steel and concrete due to thermal loading. This method of analysis is based on the equilibrium of normal forces acting on the section under consideration. The concrete and reinforcing steel stresses from the primary loads are added to the thermal stresses to determine the total stresses.

For load combination 1.05D+F+1.5P+Ta, an additional refined thermal crack analysis has been performed for the critical mid-height section of the shell to determine the effects of thermal loading and concrete cracking on the overall state of stress in the shell. The ALGOR SuperSap computer program is used in the refined thermal crack analysis. The finite element analysis used a two dimensional (2-D) model which includes a section of the shell halfway between the adjacent buttresses (a 60° segment of the containment). This 2-D model is primarily used to capture the behavior of the shell in the hoop direction. Two models, one with and one without buttresses, were used to study the effects of the buttress in cracking analysis. There are 10 layers of elements representing concrete thickness in the shell area. In addition, there is one element representing the liner plate. The reinforcing steel and the hoop tendons are also modeled as truss elements. By modeling the hoop tendons, the effects of pressurization (increase in tendon force due to internal pressure loading) is directly captured. Roller type boundary conditions have been used for this model to allow the boundary nodes to displace in the radial direction. The modeling and the method of analysis are documented in Reference 14. Figure 5H-5 depicts the finite element models used in the refined thermal cracking analysis.

The cracking of the concrete is established by the criterion in Appendix 5B which states that the principal concrete tension due to combined membrane tension, membrane shear, and flexural tension due to bending moments or thermal gradients is limited to $6(f'c)^{0.5}$. The cracking of the concrete is accomplished by introducing a very small modulus of elasticity in the hoop direction. The cracking analysis is carried out in successive analyses as follows:

1. The first analysis considers an uncracked concrete condition. In this analysis, the concrete elements with stresses in the hoop direction exceeding the Appendix 5B limit are considered cracked.

2. The second analysis includes the material properties for the cracked elements determined in the first cycle.

Based on the results of the first analysis, the second analysis was performed with all layers of concrete cracked. The reinforcing steel and tendon stresses, and the liner strain were found to be within the established UFSAR allowable limits. The results of the refined thermal crack analysis are documented in Reference 14.

5H.3.7 MAJOR PENETRATIONS EVALUATION

The 3-D finite element model includes a refined mesh at the equipment hatch and the personnel hatch locations to capture the behavior of the shell in the vicinity of these large penetrations. By modeling the penetrations in the 3-D model, the need for a local model and defining the boundary conditions and the loads at the boundaries of the local model is eliminated. Also, the effects of the shell curvature will be captured.

The thickened shell at the equipment hatch area has been taken into account by specifying the appropriate element thicknesses. In addition, the deflection of the hoop and vertical tendons around the equipment hatch and the personnel
hatch has been considered in the modeling by applying the appropriate nodal loads and element pressure loads.

The modeling of the equipment hatch and the personnel hatch is documented in References 16 and 18, respectively. Refer to Figures 5H-3 and 5H-4 for the geometric plots of the finite element mesh for the equipment and personnel hatches.

5H.4 SUMMARY OF RESULTS

Tables 5H-1A and 5H-1B include the most critical reinforcing steel stress summary as a result of the containment re-analysis. The information presented in these tables are given for representative elements in the 3-D model away from the major penetrations. The elements range from the base/shell junction to the vicinity of dome apex as shown in Figure 5H-2. The results are tabulated for all design and yield loading combinations stated in Appendix 5B. The reinforcing steel and concrete stresses, and liner strains were found acceptable for all design basis loading conditions.

In addition, the stresses in reinforcing steel and concrete, and liner plate strains in the localized areas around the major penetrations were found acceptable for all design basis loading conditions.

These results are based on the following final minimum required average prestress forces:

1. Hoop Prestress Force = 590 kips/ft
2. Dome Prestress Force = 313 kips/ft
3. Vertical Prestress Force = 250 kips/ft

The tendon forces and tendon wire forces (based on a 90 wire tendon) corresponding to these average prestress values are as follows:

TENDON GROUP	FINAL REQUIRED AVERAGE PRESTRESS FORCE (kips/ft)	TENDON FORCE (kips/tendon)	WIRE FORCE (kips/wire)
HOOP	590	491.6	5.46
DOME	313	531	5.90
VERTICAL	250	522	5.80

The methodology and results of the 1994 containment structure re-analysis are documented in Reference 17.

5H.5 REFERENCES

1. Engineering Evaluation JPN-PTN-SECJ-92-019, "Unit 3 20th Year Tendon Surveillance Hoop Tendons Low Lift-Off Force", Revision 1

2. Engineering Evaluation JPN-PTN-SECJ-92-023, "Unit 3 20th Year Tendon Surveillance Low Lift-Off Force on Hoop Tendon 42H32", Revision 0

3. Engineering Evaluation JPN-PTN-SECJ-92-024, "Unit 3 20th Year Tendon Surveillance, Extent and Probable Cause of Low Lift-Off Force on Hoop Tendons", Revision 0

4. Engineering Evaluation JPN-PTN-SECJ-92-039, "Unit 4 20th Year Tendon Surveillance - Low Lift-Off Force on Hoop Tendon 13H54 and Dome Tendon 1D40", Revision 0

5. Engineering Evaluation JPN-PTN-SECJ-92-041, "Unit 4 20th Year Tendon Surveillance - Low Lift-Off Force on Hoop Tendon 35H38", Revision 0

6. Engineering Evaluation JPN-PTN-SECJ-92-042, "Unit 4 20th Year Tendon Surveillance, Extent and Probable Cause of Low Lift-Off Force on Hoop and Dome Tendons", Revision 0

7. Calculation No. C-SJ599-01, "3-D Finite Element Model for Turkey Point Containment Building"

8. Calculation No. C-SJ599-02, "Determination of Containment Thermal Loading for Input into BSAP Finite Element Computer Program"

9. Calculation No. C-SJ599-03, "Determination of Prestress Loads on the Containment Structure for Input into BSAP Finite Element Computer Program"

10. Calculation No. C-SJ599-04, "Baseline Analysis of Turkey Point Containment Building"

5H.5 REFERENCES (Continued)

11. Calculation No. C-SJ599-05, "Software Modifications and Calculation of Prestress Loads on Containment Shell"

12. Calculation No. C-SJ599-06, "Design Loads and Load Combinations for Turkey Point Containment Structure Re-Analysis"

13. Calculation No. C-SJ599-07, "Confirmatory Analysis of Turkey Point Containment Structure for Load Case 1.05D+1.5P+F+Ta"

14. Calculation No. C-SJ599-08, "Refined Thermal Crack Analysis for Containment Shell"

15. Calculation No. C-SJ599-09, "Stress Allowables for Analysis of Turkey Point Containment Structure"

16. Calculation No. C-SJ599-10, "Turkey Point Containment Structure - Equipment Hatch BSAP Modeling and Loading

17. Calculation No. C-SJ599-11, "Turkey Point Containment Structure Final Analysis Results for all Load Combinations"

18. Calculation No. C-SJ599-13, "Turkey Point Containment Structure - Personnel Hatch and Thrust Beam Area BSAP Model and Loading"

19. Geotechnical Investigations and Foundation Analysis for Diesel Building Addition, Report No. FLO 53-20E.5000, Revision 0

TABLE 5H-1A

REBAR STRESS SUMMARY

MOST CRITICAL STRESSES AT REPRESENTATIVE SECTION OF CONTAINMENT

FOR LOAD COMBINATIONS 1, 2, OR 3 (WSD)

(BSAP ANALYSIS, WITH POST - PROCESSING)

Working Stress Design (WSD) Load Combinations are in accordance with Section B.1.5 of Appendix 5B:
(1) D + F + L + To
(2) D + F + L + P + Ta + E
(3) D + F + L + 1.15P

Element Number	MOST CRITICAL REBAR STRESSES Larger of (Primary) or (Thermal + Primary) Evaluations: Hoop Rebar (XX) (Inside Face)	Hoop Rebar (XX) (Outside Face)	Meridional Rebar (YY) (Inside Face)	Meridional Rebar (YY) (Outside Face)
11	NOT-CRITICAL	NOT-CRITICAL	-11.590 (1)	27.738 (1)
1733	NOT-CRITICAL	NOT-CRITICAL	-11.034 (1)	21.311 (1)
53	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
95	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
137	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
179	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
221	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
263	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
305	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
347	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
389	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
431	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
473	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
515	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
557	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
599	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
641	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
683	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
725	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
767	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
809	NOT-CRITICAL	NOT-CRITICAL	-0.464 (2)	12.720 (2)
851	NOT-CRITICAL	NOT-CRITICAL	-1.926 (2)	17.200 (2)
893	NOT-CRITICAL	NOT-CRITICAL	1.413 (2)	22.846 (1)
935	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
977	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1019	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1061	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1103	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1145	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1187	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1229	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1271	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1313	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1355	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL

Notes

a) Rebar Stresses for Sections not cracking under any of the Load Combinations are entered as "NOT-CRITICAL"

b) Numbers shown in parentheses with stress entry indicate governing Loading Combination.

REBAR ALLOWABLE STRESSES, ksi	
Rebar size	WSD, LC's 1-3
< #11, Fy=40ksi	20.0
#11 or larger Fy=60 ksi	30.0

TABLE 5H-1B

REBAR STRESS SUMMARY

MOST CRITICAL STRESSES AT REPRESENTATIVE SECTION OF CONTAINMENT

FOR LOAD COMBINATIONS 4, 5, 6, 7, OR 8 (WSD)

(BSAP ANALYSIS, WITH POST - PROCESSING)

Ultimate Strength Design (USD) Load Combinations are in accordance with Section B.1.6 of Appendix 5B:
(4) 1.05D + 1.5P + Ta + F
(5) 1.05D + F + 1.25P + Ta + 1.25E
(6) 1.05D + F + 1.25H + R + To + 1.25E
(7) D + F + P + Ta + H + E'
(8) D + F + H + R + E' + To

	MOST CRITICAL REBAR STRESSES			
	Larger of (Primary) or (Thermal + Primary) Evaluations			
Element	Hoop Rebar (XX)		Meridional Rebar (YY)	
Number	(Inside Face)	(Outside Face)	(Inside Face)	(Outside Face)
11	N/A	17 296 (8)	-18 281 (6)	28 440 (8)
1733	N/A	13 484 (7)	-20.269 (6)	21.825 (8)
53	N/A	20.874	20.868	15 811 (7)
95	N/A	(Note d)	19 635 (7)	17.092 (7)
137	N/A	(Note d)	10.717	25.576
179	N/A	(Note d)	18 902 (7)	26.624
221	N/A	(Note d)	12.325 (7)	24 966
263	N/A	(Note d)	N/A	24 614
305	N/A	(Note d)	N/A	24.060
347	N/A	(Note d)	N/A	24.547
389	N/A	(Note d)	N/A	25.369
431	N/A	(Note d)	N/A	26 195
473	N/A	(Note d)	N/A	27.012
515	N/A	(Note d)	N/A	27.814
557	N/A	28 166	N/A	28.596
599	N/A	28.359	N/A	20.523
641	N/A	29 002	9.222	19.379
683	14 479	28 101	9 329	19 942
725	15 326	29 379	9.026	21.073
767	20 503	35 602	6.299	20 522
809	18 684	34.225	5.328	23 088
851	14 060	29 754	7 243	25 304
893	-12 476 (8)	21 065	11 308	25.225
935	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
977	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1019	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1061	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1103	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL	NOT-CRITICAL
1145	-3 989 (5)	13 188	9 250	19 250
1187	3 805	24 186	8 336	32 918
1229	9 814	29 765	11 170	36 439
1271	13 013	31 061	13 155	34 925
1313	14 364	30 661	14 237	32 473
1355	14 784	30 126	14 829	31 101

Notes

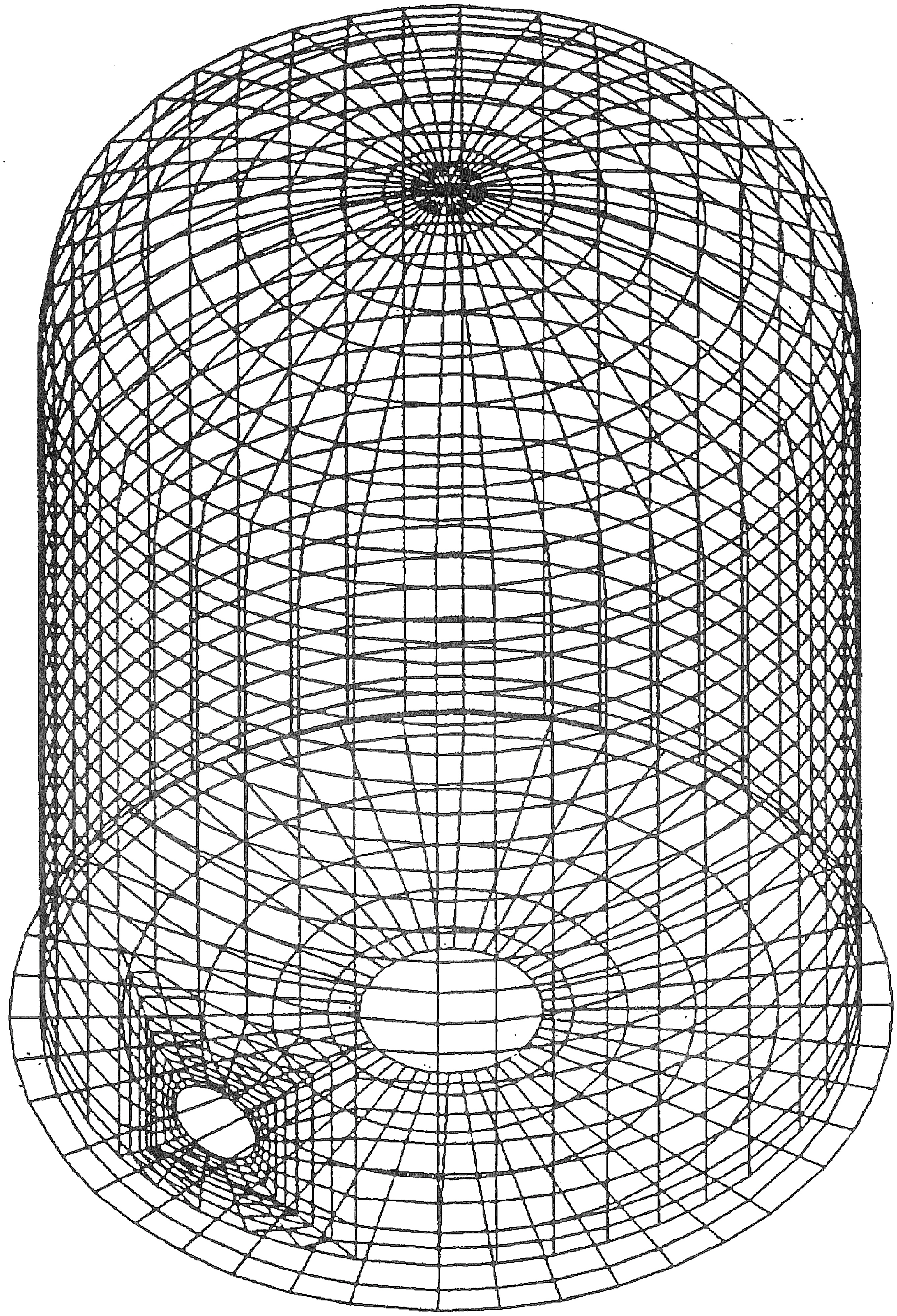
a) Rebar Stresses for Sections not cracking under any of the Load Combinations are entered as "NOT-CRITICAL"

b) Numbers shown in parentheses with stress entry indicate governing Loading Combination. For cases with no load combination shown governing Load Combination is (4)

c) "N/A" entry denotes no inside face rebar exists

d) Maximum hoop outside face rebar stress per refined thermal analysis for this area was determined to be 29 9 ksi

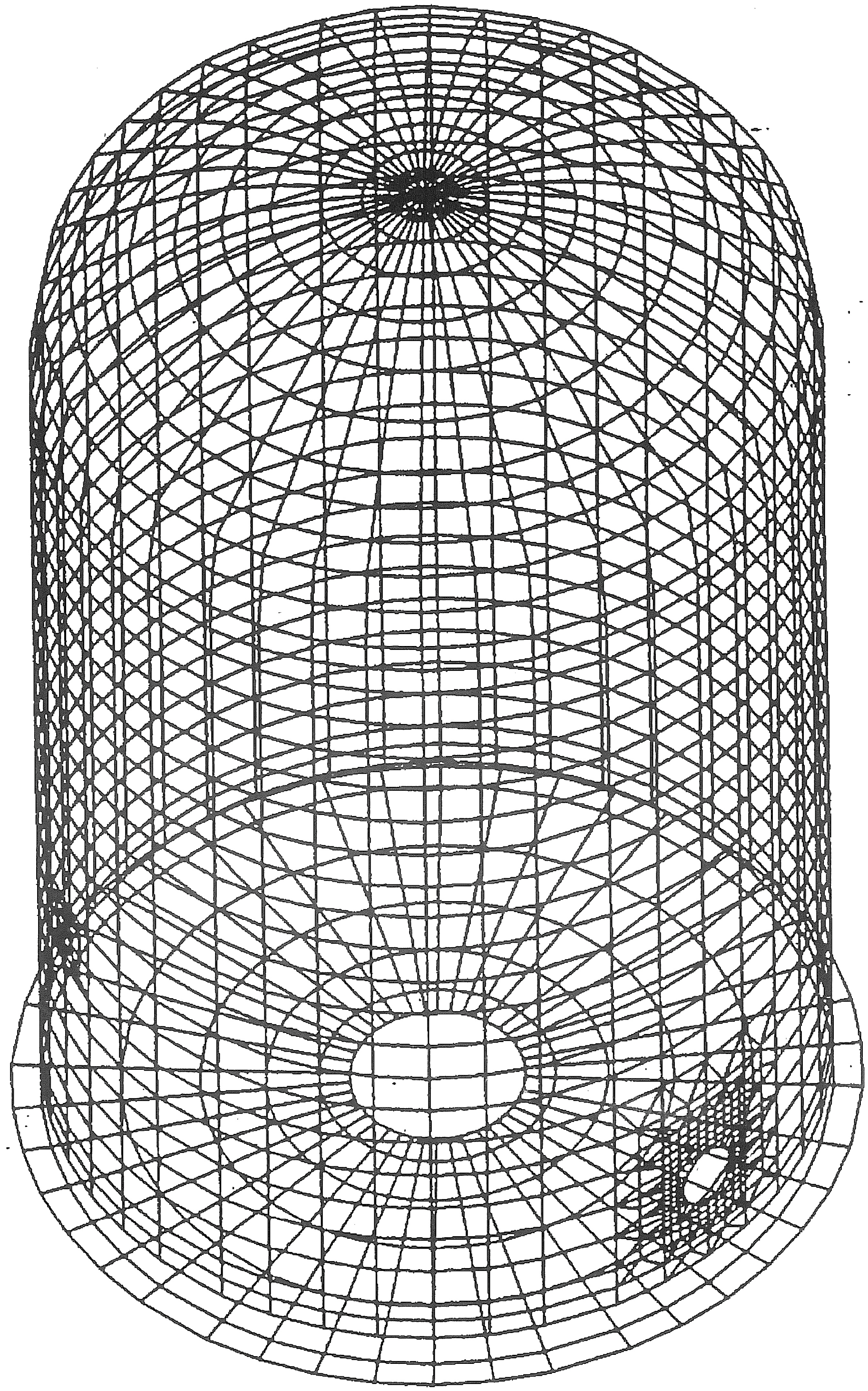
REBAR ALLOWABLE STRESSES, ksi	
Rebar size	USD, LC's 4-8
< #11 Fy=40ksi	36 0
#11 or larger Fy=60 ksi	54 0



REV. 13 (10/96)

FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

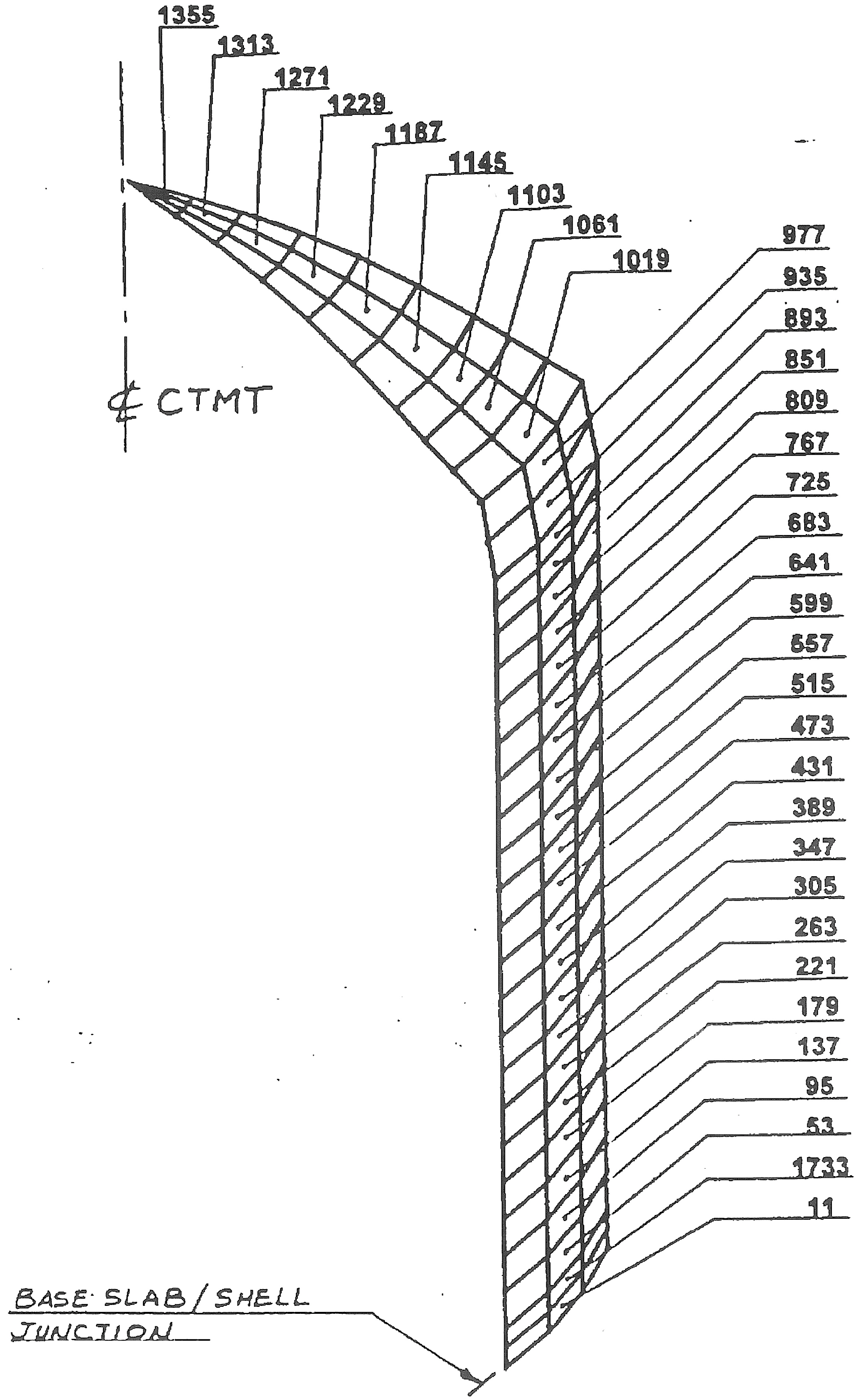
3D FINITE ELEMENT MODEL
INCLUDING
EQUIPMENT HATCH PENETRATION
FIGURE 5H-1A



REV. 13 (10/96)

FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

3D FINITE ELEMENT MODEL
INCLUDING
EQUIPMENT HATCH PENETRATION
FIGURE 5H-1B

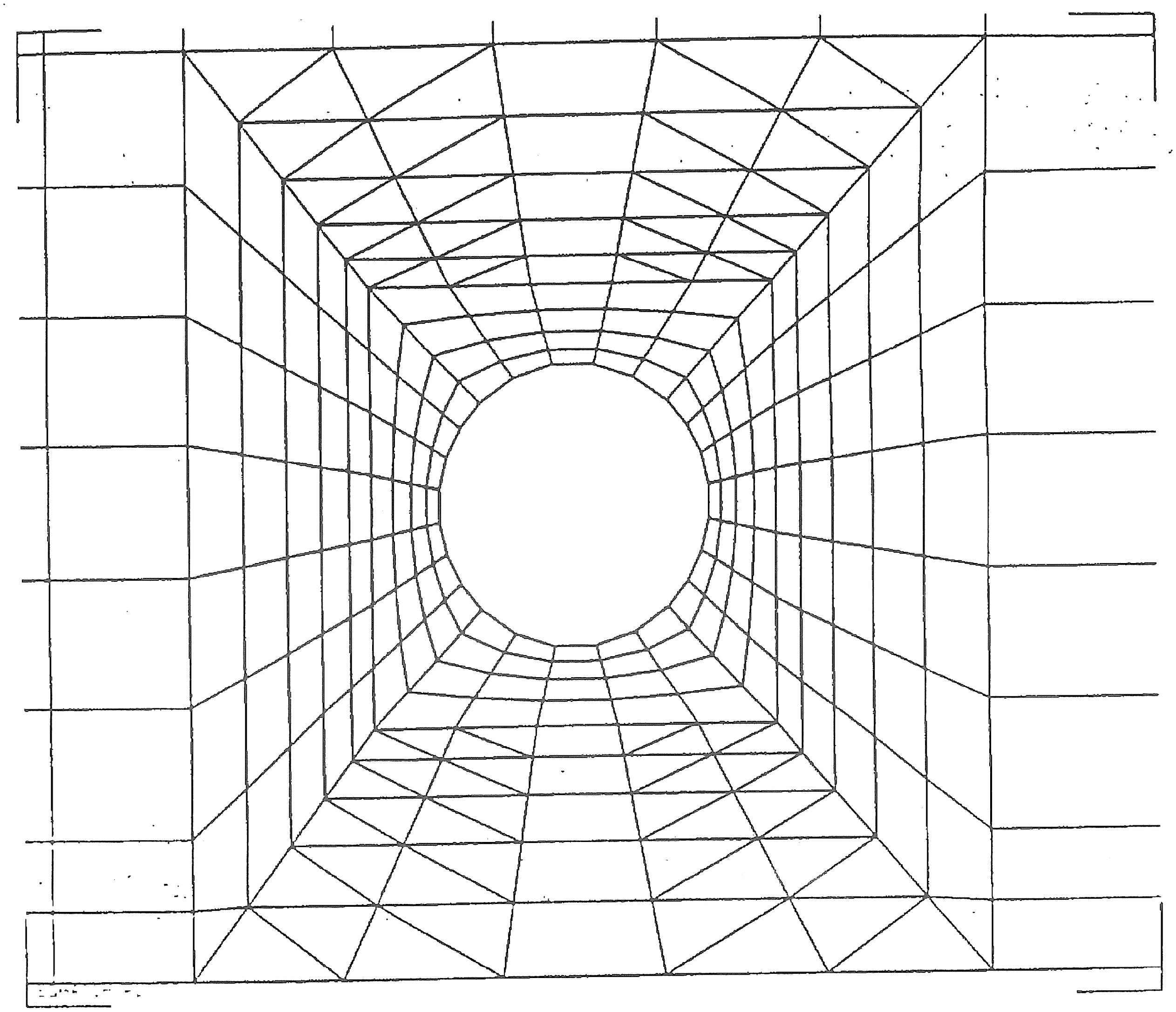


REV. 13 (10/96)

FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

REPRESENTATIVE ELEMENTS
IN 3-D MODEL

FIGURE 5H-2

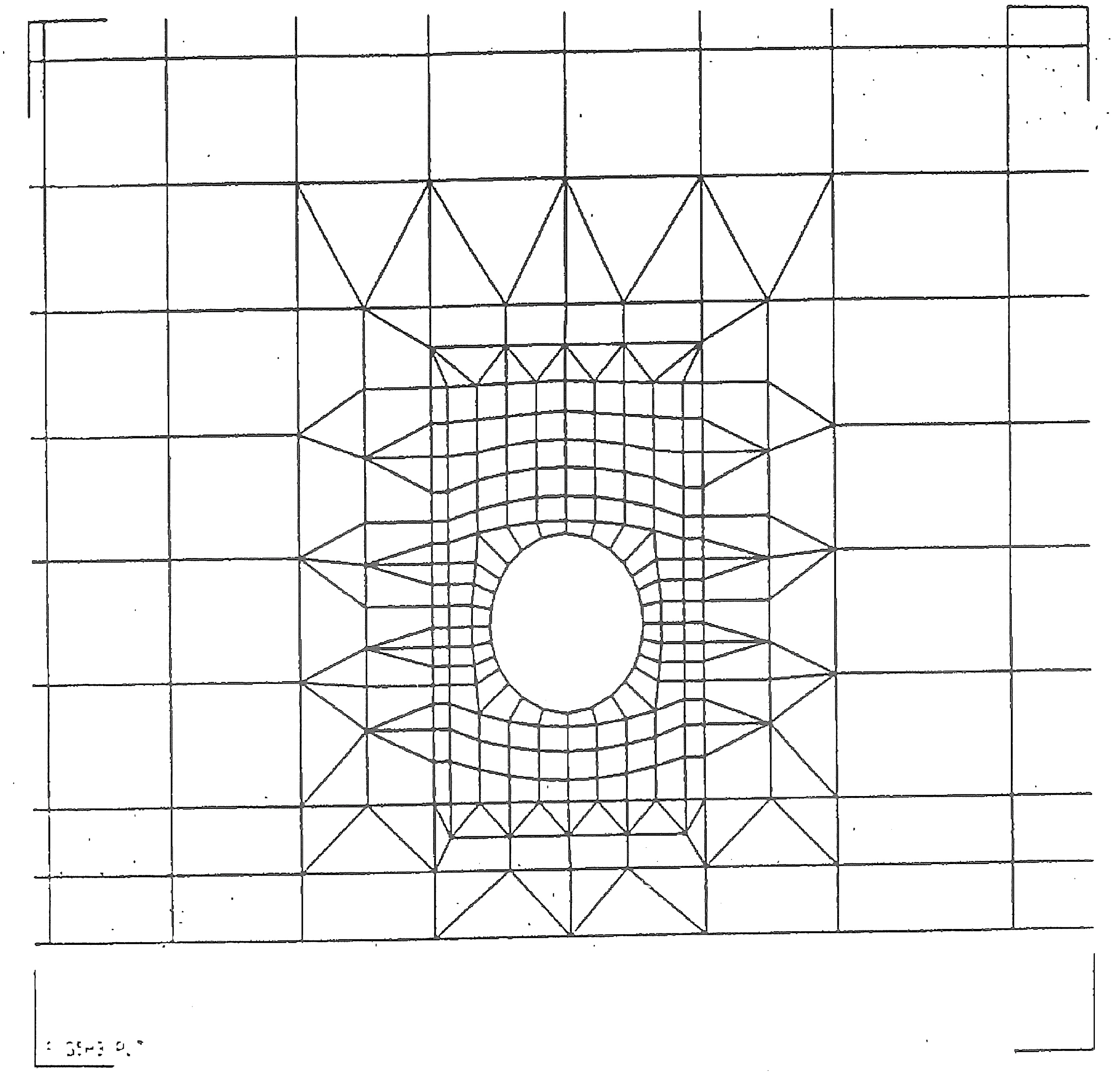


REV. 13 (10/96)

FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

EQUIPMENT HATCH
FINITE ELEMENT MESH

FIGURE 5H-3

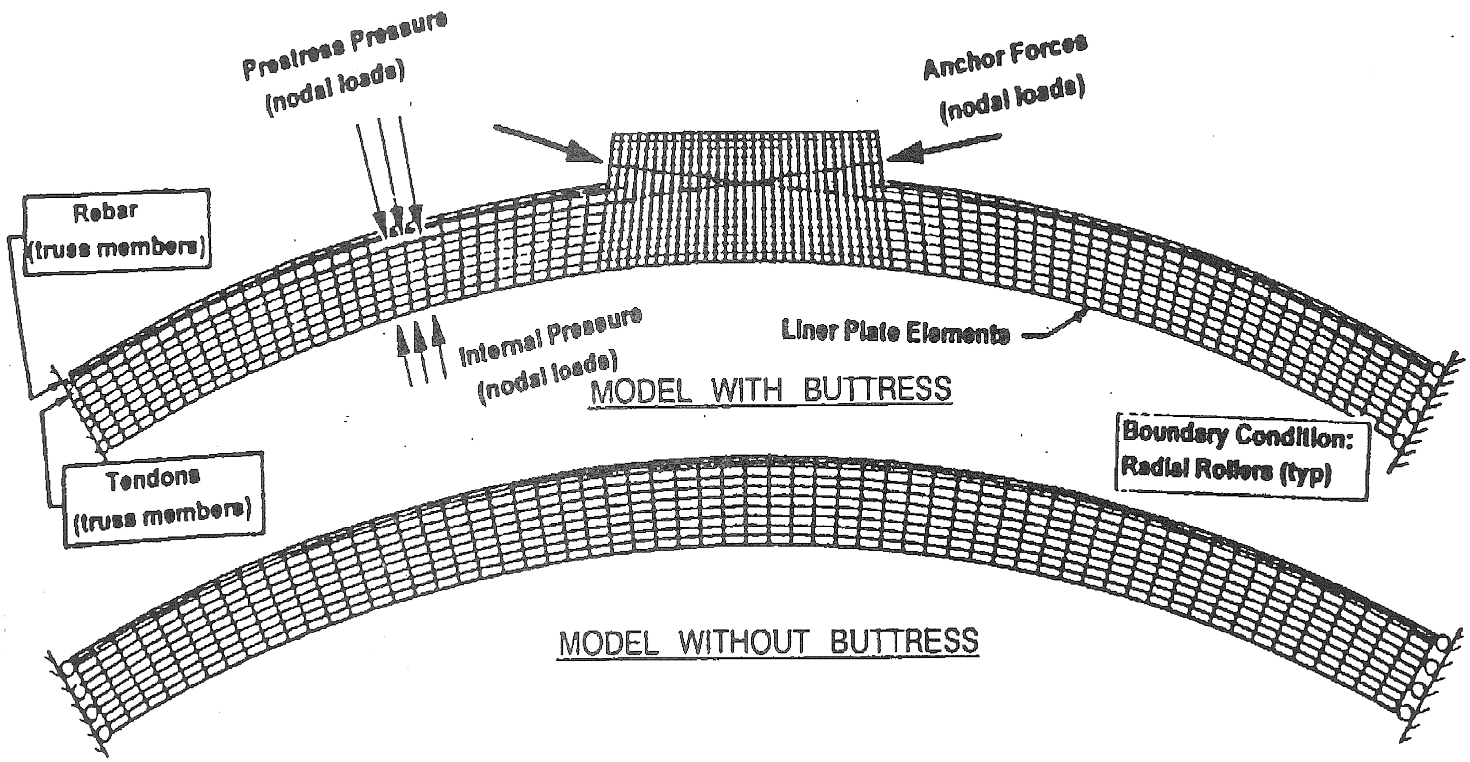


REV. 13 (10/96)

FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

PERSONNEL HATCH
FINITE ELEMENT MESH

FIGURE 5H-4



REV. 13 (10/96)

FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS 3 & 4

REFINED
THERMAL CRACKING ANALYSIS
FINITE ELEMENT MODELS
FIGURE 5H-5

Appendix 5I

Heavy Load Handling

5I.1 GENERAL

This appendix describes the program for the control of heavy loads at Turkey Point Units 3 and 4.

5I.2 BACKGROUND

The controls for handling heavy loads at Turkey Point have been reviewed in accordance with the NRC generic letter on heavy loads, dated December 22, C26
1980 (Reference 1), and have been determined to be in compliance with the applicable requirements of NUREG-0612 (Reference 2). A Safety Evaluation Report (SER) was issued by the NRC to document compliance of the Turkey Point heavy load handling program with Phase I of NUREG 0612 (Reference 3). Phase II of NUREG 0612 was retracted by the NRC as discussed in Generic Letter 85-11 (Reference 4). Based on this generic letter, a Phase II evaluation was not completed for Turkey Point.

Section 5.1.1 of NUREG-0612 identifies seven guidelines related to the design and operation of the overhead load handling systems in the areas where spent fuel is stored, in the vicinity of the reactor core, and in other areas of the plant where a load drop could result in damage to equipment required for safe shutdown or decay heat removal. These guidelines have been incorporated in the Turkey Point heavy load handling program in the following sections.

The spent fuel cask handling crane has been upgraded to the single-failure-proof criteria of NUREG-0554, Single-Failure-Proof Cranes for Nuclear Power Plants (Reference 9). For single-failure-proof lifts, the cask crane must utilize a single-failure-proof handling system. Therefore, the handling system must meet the requirements of NUREG-0612, Section 5.1.6. Single-failure-proof lifts are those deemed reliable enough such that the consequences of a load drop need not be considered. The NRC approved the use of the upgraded cask crane as single-failure-proof in License Amendments 243 and 239 (Reference 21)

5I.3 PROGRAM DESCRIPTION

Heavy Load Handling Systems

A heavy load is defined in NUREG-0612 as a load whose weight is greater than the combined weight of a single spent fuel assembly and its handling tool. At Turkey Point, the combined weight of a spent fuel assembly and the spent fuel handling tool, at the time of the NUREG-0612 evaluation, was equal to 1,760 pounds. Therefore, the definition of a heavy load used for the NUREG-0612 evaluation and defined in plant administrative procedures on heavy load handling is 1,760 lbs. In the spent fuel pool area only, the definition is 2,000 lbs, which accounts for the additional weight of a control rod assembly, and associated handling tool.

Heavy load handling systems have been identified and classified into Groups I and II as follows:

Group I - Overhead handling systems from which a load drop could result in damage to irradiated fuel or systems required for plant shutdown or decay heat removal taking no credit for inter-locks, technical specifications, operating procedures, detailed structural analysis or system redundancy. Group I systems are required to conform to the guidelines of NUREG-0612.

Group II - Overhead handling systems excluded from Group I based upon a determination by inspection that there is sufficient physical separation from any load-impact point and any safety related component. Based on this determination, Group II systems are not required to conform to the guidelines of NUREG-0612.

5I.3.1 Identification of Safe Load Paths for Group I Heavy Load Handling Systems (Guideline 1)

Safe load paths for Group I heavy load handling systems have been identified in administrative procedures. The procedures include safe load path diagrams for the following load handling systems:

1. Two Turbine Gantry Cranes (Units 1 & 2 and Units 3 & 4, sharing rails common to all four units)

2. Two Reactor Building Polar Cranes (Units 3 & 4)

3. Spent Fuel Cask Handling Crane (serves both Units 3 & 4)

4. Intake Structure Bridge Crane (serves both Units 3 & 4)

Group I systems which do not have a designated safe load path either (1) are used during refueling operations and do not handle loads weighing more than one spent fuel assembly with a handling tool, or (2) have procedural controls to prevent adverse interaction with operating safety related systems or spent fuel.

5I.3.2 Load Handling Operations (Guideline 2)

The handling of heavy loads is normally confined to the designated safe load path areas.

Administrative procedures define the requirements for heavy load handling operations and specify the safe load carrying areas to be used by crane operators. Any deviation from the safe load area requires prior written approval from designated plant management personnel. The written approval for the deviation will provide instructions for inspections and acceptance criteria before moving the load, steps and proper sequence to be followed, a load path and other special precautions.

For heavy loads which are periodically handled in the vicinity of irradiated fuel, special procedures have been developed. Special procedures are provided for the following loads:

1. Reactor Missile Shields

2. Reactor Vessel Head

3. Reactor Upper Internals

4. RCP Motor Access Hatches

5. New and Spent Fuel Elements

6. Spent Fuel Bulkhead (Keyway gate) (Unit 4 Only)

5I.3.3 Crane Operator Training (Guideline 3)

Turkey Point crane operators are trained and qualified in accordance with procedures which generally comply with the applicable sections of the 1976 edition ANSI B30.2 (Reference 5) as required by NUREG-0612. Exceptions to ANSI B30.2 are taken for eye tests, securing power to the crane, testing of the upper limit switches, and the manner in which safety is maintained utilizing plant clearance procedures.

5I.3.4 Special Lifting Devices (Guideline 4)

In accordance with Guideline 4 of NUREG-0612, Section 5.1.1, lifting devices are to be in compliance with ANSI N14.6-1978 (Reference 6). Pursuant to this, the following lifting devices were evaluated and found to be consistent with the intent of the ANSI standard:

1. Reactor vessel head lift rig

2. Internals lift rig

3. Load cell and load cell linkage

4. Reactor coolant pump motor lift sling

The use of the Acoustic Emission (AE) technique is an acceptable alternate method for inspection of the special lifting devices for compliance with NUREG-0612

The spent fuel cask handling crane has been upgraded to the single-failure-proof criteria of NUREG-0554, Single-Failure-Proof Cranes for Nuclear Power Plants. For handling casks, the cask crane must utilize a single-failure-proof lift system. Therefore, special lifting devices must meet the requirements of NUREG-0612, Section 5.1.6(1)(a). C25

5I.3.5 Standard Lifting Devices (Guideline 5)

Standard lifting devices used at Turkey Point are in compliance with ANSI B30.9-1971 (Reference 7) with the exception that sling selection is based on static loading and does not include dynamic loading effects. It is determined that sufficient margin is present from the 5:1 safety factor stipulated in the standard to account for any dynamic loading effects.

The spent fuel cask handling crane has been upgraded to the single-failure-proof criteria of NUREG-0554, Single-Failure-Proof Cranes for Nuclear Power Plants. For single-failure-proof lifts, the cask crane must utilize a single-failure-proof lift system. Therefore, lifting devices that are not specially designed must meet the requirements of NUREG-0612, Section 5.1.6(1)(b). Single-failure-proof lifts are those deemed reliable enough such that the consequences of a load drop need not be considered. Lifting devices that are not specially designed are usually slings. The use of synthetic slings is prohibited for single-failure-proof lifts in accordance with Revision I to Section 9.1.5, "Overhead Heavy Load Handling System," of the NRC Standard Review Plan, NUREG-0800. Metal slings (chain or wire rope) shall be used when use of a single-failure-proof handling system is necessary. C25

5I.3.6 Inspection Testing and Maintenance (Guideline 6)

The Turkey Point crane inspection, testing and maintenance program complies with the requirements of ANSI B30.2-1976, Chapter 2-2. However, as allowed in Guideline 6 of NUREG-0612 Section 5.1.1, Turkey Point has taken exception to the ANSI B30.2 requirement (Reference 5) that calls for certain inspections to be performed daily or monthly. Turkey Point performs the tests prior to use where it is not practical to meet the frequencies of testing required.

The spent fuel cask handling crane has been upgraded to the single-failure-proof criteria of NUREG-0554, Single-Failure-Proof Cranes for Nuclear Power Plants. The procedures for use in the checking, testing and maintenance program are based upon the requirements of OSHA 1910.179, ASME B30.2 and ASME NOG-I. C25

5I.3.7 Crane Design (Guideline 7)

In accordance with Guideline 7 of NUREG-0612, Section 5.1.1. heavy load handling system cranes shall comply with CMAA Specification 70 (Reference 8), and Chapter 2-1 of ANSI B30.2 (Reference 5) 1976 edition. The design of the Unit 3 and Unit 4 polar cranes has been upgraded and reconciled to the provisions of ANSI B30.2 (Reference 5) applicable in 1998. The design of the turbine gantry crane has been upgraded and reconciled to the provisions of ANSI B30.2 (Reference 5) applicable in 2005. The following Turkey Point cranes fall within the scope of the above standards: C26

1. Reactor Building Polar Cranes

2. Intake Structure Crane

3. Turbine Gantry Cranes

The cranes listed have been reviewed for compliance with Guideline 7 and have been found to meet the more restrictive design requirements of CMAA-70 which affect the cranes ability to handle heavy loads safely.

The spent fuel cask handling crane has been upgraded to the single-failure-proof criteria of NUREG-0554, Single-Failure-Proof Cranes for Nuclear Power Plants. The crane including main and auxiliary hoists, trolley and bridge are designed, manufactured and tested in accordance with the specific requirements of ASME NOG-1-2004 (Reference 22) for a Type 1 Crane (i.e. single-failure-proof crane). The crane also meets the requirements of CMAA Specification #70-2004 (Reference 23) and NUREG-0554 as provided in Florida Power and Light Company letter L-2010-022 to the NRC (Reference 24).

5I.3.8 Technical Specifications

Technical Specifications governing cask handling operations in the spent fuel pool area and prohibiting heavy loads from travelling over irradiated fuel assemblies in the spent fuel pool were removed by Reference 21.

5I.4 Reactor Vessel Closure Head (RVCH) Lifting Procedures

To control RVCH lifts, station procedures are used to control the lift and replacement of the RVCH. These procedures establish limits on the load height, load weight, and medium present under the load. These procedures (1) are consistent with the analytical methods and acceptance criteria of the RVCH drop analysis (References 15 – 17, 25); (2) comply with the guidelines in NEI 08-05 (References 18 – 20); and (3) provide additional assurance that the core will remain covered and cooled in the event of a postulated RVCH drop. C28

5I.5 References

1. NRC Generic Letter 80-113, "Control of Heavy Loads", December 22, 1980.

2. NUREG-0612, "Control of Heavy Loads at Nuclear Power Plants", July, 1980.

3. NRC letter, "Turkey Point Plant, Units 3 and 4 - Control of Heavy Loads, Phase I", November 1, 1983.

4. Generic Letter 85-11, "Completion of Phase II of Control of Heavy Loads", June 28, 1985.

5. ANSI B30.2, "Overhead and Gantry Cranes".

6. ANSI N14.6-1978, "Standard for Special Lifting Devices for Shipping Containers Weighing 10,000 Pounds (4,500 kg) or More for Nuclear Materials".

7. ANSI B30.9-1971, "Slings".

8. CMAA-70, "Specifications for Electric Overhead Traveling Cranes".

9. NUREG-0554, “Single-Failure-Proof Cranes for Nuclear Power Plants”, May 1979.

10. PTN-ENG-SECS-99-012, “Safety Evaluation for Use of Acoustic Emission Technique as an Alternative Method for NDE of Special Lifting Devices.”

11. Deleted

12. Deleted

13. Deleted

14. Deleted

5I.5 References(Continued)

15. Sargent & Lundy Calculation No. 2008-08528, Revision 1, “Analysis of Postulated Reactor Vessel Head Drop onto the Reactor Vessel Flange,” September 23, 2015. C28

16. Sargent & Lundy Report No. SL-009440, “PTN 3 & 4 Reactor Head Drop Analysis Summary Report,” March 21, 2008.

17. PTN-ENG-SENS-09-021, Revision 0, “Engineering Evaluation to Incorporate Reactor Vessel Closure Head Drop Analysis into the Licensing Basis Documents,” April 7, 2010.

18. Nuclear Energy Institute (NEI) 08-05, Revision 0, “Industry Initiative on Control of Heavy Loads,” July 2008.

19. NRC letter, W.H. Ruland (NRC) to T.C. Houghton (NEI), “Industry Initiative on Control of Heavy Loads” [Safety Evaluation Report], September 5, 2008.

20. NRC Regulatory Issue Summary (RIS) 2008-28, “Endorsement of Nuclear Energy Institute Guidance for Reactor Vessel Head Heavy Load Lifts,” December 1, 2008.

21. Turkey Point Unit 3 and 4 - Issuance of Amendments Regarding Technical Specification Changes Related to Movement of Heavy Loads Over Spent Fuel (TAC Nos. ME3379 and ME3380) dated February 25, 2011.

22. ASME NOG-1-2004, “Rules for Construction of Overhead and Gantry Cranes”

23. CMAA Specification #70-2004, “Specifications for Electric Overhead Traveling Cranes”.

24. Florida Power and Light Company letter L-2010-022 dated February 16, 2010 “License Amendment Request No. 202, Technical Specification Changes Regarding Heavy Loads over the Spent Fuel Pools”.

25. EC 284674, Revision 0, “Qualification of Reactor Head Lift with Studs,” Units 3 and 4. C28