NRC TRANSMITTAL RECEIPT

WORK ORDER NO.: NRC LB-85- 21

ACE-FEDERAL CONTROL NO.:

NAME OF PROCEEDING: SHEARON HARRIS NUCLEAR POWER PLANT

DOCKET NO.: 50-400-OL & 50-401-OL

HEARING DATE: THURSDAY, OCTOBER 25, 1984

LOCATION: APEX, NORTH CAROLINA

PAGES: 5574 TO 5843

EXHIBITS FORWARDED

Applicants' Exhibits 9 through 22 inclusive
" " 25

RECEIVED BY: JAMES L. KELLY, ESQ.

DATE: 10/26/84

50-400-OL
10/22/84

A-9

DOCKETED
NOV 27 1984
DOCKETING & SERVICE BRANCH SECY-NRC

Applicants' Exhibit 9
Eddleman Contention 65
Docket No. 50-400

Final Safety Analysis Report
Section 3.8.1
Concrete Containment

NUCLEAR REGULATORY COMMISSION

Docket No. 50-400-OL Official Exh. No. 9
In the matter of Shearon Harris
Staff ____ IDENTIFIED ✓
Applicant ✓ RECEIVED ✓
Intervenor ____ REJECTED ____
Cont'g Off'r ____
Contractor ____ DATE 10-22-84
Other ____ Witness ____
Reporter Jw.

3.8 DESIGN OF CATEGORY I STRUCTURES

3.8.1 CONCRETE CONTAINMENT

3.8.1.1 Description of the Containment

3.8.1.1.1 General Description

The Concrete Containment Structure (CCS) is a steel lined reinforced concrete structure in the form of a vertical right cylinder with a hemispherical dome and a flat base with a recess beneath the reactor vessel.

The structure, shown on Figure 3.8.1-1, consists of a cylindrical wall measuring 160 ft. in height from the liner on the base to the springline of the dome and has an inside diameter of 130 ft. The cylinder wall is 4 ft. 6 in. thick. The inside radius of the 2 ft. 6 in. thick dome is equal to that of the cylinder so that the discontinuity at the spring line due to the change in thickness is on the outer surface. The base mat consists of a 10 ft. thick structural concrete slab and a metal liner. The liner is welded to inserts embedded in the concrete slab. The base liner is covered with concrete, the top of which forms the floor of the containment. The base mat is supported by sound rock. 2

The basic structural elements considered in the design of the containment structure are the basemat, cylinder wall, and dome. These act essentially as one structure under all loading conditions. The liner plate is 3/8 in. thick in the cylinder, 1/4 in. thick on the bottom, and 1/2 in. thick in the dome. The liner is anchored to the concrete shell by means of anchor studs fusion welded to the liner plate so that it forms an integral part of the containment structure. The liner functions primarily as a leaktight membrane. An impervious plastic waterproofing membrane is placed between the containment foundation mat and the ground. Before laying the membrane, a concrete leveling surface is placed on the rock. After installing the membrane, a concrete protective layer is installed before placing reinforcement for the foundation mat. The waterproofing membrane for the Containment Building is continuous under the containment foundation mat and terminates into waterstops at the joints with adjacent structures.

The arrangement of the Containment and the relationship and interaction of the shell with the interior compartment shielding walls and floors are shown on Figures .8.1-1, 3.8.3-1, and 3.8.3-2. 2

The containment wall is independent of adjacent interior and exterior structures; sufficient space is provided between the containment wall and adjacent structures to prevent contact under any combination of loading. The interior grating platforms and concrete slabs are supported on steel beams which span between the secondary shield wall and the containment wall. These beams are independently supported, near the containment wall, by steel columns resting on the concrete mat.

The circular polar crane runway girder is supported by a series of uniformly spaced steel plate brackets which extend from the inside face of the containment wall and are attached to the liner plate. The crane runway circle is not concentric with that of the Containment, but is offset to provide a passageway on one side for the pipe runs of the containment spray header piping mounted in the dome. The liner plate is thickened to one inch to support the brackets and is anchored to the concrete containment wall.

The Concrete Containment Structure and associated parts and appurtenances is designed for an expected operating life of 40 years.

Basically three materials, concrete, reinforcing steel, and steel liner plate, are used for construction of the Containment.

The concrete has a compressive strength of 4000 psi at 28 days after placement except at the bottom portion of the cylindrical wall and around major penetrations where the concrete has a compressive strength of 5000 psi at 28 days after placement. The reinforcing steel is new billet steel in accordance with ASTM A615 Grade 60. Where called for on the design drawings, weldable grade reinforcing steel in accordance with ASTM A706 was used.

The steel liner plate is carbon steel conforming to ASTM A 516 Grade 70. This steel has a minimum yield strength of 38,000 psi and a minimum ultimate tensile strength of 70,000 psi.

The Containment encloses the reactor pressure vessel, pressurizer, steam generators, reactor coolant pumps, and piping, and portions of the Engineered Safety Features Systems. The containment wall protects the Reactor Coolant System from site environmental conditions. It is designed as a Seismic Category I structure for earthquake, tornado, and external missile loading conditions. It also limits the release of radioactive fission products to the environment in the unlikely event of a Loss of Coolant Accident (LOCA), and in addition, provides biological shielding for both normal and accident conditions. The functional requirements of the Containment are discussed in detail in Section 6.2.1.

The cylindrical section of the containment shell includes large openings for access hatchways and penetrations. The concrete wall is locally thickened and additional reinforcement is provided at these large penetrations. Penetrations are anchored in the containment wall.

A permanent steel ladder with a safety cage is provided on the exterior cylinder portion of the Containment Building for access to the bottom portion of the dome. Another ladder with a safety rail is provided on the exterior of the dome for access to the top. A guard rail is provided around the entire springline of the Containment. U-shaped steel bolts are embedded in the top and bottom of the dome to allow for the hanging of scaffolding to inspect the entire dome and cylinder portion of the Containment Building.

3.8.1.1.2 Foundation Mat

The foundation mat is a conventionally reinforced concrete mat of circular shape and 12 ft. uniform thickness. The top of the mat is 44 ft. below finished grade.

The entire mat is structurally independent of adjacent Seismic Category I foundations. The mat has a recess in the central portion to house the reactor pressure vessel, and in the engineered safety features (ESF) area, there is a recess to house the ESF system sumps for the containment spray header water which exits the Containment through two collection sumps and embedded drain pipes.

The foundation mat, inside the Containment and including the reactor cavity, is covered with 1/4 in. thick carbon steel liner plate, except at the connection with the wall liner plate, where a 3/8 in. thick liner plate is provided. A five ft. thick concrete internal mat is provided over the liner for protection and support of internal primary and secondary shield walls.

In order to protect the mat liner plate against groundwater hydrostatic pressure, an impervious waterproofing membrane is placed continuously below the foundation mat and terminates into waterstops at the joints with adjacent structures. The seismic gaps between adjacent structures are cut off from groundwater by double rows of horizontal waterstops. As described in Section 3.4.1.1, any leakage through the waterproofing membrane will be drained through porous concrete drains placed between the membrane and the concrete mat.

The primary and secondary shield walls are supported by the internal foundation mat which in turn is resting on the external foundation mat. No anchorage of the interior structures through the liner plate and into the external mat is provided.

The reinforcing steel of the foundation mat, shown on Figure 3.8.1-2, consists of radial and circumferential reinforcement placed at the top and bottom of the mat. Radial bars have no splices; circumferential bars utilize the longest length possible so that the number of splices is minimized. Splices are staggered whenever practical. Shear reinforcement is provided whenever required by design. The base mat is considered a circular flat slab resting on an elastic foundation and the finite element approach was used for analysis. The mat is designed to withstand the loading defined in Section 3.8.1.3.

3.8.1.1.3 Cylindrical Wall

3.8.1.1.3.1 Reinforcing Steel Arrangement

The reinforced concrete cylindrical wall is designed to withstand the loadings and stresses anticipated during the operating life of the plant, as defined in Section 3.8.1.3. The steel liner is attached to, and supported by, the concrete. The liner functions primarily as a gas-tight membrane and also transmits loads to the concrete. During construction, the steel liner serves as the inside form for the concrete wall and dome. The containment structure does not require the participation of the liner as a structural component.

Hoop tension in the cylindrical concrete wall is resisted by horizontal reinforcing bars near both the outer and inner surfaces of the wall.

Horizontal circumferential bars, including those in the dome, have their splices staggered wherever possible.

Longitudinal tension in the cylindrical wall is resisted by rows of vertical reinforcing bars placed near the interior and exterior faces of the wall, with cadweld splices staggered whenever practical.

Figure 3.8.1-3 shows typical reinforcing steel for the cylindrical wall.

Reinforcing steel which terminates in locations where biaxial tension is predicted, such as at penetrations, is anchored by hooks, bends, or positive mechanical anchorage in such a manner that the force in the terminated bar is adequately transferred to other reinforcement. Also, bar development length at such location is increased.

The main vertical and hoop reinforcing steel in the containment wall and dome have a concrete cover of 4 inches. Concrete cover for reinforcing steel other than these are governed by provisions listed in the ASME/ACI 359 code.

The juncture of the cylinder to the base slab is considered to be rigidly connected. The cylinder at this point cannot expand but joint rotation is considered as the wall deforms under the internal pressure, temperature, and dead load conditions; hence, radial shear and moments are introduced into the cylinder wall. All the radial shears at the base of the cylinder wall are resisted by reinforcing steel. This shear reinforcing is horizontal.

The nonaxisymmetric loads, such as wind, tornado, and seismic excitations, induce tangential shears into the cylindrical concrete wall and concrete dome. Although the liner plate in the cylindrical wall and dome has shear capacity available to resist tangential shear, no credit was taken for this capacity. The tangential shear carried by the concrete does not exceed 60 psi and 40 psi for abnormal load combinations associated with the safe shutdown earthquake and operating basis earthquake, respectively, as required by Standard Review Plan 3.8.1. The excess tangential shear is taken by diagonal seismic reinforcing bars. The seismic reinforcement, shown on Figures 3.8.1-4 and 3.8.1-5, extends diagonally into the dome until a point is reached where the concrete alone can resist the tangential shear. Sufficient overlap is made between the linear and diagonal reinforcing to allow transfer of shears. At the major penetrations, the seismic reinforcement is either bent around the penetration or is cut off, in which case a mechanical embedment, consisting of a cadweld sleeve welded to an anchorage, is provided.

The concrete thickness of the wall is increased from 4 ft. 6 in. to 6 ft. 6 in. around the major penetrations such as the equipment hatch, personnel lock, emergency air lock, main steam penetrations, and feedwater penetrations. In all of these areas, the main hoop and vertical reinforcement are bent around openings, hooked into the wall, or terminated using a mechanical embedment. Additional circular radial and shear reinforcement is provided to withstand stress concentrations and additional radial and in-plane shear developed in these areas by the loading combinations described in Section 3.8.1.3.

Figure 3.8.1-6 shows the reinforcement in the equipment hatch area of the containment structure.

Figures 3.8.1-7 and 3.8.1-8 show the reinforcement in the personnel air lock, emergency air lock, and HVAC penetrations areas. In all of these areas, the anchorage of the steel penetration into the concrete wall is provided by steel anchorages welded to the penetrations sleeves. For all penetration sleeves designed in accordance with requirements of the ASME Code Section III Division 1, such as the equipment hatch, personnel air lock, emergency air lock, and Type I penetration sleeves, special anchorages were provided using ASME Code material and manual welding. For all penetration sleeves designed in accordance with requirements of the ASME Code Section III Division 2, in the portion backed by concrete, such as Type II and Type III penetration sleeves, double headed machine welded Nelson Studs were provided.

Figure 3.8.1-9 shows the reinforcement in the main steam and feedwater penetration area. In addition to the main circumferential and vertical reinforcement bent around penetrations, additional circular reinforcement is provided around each individual penetration and radial interconnecting reinforcing bars. In order to provide for sufficient resistance against excessive rupture loads and to accommodate the interaction between the concrete structure and steel penetrations, the attachments of the penetration sleeves are directly connected with the radial reinforcing bars transferring the loads into the concrete wall.

Figure 3.8.1-10, Section p-p, shows the 6 in. attachments shop welded to the penetration sleeve. No. 18 radial reinforcing bars are connected through a cadweld mechanical connection to 9 in. attachments, which in turn are field welded to the 6 in. attachments connected to the sleeves.

The reinforcement arrangement around penetrations smaller than 18 in. is shown on Figure 3.8.1-11. Structural built-up steel members are provided to transfer the forces from the main circumferential and vertical reinforcing bars to special bars, closely spaced, or reinforcing bars were bent around openings. Additional inclined reinforcement is provided when required.

3.8.1.1.3.2 Liner Plate

A continuous welded steel liner plate is provided on the entire inside face of the concrete containment cylindrical wall to limit the release of radioactive materials into the environment. The thickness of the liner in the cylindrical wall area is 3/8 in. A one inch thick liner plate is provided at the crane girder brackets elevation. Ring collars up to 2 in. thick are provided around all penetrations and shop welded to the penetration sleeves, as required by ASME Section III Division 2/ACI 359 Code, Section CC4552.2.1.

Figures 3.8.1-12 and 3.8.1-13 show liner plate details. An anchorage system, consisting of Nelson Studs 5/8 in. diameter by 4 in. long, is provided to prevent instability of the liner for all load combinations described in Section 3.8.1.3.

In order to minimize liner stresses, strains and deformations under the design loading condition described in Section 3.8.1.3, the cylindrical wall liner plate connection with the foundation mat lower plate is an unanchored embedded 90 degree free-standing welded connection. No anchor studs are provided on a 5 ft. vertical portion and on a 3 ft. horizontal portion of the liner plate. In order to allow free deformation of the liner plate during test pressure conditions, an inch of ethafoam is provided on the inside face of the liner plate facing the concrete of the internal mat. In order to allow vertical movement at the concrete connection during the same test pressure conditions, ethafoam is also provided against the back up plate and the end of the horizontal liner plate, as shown on detail X on Figure 3.8.1-12.

The one inch liner plate at the crane girder brackets area is anchored into the concrete wall with shear lugs, anchor bolts connected to embedded plates, special anchorages, and Nelson studs, as shown on detail Y and Section A-A on Figure 3.8.1-12, in order to withstand the complexity of loading induced during operation of the crane and/or seismically induced loads.

Figure 3.8.1-13 shows the arrangement of anchor studs around different types of containment penetrations.

Leak chase channels or angles are provided at the liner seams for leak tightness examination.

There are no through liner attachments. The supports for HVAC ducts, piping hangers, and ladders, are welded to the liner plate, which is locally reinforced with additional studs in the region of surface attachments.

A yield strength of 45.6 ksi (70F-100F) was used for the 3/8 inch thick plate. This yield strength is the basis for considering that, for both service and factored load conditions, the yield stress is not exceeded in the regions identified as overstressed (if plate yield stress is 38 ksi (70F-100F)). This is a conservative value and was obtained as follows:

a) All certified mill test reports for the 3/8 inch thick plate that was supplied were reviewed. The least yield stress value from all reports for that thickness plate is 45.6 ksi. This is the value that was used. It was reduced for higher temperatures (temperatures from 100F to 240F) by the application of ASME Section III Division I Appendix Table I-2.1 "Yield Strength Values Sy for Ferritic Steels", values for SA 516 Grade 70. Two straight line reductions in yield strength were obtained from the table, the first, for reduction in strength between 100F and 200F, and the second, for
14 reduction in strength between 200F and 300F. The slopes of the two lines were expressed in terms of reduction in strength, ksi, per degree F temperature increase and applied to the 45.6 ksi least yield strength value to obtain reduced yield strength values for temperatures up to 240F.

b) Reductions in modulus of elasticity for the material due to increase in temperature were also evaluated, based on ASME Section III Appendix I Table I-6.0 "Moduli of Elasticity E of Materials for Given Temperatures", and considered in the determination of strain at various temperatures.

c) The certified test reports of all the welding electrodes for the liner plate joining welds were also reviewed. The least value of yield was found to be 58.0 ksi. It was concluded that the electrodes supplied do not adversely affect the yield strength of the liner plates.

d) Verification of liner strains due to containment pressurization is obtained from the liner strains measurements made for the containment building structural integrity test. The test is described in Section 3.8.1.7.1. The liner strain gage locations are shown in Figures 3.8.1-47, 48, and 49. 14

3.8.1.1.3.3 Containment Penetrations

Access into the Concrete Containment Structure is provided by an equipment hatch, a personnel air lock, and an emergency air lock.

The equipment hatch is a welded steel assembly having an inside diameter of 24 ft. 0 in. with a weld-on cover with sufficient material to allow for six removals and rewelding. A 15 ft. 0 in. inside diameter bolted cover is provided in the equipment hatch cover for passage of smaller equipment during plant operation. Provision is made to pressurize the space between the gaskets of the bolted hatch cover to 36.7 psig. Figure 3.8.1-14 shows the equipment hatch.

One breech-type personnel air lock (Figure 3.8.1-15) and one personnel emergency air lock (Figure 3.8.1-16) are provided. Each lock is a welded steel assembly having two doors which are double-gasketed with material resistant to radiation. Provisions are made to pressurize the space between the gaskets. The doors of each lock are equipped with quick acting valves for equalizing the pressure across each door and the doors are not operable unless pressure is equalized. There is visual indication outside each door showing whether the opposite door is open or closed and whether its valve is open or closed. Provisions have been made outside each door for remotely closing and latching the opposite door so that in the event that one door is accidently left open it can be closed by remote control. Interior lighting and communications systems were installed. These systems are not capable of operating from emergency power supply.

Two pressure gages are placed at each end of the personnel locks, one reads from outside the lock and measures lock pressure. The other reads from inside the lock and measures containment pressure. Nozzles are installed which permit pressure testing of the locks at anytime.

The breech-type personnel air lock has a 9ft.-0in. inside diameter with full diameter breech doors to open outwardly from each end of the lock. Doors for the lock are hydraulically sealed and electrically interlocked. During plant shutdown, it will be necessary to open both doors at the same time; therefore, a key-operated electrical interlock defeat system which is under strict administrative control is provided. Opening of the doors after unsealing will be done with a hydraulic motor, as will closing before sealing. Manual (hand pump) operation of the sealing ring and door swing mechanism is provided in case of a power failure.

All leakage and pressure testing on the breech-type personnel air lock will be done without the use of the test clamps since sealing is accomplished by forcing the doors against the seals when the rotating third seal ring is rotated into the breech locked position. Since the pressure applied to the double seals of the lock during testing is exerted by the third ring, the effectiveness of the seal cannot be increased beyond that seen during operating or accident condition. Test connections are provided for continuous testing between the double seals of each door for leakage.

The personnel emergency air lock has an outside diameter of 5 ft. - 0 in. with a 2 ft. - 6 in. diameter door located at each end of the lock. The doors of the lock are in series and are mechanically interlocked to ensure that one door cannot be opened until the second door is sealed. Violation of the interlock can only be made by use of special tools and procedures under strict administrative control.

Test clamps are provided for leakage and pressure testing of the personnel emergency air lock. This set of clamps fits either door and is designed to withstand, as a minimum, the full peak containment internal pressure. Compression of the double seals on each of the doors is limited to that which occurs before a metal to metal seat is achieved between the door and the protruding metal flange adjacent to the seals on the lock barrel. The internal containment pressure (or pressure exerted by the test clamps) necessary to achieve the metal to metal seat is approximately 3 PSI over the surface of the door. Effectiveness of the seals during testing, therefore, cannot be artificially increased beyond that seen during operating or accident conditions by overtightening of the clamps. Mechanical and electrical penetrations are provided in the cylindrical wall of the containment structure to provide access for mechanical piping and electrical cables.

Mechanical penetrations are divided into two general types:

a) Type I - High pressure, high temperature piping (above 200 F).

b) Type II - General piping (penetrations which are subject to only relatively small pipe rupture forces and temperatures up to 200 F).

Type I mechanical piping hot penetrations are provided for high pressure and high temperature (above 200 F) lines which penetrate the concrete containment structure. The process pipe is connected to a containment penetration sleeve (which is partially embedded in the concrete wall) by a forged flued head fitting. The flued head fittings are designed to carry the forces and moments due to the normal operating conditions and due to the postulated pipe rupture loads by transferring these forces to the containment penetration sleeves and further into the concrete containment wall.

Figure 3.8.1-17 shows a Type I mechanical penetration.

Type II mechanical piping cold penetrations are provided for low temperature (below 200 F) lines which penetrate the concrete containment structure. As shown on Figure 3.8.1-18, the process pipe passes through a containment penetration sleeve which is partially embedded and anchored into the concrete wall. The annular gap between the process pipe and the sleeve is sealed on both the inside and outside faces of the concrete wall. The inside plate is designed to withstand the internal pressure and to transfer all of the normal operating loads and/or the postulated accident piping rupture loads from the piping system to the penetration sleeve and then into the concrete wall. The outside seal is flexible to accommodate thermal expansion movements.

Type II penetrations also include HVAC penetrations and groups of small diameter lines (instrument, sampling lines) which incorporate socket weld couplings welded to closure plates. Two categories of penetration are included in Type II penetration: Type IIA for single tubing or multiple pipes and/or tubings and Type IIB for single pipe.

Electrical penetrations are included within the Type III penetrations. Modular type penetrations are used for all electrical conductors passing through the containment wall. Each penetration assembly consists of a stainless steel header plate attached to a carbon steel welded ring which is in turn welded to the pipe sleeve. The header plate accepts either three or six modules depending on the penetration diameter and voltage classification. The modules are held in the header plates by means of retaining clamps. Each module is a hollow cylinder through which the conductors pass. The conductors are hermetically sealed into the module with an epoxy compound. Each module is provided with a pressure connection to allow pressurization for testing. Figure 3.8.1-19 shows typical electrical penetrations. The header plates are attached to penetration sleeves located in the wall of the containment vessel and welded to the containment liner. Sealing between the header plates and the sleeves is accomplished by welding. All materials used in the design are selected for compatibility with all possible environmental conditions during normal, accident, or post-accident periods. Spare electrical penetration sleeves are provided for possible future uses. Each penetration is sealed and tested at the factory for leakage. The only seals that need to be made in the field are the welds attaching the header plates to the sleeves.

HVAC penetration sleeves, 48 in. and 24 in. diameter, are similar to the mechanical Type II penetration sleeves.

A fuel transfer penetration is provided to transport fuel assemblies between the refueling canal in the Containment and the fuel transfer canal in the Fuel Handling Building. This penetration consists of a 20 in. diameter stainless steel pipe installed inside a 26 in. pipe. The inner pipe acts as the transfer tube and is fitted with a double-gasketed blind flange in the refueling canal and a standard gate valve in the fuel transfer canal. This arrangement prevents leakage through the transfer tube in the event of an accident.

The penetration sleeve is welded to the steel liner and anchored into the concrete wall.

Provision is made for testing welds essential to the integrity of the liner. Bellows expansion joints are provided to compensate for any differential movement between the structures, due to operating thermal expansion and seismic movements.

The fuel transfer tube expansion joints are not part of the containment pressure boundary. Rather the transfer tube is rigidly attached to the containment penetration sleeve. Two bellows type expansion joints are installed, the first forming a flexible joint between the transfer tube and the transfer canal inside the Containment; the second forming a flexible joint between the transfer tube and the Fuel Handling Building fuel transfer canal. Figure 3.8.1-20 shows the design of the fuel transfer tube.

The expansion joint inside the Containment is accessible for visual inspection at any time. The expansion joint in the Fuel Handling Building is also accessible for inspection at any time except when the transfer canal is flooded during the actual fuel transfer period.

Also included are four valve chambers and their appurtenances. The valve chambers and their appurtenances, shown on Figure 3.8.1-21, are 9 ft - 0 in. diameter by 10 ft. - 0 in. long airtight enclosures which function as a secondary containment boundary to completely enclose the containment sump lines and isolation valves.

3.8.1.1.4 Containment Dome

The containment dome is a lined reinforced concrete hemispherical dome of 2 ft. 6 in. uniform thickness. A continuous welded steel liner plate. one-half inch thick, is provided on the inside face of the dome. The arrangement of the studs in the dome is shown on Figure 3.8.1-12. Nelson studs 5/8 in. diameter by 4 in. long are used to connect the liner to the concrete. | 2

The reinforced concrete dome is designed to withstand the loads anticipated during the operating life of the plant and postulated accidents and events described in Section 3.8.1.3. Meridional and circumferential reinforcing bars are provided to resist the resulting tensile forces and bending moments.

Figures 3.8.1-22 and 3.8.1-23 show the arrangement of the reinforcement in the dome.

The dome reinforcement consists of layers of reinforcing steel placed meridionally, extending from the vertical reinforcing of the cylindrical wall, and horizontal layers of circumferential bars. The layers are located near both the inner and outer faces of the concrete. The radial pattern of the meridional reinforcing steel, terminating in the containment dome, results in a high degree of redundancy of reinforcing steel in the dome. Bars are terminated beyond a point where there is more than twice the amount of steel required for design purposes. The rate of convergence of these bars, and the low stress requirements dictated by this arrangement, results in a satisfactory development length of the meridional reinforcing bars. Near the crown of the dome, the meridional reinforcing bars are welded to a steel hub plate, cast in the concrete, concentric with the dome centerline.

Although the liner plate is not considered as a structural element to sustain the loading imposed on the dome, during construction the liner plate is used as a form to withstand the weight of the reinforcing steel and fresh concrete during placement. To minimize the locked in stresses during construction, the placement of the concrete in the dome area is made in lifts of 4 to 5 ft. of concrete. The next placement of the concrete is added only when the concrete previously placed has enough strength to take additional construction loads.

Ventilation openings are provided at the top of the dome to be used during construction. These openings are filled with concrete when construction is finished.

3.8.1.1.5 Containment Structural Boundaries

The Containment is a composite steel and reinforced concrete assembly that is designed as an integral part of the containment's pressure-retaining barrier to retain and control the release of radioactive or hazardous effluents released from the nuclear power plant equipment which the containment encloses.

The design, materials, fabrication, construction, testing, examination, structural integrity test, and quality assurance for the Containment Building, consisting of a reinforced concrete mat, cylindrical wall and dome, lined with steel liner, and associated materials, parts, and appurtenances, are in accordance with ASME Code Section III Division 2/ACI 359 Code winter 75 addendum, with the exceptions listed in Appendix 3.8A.

All pressure-retaining, leak-resisting, and load-bearing concrete and steel portions of the Containment and all parts or appurtenances that act integrally with the pressure-retaining portion to carry the fluid pressure loads are covered by the ASME Code Section III Division 2/ACI 359 Code, except that:

a) Parts and appurtenances under the jurisdiction of Section III Division 1 are considered only with respect to their functional collaboration with the concrete and steel portions of the component in carrying loads.

b) Parts and appurtenances under the jurisdiction of Section III Division 1, whose directional loadings can be described by moments and forces acting on portions of the concrete component for design purposes, are characterized by such loading conditions which for the concrete containment can be shown to be functionally acceptable.

c) Parts and appurtenances specified to meet the requirements of Section III Division 1 and furnished before April 29, 1977, meet the requirements of Subsection NA of Division 1. Parts and appurtenances furnished after April 29, 1977, meet the requirements of Subsection NA of Division 2. The parts and appurtenances which are designed under the jurisdiction of Section III Division 1 are presented in Table 3.8.1-1.

The boundaries of the Containment Building and the different parts and appurtenances are shown on Figures 3.8.1-24 and 3.8-1-25.

For the design of the Equipment Hatch, Personnel Air Lock, Emergency Air Lock, and all penetrations, at the transition portion from concrete to steel, the following aspects are considered:

a) Metal sections not backed by concrete meet the requirements of Division 1 and consider the concrete confinement except that proof testing is in accordance with CC-6000 of the ASME Code Section III, Division 2/ACI-359 Code.

b) Metal sections are attached to concrete sections by one of the following:

1) Tension attachment to the primary reinforcement of the concrete containment.

2) Anchorage system attached to the metal shell and extended into the concrete. The metal shell is not reduced below the minimum thickness required for primary mechanical loads for a distance of 25 t from the point where the concrete-to-metal junction occurs, where t is the thickness of the metal penetration sleeve at the transition section.

Where the penetration sleeves or the liner is backed by compressible material to provide local flexibility, the penetration sleeves or the liner meet all requirements for material, design, fabrication, and examination of the ASME Code, Section III, Division 1 in the region where compressible material is present. Where penetration sleeves or liner are attached to concrete directly or to embedded members, only the requirements for liner apply.

3.8.1.2 Applicable Codes, Standards, and Specifications

The structural design, materials, fabrication, construction, testing, inservice surveillance, and quality assurance for the Containment conform to the codes, standards, regulations, and specifications listed below, except where specifically stated otherwise.

General Codes and Standards

OSHA	Occupational Safety and Health Administration, Federal Safety Regulations (1975 listing)

North Carolina State Building Code, 1969 Edition

ACI	American Concrete Institute Standards
211.1-1974	Recommended Practices for Selecting Proportions for Normal and Heavy Weight Concrete
301-1975	Specifications for Structural Concrete for Buildings
304-1973	Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete
305-1972	Recommended Practice for Hot Weather Concreting
306-1966	Recommended Practice for Cold Weather Concreting
309-1974	Recommended Practice for Consolidation of Concrete
315-1974	Manual of Standard Practice for Detailing Reinforced Concrete Structures
318-1971	Building Code Requirements for Reinforced Concrete
347-1968	Recommended Practice for Concrete Formwork
SP-2-1975	Manual of Concrete Inspection
AISC	**American Institute of Steel Construction**
	Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings (AISC Specification) (2/12/69, with Supplements 1- 11/1/70, 2- 12/8/71, and 3- 6/12/74)
ASME	**American Society of Mechanical Engineers**
ASME Section III Division 2	Code for Concrete Reactor Vessels and Containments (ASME Boiler and Pressure Vessel Code, Section III, Div 2) 1975 Edition, with winter 1975 Addenda and other ASME Code Sections as required by ASME Section III, Division 2/ACI 359 Code.
	Exceptions to the ASME Section III, Division 2/ACI 359-75 Code are listed in Appendix 3.8A.
Section II	Material Specifications
Section III Division 1	Nuclear Power Plant Components, Subsection NE for Class MC Components

Section IX	1971 Edition with Summer 73 Addenda. Welding and Brazing Qualifications. Field welding is performed to 1971 Edition with Winter 1976 Addenda, Welding and Brazing Qualifications.
AWS	American Welding Society
D 2.0	Welded Highway and Railway Bridges with 1967 and 1970 revisions, for services performed prior to 4/29/77
D 1.1-75	Structural Welding Code, with Revisions 1 (1976) and 2 (1977) for services performed after 4/29/77
D 12.1-75	Recommended Practices for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Construction
SSPC	Steel Structures Painting Council
SP-6	Commercial Blast Cleaning
USNRC	United States Nuclear Regulatory Commission

The following NRC Regulatory Guides as identified in Section 1.8 are applicable:

1.10	Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures
1.15	Testing of Reinforcing Bars for Category I Concrete Structures
1.18	Structural Acceptance Test for Concrete Primary Containment
1.19	Nondestructive Examination of Primary Containment Liner Welds
1.54	Quality Assurance Requirements for Protective Coatings Applied to Water-Cooled Nuclear Power Plants
1.55	Concrete Placement in Category I Structures
1.57	Design Limits and Loading Combinations for Metal Primary Reactor Containment System Components
1.60	Design Response Spectra for Seismic Design of Nuclear Power Plants
1.61	Damping Values for Seismic Design of Nuclear Power Plants
1.63	Electric Penetration Assemblies in Containment Structures for Water-Cooled Nuclear Power Plants

1.76	Design Basis Tornado for Nuclear Power Plants
1.92	Combination of Modes and Spatial Components in Seismic Response Analysis
1.94	Quality Assurance Requirements for Installation, Inspection, and Testing of Structural Concrete and Structural Steel during the Construction Phase of Nuclear Power Plants
1.122	Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components
<u>ANSI</u>	<u>American National Standards Institute</u>
N6-2	Safety standard for the design, fabrication and maintenance of steel containment structures for Stationary Nuclear Power Reactors.
N-101.2-1972	"Protective Coatings (Paints) for Light Water Nuclear Reactor Containment Facilities."
N-101.4-1972	"Quality Assurance for Protective Coatings Applied to Nuclear Facilities."
N512-1974	"Protective Coatings (Paints) for the Nuclear Industry."
N45.2.2-1972	"Packaging, Shipping, Receiving, Storage, and Handling of Items for Nuclear Power Plants." (During the construction phase of SHNPP) and associated Amendments.
N45.2.5-1974	"Supplementary Quality Assurance Requirements for Installation, Inspection, and Testing of Structural Concrete and Structual Steel during the Construction Phase of Nuclear Power Plants," except that bolt threads will be 11 allowed to be flush with the top of the connecting nut in accordance with ANSI N45.2.5-1978.

<u>Industry Standards</u>

Industry standards, such as those published by the American Society for Testing and Materials (ASTM) or the American Association of State Highway and Transportation Officials (AASHTO), are used whenever possible to describe material properties, testing procedures, fabrication, and construction methods.

<u>Specifications</u>

The following specifications specify the requirements for materials, design criteria, fabrication, erection, inspection, and quality assurance. These specifications, in general, reflect and expand on the requirements set forth in ASME Section III, Division 2/ACI 359 Code.

a) Ebasco Specification CAR-SH-AS-1 "Containment Liner, Air Locks, and Hatch"

b) Ebasco Specification CAR-SH-AS-7 "Structural Steel"

c) Ebasco Specification CAR-SH-M-54 "Mechanical Penetrations"

d) Ebasco Specification CAR-SH-E-30 "Electrical Penetrations"

e) Ebasco Specification CAR-SH-CH-6 "Concrete"

f) Ebasco Specification CAR-SH-CH-7A "Concrete Reinforcing Steel"

g) Ebasco Specification CAR-SH-CH-7 "Weldable Concrete Reinforcing Steel"

h) Ebasco Specification CAR-SH-CH-12 "Waterstops"

i) Ebasco Specification CAR-SH-CH-13 "Waterproofing"

j) Ebasco Specification CAR-SH-CH-15 "Mechanical Splicing of Concrete Reinforcing Steel"

k) Ebasco Specification CAR-SH-CH-16 "Dome Hub Plates and Reinforcing Steel Splice Assembly"

l) Ebasco Specification CAR-SH-CH-22 "Structural Integrity Test of Concrete Containment Building"

3.8.1.3 Loads and Loading Combinations

3.8.1.3.1 Definitions of Loads

The following nomenclature and definitions apply to all loads encountered and/or postulated for the design of the Containment:

a) Dead Loads, (D) - Dead load consists of the weight of the concrete wall, dome, base slab, equipment deadweight, and all internal concrete, including hydrostatic loads. Uplift forces which are created by the displacement of groundwater, assumed to be at Elevation 251 ft., are accounted for in the design of the structure. Included are the weights of piping, cable trays, and ductwork.

A reinforced concrete density of 143 pcf, with a possible minimum of 137 pcf, was used in the design. The density of the steel reinforcing and liner plate used in the design was 489 pcf.

The deadweight of the crane bridge and trolley was also considered in the design. Equipment permanent operating loads as specified by the equipment manufacturers were included in the dead loads of the structure.

b) Live Loads, (L) - Live load consists of loads on the dome which are uniformly applied to the top surface of dome at an assumed value of 20 psf of horizontal plan projection to assure a strength adequate to support snow loading. A random temporary loading condition during construction or maintenance was assumed to be 50 psf. The design also accounts for a load of 250 tons supported by the polar crane during construction and maintenance operation (load combination 2) and 175 tons during load combinations 4, 5, 6, 7, and 9, described in Section 3.8.1.3.2.

c) Normal Operation Temperature Load (To) - Normal operation temperature loads consist of the loads induced by thermal gradients existing through the concrete wall and dome and those exerted on the concrete by the liner under normal operating conditions. The temperature gradient through the wall is essentially linear and is a function of the operating temperature internally and the average ambient temperature externally. The temperature gradient between the outside and inside of the Containment during operation induces stresses in the structures which are of internal nature, tension outside and compression inside the shell. Both summer and winter operating conditions are considered. In all cases the conditions assumed are considered of long enough duration to result in a straight line temperature gradient. The gradients considered are:

Summer Operation	Operating	Shutdown
Operating temperature inside building	120 F	65 F
Exterior sustained concrete temperature	90 F	90 F

Winter Operation	Operating	Shutdown
Operating temperature inside building	120 F	50 F
Exterior sustained concrete temperature	20 F	20 F

For all cases, the "as constructed" temperature is assumed to be 60 F.

Transient thermal gradients during startup and shutdown are considered in the analysis.

d) Operating Pipe Loads (Ro) - The pipe reaction anchor loads during normal operating or shutdown conditions are the loads exerted upon the containment structure by pipe restraints under the normal operating or shutdown thermal conditions of various piping systems.

e) Internal Pressure Load (Pv) - An internal negative pressure (other than due to a LOCA) of 2 psig is considered in the design.

An internal positive pressure (other than due to a LOCA) of 3 psig is considered in the design.

Either the negative pressure or the positive pressure is used in the load combination, whichever is more critical for the particular item of interest.

f) Test Pressure Load (Pt) - Section CC 6201 of the ASME Section III, Division 2/ACI 359 Code, requires that an internal pressure of up to 115 percent of the design pressure be considered in the design of the

Containment. To meet this requirement, an internal pressure of 51.75 psig was used in the design.

g) Test Thermal Load (Tt) - Thermal loads during pressure tests, including liner expansion and temperature gradients in the wall and dome, are considered in the design of the Containment.

h) Design Basis Accident Pressure Loads (P) - The design basis accident pressure loads, due to a loss of coolant accident or other postulated pipe breaks, are considered in the design of the Containment. An equivalent static design pressure of 45 psig was used in the design of the Containment Structure. The use of an equivalent static load in the design of the containment for LOCA loadings is justified. Comparison of the time of LOCA pressure rise to the initial peak value, and the natural period for the first circumferential ("breathing") mode indicates that the ratio of the time of LOCA pressure rise to the first period of vibration is on the order of about 500:1. Therefore, the load can be considered to be statically applied, and the dynamic load factor for the LOCA pressure loading is essentially unity. 14

Axisymmetric dynamic analysis studies indicated that the contribution of the higher (oval) modes to the maximum responses are relatively small. Therefore, these modes were not considered in the dynamic analysis of the containment building.

i) Design Basis Accident Thermal Loads (Ta) - Thermal stresses due to an internal temperature increase caused by the design basis accident are considered.

The containment liner design temperature under the design basis accident is assumed equal to 233 F, associated with 1.0, 1.25, and 1.5 times the accident pressure, as described in Section 3.8.1.3.2. Accident temperatures mainly affect the liner, rather than the concrete and reinforcing bars, due to the insulating properties of the concrete. By the time the temperature of the concrete within the interior of the concrete begins to rise significantly, the internal pressure and temperature in the Containment due to the accident have been drastically reduced from their maximum.

The concrete wall is designed for a steady-state temperature gradient, with the interior face subjected to a normal operating temperature of 120 F and the exterior face subjected to summer or winter operation temperature, as specified in Section 3.8.1.3.1.C). In addition, due to the interaction between the liner which is subjected to the containment design accident temperatures of 233 F, and the concrete wall which is subjected to a steady-state temperature gradient, increased stresses induced in the reinforcing steel and concrete are considered in the design.

j) Earthquake Loads (E, E') - Earthquake loads are computed using the following:

1) Operating Basis Earthquake (E) horizontal ground acceleration is 0.075g.

2) Safe Shutdown Earthquake (E') horizontal ground acceleration is 0.15g.

3) To account for the simultaneous action of the three spatial components of the earthquake, the representative maximum value of a particular response is obtained by taking the square root of the sum of the squares of the corresponding maximum values of the response to each of the three spatial components calculated independently.

Specific loads resulting from the application of the above accelerations are obtained by the seismic dynamic analysis, as described in detail in Section 3.7.2.

k) Wind Load (Hu) - As described in Section 3.3.1, wind loading for the containment structure is based on a 179 mph wind, with gust factors included, at 30 ft. above ground level. Distribution of the wind load is made in accordance with References 3.8.1-46 and 3.8.1-47, as described in Section 3.3.1.3.

l) Tornado Load (W) - As described in Section 3.3.2 tornado loading for the containment structure is based on the following characteristics:

1) External wind forces resulting from a tornado funnel with a horizontal peripheral tangential velocity of 290 mph and a horizontal translational velocity of 70 mph, W. Conservatively, this is taken as 360 mph wind applied uniformly over the entire height of the Containment. The loading distribution around the structure is in accordance with References 3.8.1-47; gust factors are taken as unity.

2) Decrease in atmospheric pressure of 3 psi at a rate of pressure drop of 2 psi/sec., Wp. Venting of the structure is not considered.

3) The missile spectra given in Section 3.5.1.4 is used in the design of the containment structure, W_m. The methods of establishing the overall structural response due to missile impact are described in Section 3.5.3.2.

4) In determining the total tornado load, W, the effects of the uniform tornado wind load, W_w, the tornado differential pressure load, Wp, and the tornado missile load, Wm, are considered by using the combinations listed in Section 3.3.2.2.4.

m) Design Basis Accident Piping Loads (Ra) - The pipe reaction anchor loads during accident conditions are the loads exerted upon the containment structure by pipe restraints under the thermal conditions generated by the design basis accident, including Ro.

n) Pipe Accident Loads (Rr) - The pipe accident loads are the loads exerted upon the containment structure due to local effects of the design basis accident and include:

1) Rrr - Equivalent static load on the structure generated by the reaction of a broken high-energy pipe during the postulated break, including an appropriate dynamic load factor to account for the dynamic nature of the load.

2) Rrj = Jet impingement equivalent static load on a structure generated by or during the postulated break, including an appropriate dynamic load factor to account for the dynamic nature of the load.

3) Rrm = Missile impact equivalent static load on a structure generated by or during the postulated break, as from pipe whipping or small pieces of equipment travelling at high velocities, including an appropriate dynamic load factor to account for the dynamic nature of the load.

o) Post-LOCA Flooding (Hq) - Post-LOCA flooding of the Containment for the purpose of fuel recovery is not a design condition. When access to the Containment is required following a LOCA, all necessary repairs will be made to permit fuel recovery.

3.8.1.3.2 Load Combinations

The design of the Concrete Containment Structure incorporates two general loading categories, the Service Load Category and the Factored Load Category.

3.8.1.3.2.1 Service Load Combinations

Service load combinations are any conditions encountered during construction and normal operation of the plant. Included in such conditions are any anticipated transient or test conditions during normal and emergency startup and shudown of the nuclear steam supply, safety, and auxiliary systems. Also included in this category are those severe environmental conditions (operating basis earthquake and wind load) which may be anticipated during the life of the facility. The service load combinations are presented in Table 3.8.1-2.

3.8.1.3.2.2 Factored Load Combinations

Factored loads include loads encountered in the life of the facility such as severe environmental loads (wind loads, operating basis earthquake), extreme environmental loads (tornado loads, safe shutdown earthquake), and abnormal loads (loads generated by the design basis accident, P, Ta, Ra, and Rr). The factored load combinations are presented in Table 3.8.1-2.

3.8.1.4 Design and Analysis Procedures

3.8.1.4.1 General Considerations

The analysis of the containment shell is based on the classical theory of thin elastic shells of revolution in accordance with Section CC-3300 of the ASME Code, Section III, Division 2. The shell is assumed to be ideally elastic, homogeneous, and isotropic. Reinforcement and the steel liner are neglected in calculating the member stiffness.

The design of the Containment demonstrates that, for factored load conditions, the following requirements are met:

a) The summation of external and internal forces and moments satisfies the laws of equilibrium and does not bring any structural section to a general yielding state.

b) Tensile yielding in the reinforcement is acceptable when thermal gradient temperature effects are combined with other applicable loads, provided that the temperature induced forces and moments are reduced as yielding in the reinforcement occurs, and the increased concrete cracking does not cause deterioration of the Containment.

The liner plate is not used as a strength element. Interaction of the liner with the Containment is considered in determining liner behavior.

The general requirements used in the design of the metallic liner are as follows:

a) The liner plate is designed to withstand the effects of imposed loads and to accommodate deformation of the concrete containment without jeopardizing leaktight integrity.

b) The liner plate is welded using weld details which do not jeopardize leaktight integrity of the Containment.

c) The liner plate is anchored to the concrete containment. This does not preclude local flexural deformation between anchor points.

d) The liner plate is designed within the limits of stress, strain, and deformation specified in Table 3.8.1-3.

The liner anchorage system is designed to accommodate all design loads and deformations without loss of structural or leaktight integrity.

The anchorage system is designed so that a progressive failure of the anchorage system is precluded in the event of a defective or missing anchor.

Penetration assemblies, including sleeves, reinforcing plates, and penetration anchors, are designed to accommodate all design loads and deformations without loss of structural or leaktight integrity. Effects such as temperature and shrinkage are considered.

Temporary or permanent brackets and attachments are designed to resist the design loads without loss of the liner integrity due to excessive deformation from bracket or attachment loads.

The design of penetration sleeves, not backed by concrete and designated as Class MC components, as defined in Section 3.8.1.1, is covered in Section 3.8.2.

3.8.1.4.2 Assumptions on Boundary Conditions

Basically three structural components are analyzed by assuming that each is in equilibrium with loads applied to it and compatible with deformations at the juncture of the structures. The three structures are:

a) The 130 ft. I.D. Hemispherical dome.

b) The 130 ft. I.D. and 160 ft. high cylindrical wall.

c) The circular foundation mat.

Mathematically, the dome and cylinder are considered as thin-walled shells in the form of surfaces of revolution. The classical theory of thin shells is used to determine both membrane and bending stress resultants due to each individual load, but redistribution of moments and forces is considered due to the cracking of concrete of these statically indeterminate structures, as described in Section 3.8.1.4.4.4.

3.8.1.4.3 Circular Foundation Mat Analysis

The concrete foundation mat which supports the Concrete Containment Structure and the internal structures is designed in accordance with the ASME Section III, Division 2/ACI 359 Code, Winter of 1975 Addenda.

The analysis of the foundation is concerned primarily with the determination of shear and moment in the reinforced concrete foundation mat and the determination of the interaction of the mat with the underlying bearing material.

For this foundation supported by rock, the pertinent requirements of the design are the maintenance of bearing pressures within allowable limits, particularly due to overturning moments, and the assurance that there is adequate resistance to sliding of the structure if it is subjected to lateral loads. The stability of the foundation mat is further discussed in Section 3.8.5.

The design loads considered for the analysis of the foundation mat are the maximum resulting forces from the superstructure due to static and dynamic load combinations and those loads directly applied on the base slab, such as dead, live, hydrostatic, internal pressure, temperature, and equipment loads.

In the analysis, the foundation mat is treated as a plate supported on an elastic foundation; the finite element method of analysis is used, employing proven, industry accepted computer programs. The subgrade modulus considered in the analysis is determined by using appropriate correlations with the engineering properties of the foundation materials used at the site, as described in Section 3.7.2.4.

The containment and internal structure walls supported by the foundation mat are represented by force boundary conditions and appropriate nodal displacement restraints in the finite element mathematical model.

The rock foundation is simulated by discrete springs acting at the grid points of the mat elements. For the initial step of analysis all springs are assumed active. The resulting forces in the springs for the critical load combinations indicate which springs are under tension and should be eliminated.

Thereafter, the second step is commenced by deactivating the resulting tension springs for the most severe load combinations. The final check of the assumed spring supports is required to demonstrate that no tension exists in the springs. The analysis then supplies the required forces and moment for the final design of the mat.

The STARDYNE computer program is used for this analysis (see Appendix 3.8B).

The cavities in the mat are analysed together with the base mat. Figure 3.8.1-26 shows the foundation mat shear forces and bending moments for the most critical load combinations, which govern the design.

3.8.1.4.4 Cylindrical Wall and Dome Analysis

The analysis of the hemispherical dome and cylinder due to axisymmetric loads, such as accident pressure, test pressure, gravity, and temperature loads, is based on the primary membrane theory. In addition, the local bending moment and radial shear in the vicinity of the dome-cylinder juncture, as well as the cylinder-base juncture, are also analyzed by applying the condition of compatibility at the junctures. The analytical procedure and formulations are based on those contained in References 3.8.1-42, 3.8.1-43, 3.8.1-44, and 3.8.1-45. The change of the sectional properties due to cracking (or partial cracking) of concrete under accident pressure and test pressure are considered in the analysis.

The expansion of the liner in the dome and cylinder due to an increase in temperature creates tension in the reinforcing and compression in the liner. Compatibility of these thermal strains, and the mechanical strains due to pressure, are accounted for in the analysis.

The effects of concrete cracking are carefully considered in the containment analysis. The following three types of cracking are considered:

a) Membrane crack, an axisymmetrical crack or a crack formed around the whole circumference. This crack could result from internal pressure loading, or from internal pressure combined with other unsymmetric loads.

b) Local membrane crack, an unsymmetrical local crack constituting only a part of the circumference. This crack could result from a seismic load in the normal operating condition.

c) Partial bending crack, only a portion of the section (along the thickness) is assumed to be cracked. For example, a horizontal crack due to the discontinuity moment at the lower portion of the containment wall under the accident pressure load is considered to be a partial bending crack.

3.8.1.4.4.1 Treatment of Axisymmetric and Non-axisymmetric Loads

The concrete containment structure loading cases under axisymmetric loads are analyzed by the finite element method, by using the beam on elastic foundation approach to represent the actual cylindrical shell of revolution, and taking into consideration the effects of cracking of concrete, as described in Section 3.8.1.4.4.4. A three-dimensional model, using the STARDYNE computer program, was also used to verify the results of the Ebasco computer program.

The Concrete Containment Structure loading cases under non-axisymmetric loads are analyzed by the finite element method, using a three-dimensional finite element model and industry proven computer programs, as described in Section 3.8.1.4.4.4.

3.8.1.4.4.2 Treatment of Transient Loads

As presented in Section 3.8.1.3.1 c), during normal operating conditions of the plant a linear temperature gradient across the containment wall thickness develops, with the inside face of the wall subjected to an operating temperature of 120 F and the outside face of the wall subjected to a temperature of 90 F and 20 F in summer and winter conditions, respectively.

The normal operation thermal loads are determined considering the thermal gradients in summer and winter which are adjusted by subtracting the construction temperature from the surface temperatures for the thermal input into the containment analysis.

The design accident thermal loads consist of normal operation thermal gradient and the temperature increment generated by the postulated accident.

As described in Section 3.8.1.3.1i), the accident temperature mainly affects the liner, rather than the concrete and reinforcing bars, since the concrete has a much lower thermal conductivity than the steel liner and the accident temperature drops off very rapidly. Therefore the accident thermal increment cannot penetrate very far into the concrete, as evidenced by numerous transient thermal analyses. Thus, at the moment of the higher accident temperature, the Containment is subjected to a "skin temperature effect" imposed by the liner plate. Due to the interaction between the concrete wall, subjected to a steady-state temperature gradient, and the liner plate, subjected to a temperature of 233 F, increased stresses induced into the reinforcing steel and into the concrete are determined.

3.8.1.4.4.3 Treatment of Localized Loads

The Concrete Containment Structure is designed for localizing loads, such as jet impingement loads and tornado generated missile loads.

The Concrete Containment Structure is designed to withstand, without loss of function or perforation, a representative tornado-driven missile spectra as described in Section 3.5.1.4, using the combinations of loads listed in Section 3.3.2.2.4 and in Table 3.8.1-2.

An impactive dynamic analysis is performed in order to investigate the following aspects of the problem:

a) The penetration of the target by a missile, local damage to the impact area, estimation of the depth of penetration, and the potential generation of secondary missiles by spalling or scabbing, as described in Section 3.5.3.1.

b) The structural response of the member, the overall response of the structure to missile impact, assuming acceptable ductility ratios and estimates of forces, moments, and shears induced in the structure by the impact force of the missile to check for structural integrity, as described in Section 3.5.3.2.

A three-dimensional finite element model is used for this investigation, with the tornado-generated missile concentrated load applied independently at different locations on the outside face of the containment wall in order to determine the equivalent spring constants, equivalent masses, and the natural frequencies of the equivalent simplified dynamic models used in the investigation of the structural responses.

3.8.1.4.4.4 Effects of Cracking of Concrete

The following considerations are included in evaluating the effects of concrete cracking:

a) Analysis for Axisymmetric Loads - When the Containment is subjected to axisymmetric loads, the shell is analyzed by the methods specified below. The accident pressure is the load that causes membrane cracks in the shell and partial bending cracks at the boundaries, as described in Section 3.8.1.4.2. The membrane stress resultants are not affected by the sectional properties of the shell; however, the boundary discontinuity moments are affected by the sectional properties of the shell. Since this is a non-linear material problem, an iteration process is employed to obtain reliable results.

The containment crack modeling is shown on Figure 3.8.1-27. The containment analysis used to account for section property variations and changes due to concrete cracking is a finite element analysis which includes the beam on elastic foundation approach to represent the actual cylindrical shell of revolution. The finite element analysis and the Ebasco computer program used in the analysis are described in Appendix 3.8B. The finite element analysis consists of the following procedures:

1) The meridian and circumferential membrane force resultants which are independent of the sectional properties are first calculated by the classical membrane theory.

$$N_x = \frac{PR}{2} \qquad (1)$$

$$N_\theta = PR \qquad (2)$$

Where P = pressure load, PSF
R = Radius of the containment in ft.
N_x = Meridian membrane force in k/ft.
N_θ = Circumferential membrane force in k/ft.

2) The radial displacements are calculated by the membrane theory, with a free boundary condition and completely cracked section.

$$d_i = P/K_i \qquad (3)$$

$$K_i = \frac{Et_i}{R^2} \qquad (4)$$

Where d_i = free boundary radial displacement for i^{th} element.

E = Young's modulus

t_i = Equivalent thickness of the reinforced steel for i^{th} element.

K_i = Shell equivalent modulus of elastic foundation.

3) At the vicinity of the boundary, where discontinuity moments and radial shear develop, the axisymmetrical bending theory is used and its closed form solution (see Reference 3.8.1-45) is employed to construct the flexibility matrix. It is shown on Figure 3.8.1-27 that a finite number of elements can be subdivided, each of which may be assigned different sectional properties based on the presumed compression uncracked zone. The equation is written in matrix form;

$$[f]\{F\} = \{d\} \qquad (5)$$

where:

[f] is the flexibility matrix size 2N x 2N (detailed in Appendix 3.8.B)

{F} is the generalized forces, including 2N elements.

{d} is the relative incompatible displacement, which is obtained as described in equation (3) above.

4) After the shears and moments are computed from equation (5), the total moments and meridianal membrane forces for each specific loading combination are obtained by summing up all the moments and meridianal membrane forces due to the individual factored loads.

5) When the total meridianal membrane forces and moments at each node are determined, the compression zone at each node point is computed to check with the presumed compression zone at each node point. If they are sufficiently close, the iteration process is completed and the final stresses are reached. If they are not close, another trial is attempted.

6) Superposition is not valid in this process; complete cycle iteration is performed for each load combination case.

b) Analysis for Unsymmetric Loads - When the Containment is subjected to unsymmetric loads (seismic and wind loads), the stress resultants of major concern are the vertical (meridian) membrane and the tangential shear. There are local bending moments which are considered to be minor. The type of cracks expected is dependent upon the load combinations. Both membrane cracks and local membrane cracks could develop. Membrane cracks could form under accident conditions and local membrane cracks could form under normal operating conditions.

The structural analysis for unsymmetric loads is performed by using a finite element computer program which has been developed primarily for analyzing uniform and isotropic linear elastic material. For the accident condition with membrane cracking and a uniform section, the major analysis results are reliable. In the normal operating condition with local membrane cracks, the results are affected by discrepancies in the sectional properties. However, the shear forces and bending moments developed in the Concrete Containment Structure due to the axisymmetric loading conditions are less than 10 percent of the shear forces and bending moments developed in the concrete cylindrical wall due to the axisymmetric loading conditions generated during the postulated accident. Therefore the discrepancies in the sectional properties for the normal operating conditions are insignificant.

A three-dimensional finite element approach is used to analyze the hemispherical dome and cylinder due to non-axisymmetric loads such as wind, tornado, and seismic loads. The CDC "ANSYS" or "STARDYNE" finite element computer program is used to perform the analysis (see Appendix 3.8B). Elements are refined at the vicinity of the junctures where change of stress resultants are expected. These programs are developed based on a linear material properties assumption. No iteration is performed to consider concrete cracking automatically. Therefore, the cracked sections are predicated as an input to account for concrete cracking.

An equivalent thickness of the shell is used to modify concrete cracking. The stress resultants which are used in the design are not significantly affected by the change of section rigidity.

A comparative study was performed using the finite element analysis (described in the axisymmetric load analysis) and other industry proven computer programs such as ANSYS and STARDYNE. Figure 3.8.1-28 shows the results of the comparative study. ANSYS was used for the computer program analysis of the polar crane region because the polar crane runway girder and support brackets were represented in the ANSYS model. The design of the containment structure wall, dome, and penetrations used the results from the STARDYNE output because the penetrations were included in the STARDYNE model. The in-house finite element analysis program was used to verify the results obtained for the design of the cylindrical wall.

The containment STARDYNE model used triangular plate elements for the static analysis of the building. The elements were assumed to be homogeneous and isotropic. For that type of element, two factors determine the element properties: the modulus of elasticity (E) of the material of the element and
14 the element moment of inertia (I) derived from the thickness of the element.

Cracked section properties were accounted for in the model by modifying the value of E in the inputs such that the product EI corresponded to the cracked condition of the wall at the location of the element. This was done to the EI for both the vertical and horizontal directions, using the cracks determined from the cracking analysis by the SHELL computer program. For the vertical direction, the wall was divided into zones, and the average crack size for zone was used for the zone. 14

Figure 3.8.1-27 illustrates the wall finite element model and modeling of cracks.

Figures 3.8.1-29 through 3.8.1-31 show the cylindrical wall and dome shear forces, bending moments, and displacements from the most critical load combinations, which govern the design.

The Concrete Containment Structure is a conventionally reinforced concrete structure in which shrinkage tends to develop stresses in a reverse direction from that developed by the design basis accident; therefore shrinkage is not considered in the design. During construction of the containment structure, construction techniques, as described in Section 3.8.1.6.3 (a), are used in order to minimize the effects of shrinkage.

3.8.1.4.4.5 Description of the Computer Programs Utilized

Descriptions of the computer programs utilized in the analyses and design of the Concrete Containment Structure are presented in Appendix 3.8B. Basically they are industry proven computer programs, such as STARDYNE, NASTRAN, and ANSYS. For the dynamic analysis of the containment structure, the STARDYNE computer program was used for the three-dimensional dynamic model and an Ebasco computer program was used for the two-dimensional dynamic model. The Ebasco computer program is described in Appendix 3.8B.

The finite element computer program used to account for the effects of concrete cracking is an Ebasco computer program, which uses the beam on elastic foundations approach to represent the real cylindrical shell; it is described in Appendix 3.8B. In order to demonstrate that the results obtained by using this computer program are substantially identical with the results obtained by using industry proven computer programs, a comparative study was performed, as described in Section 3.8.1.4.4.4; the results are presented on Figure 3.8.1-28.

3.8.1.4.4.6 Treatment of the Effects of Induced Shears

a) Tangential Shear - The tangential shear force, V_u, is due primarily to earthquake, wind, or tornado loading. For earthquake loading, the tangential shear force is determined from the square root of the sum of the squares of the multiple components of earthquake loading. For wind or tornado loading, the tangential shear forces are determined based on the direction of loading under consideration and are compatible with the determination of N_{he} and N_{ve}, defined in this Section.

The criteria for tangential shear are as follows:

1) All membrane forces, including thermal effects, N_{ht} and N_{vt}, are considered.

2) The allowable tangential shear force, V_c, is defined in Section 3.8.1.5.1.1.C)2).

3) The meridional and hoop reinforcing with or without diagonal reinforcing is proportioned for the vertical and horizontal forces respectively plus that portion of the shear force not carried by the diagonal reinforcing.

4) When diagonal reinforcing is required by Section 3.8.1.5.1.1.C)2) the following equations are used for a four (4) way reinforcing system with 45° inclined bars, for factored load combinations presented in Table 3.8.1-2:

$$A_{sh} = \frac{N_h + [N_{he}^2 + V_u^2]^{1/2}}{.9f_y} \qquad (6)$$

$$A_{sv} = \frac{N_h + [N_{ve}^2 + V_u^2]}{.9f_y} \qquad (7)$$

$$A_{si} = \frac{V_u - V_c}{0.9f_y} \qquad (8)$$

where:

A_{sh} = area of reinforcing steel in the horizontal direction (in.^{2}/ft.)

A_{sv} = area of reinforcing steel in the vertical direction (in.^{2}/ft.)

A_{si} = area of reinforcing steel in the inclined direction (in.^{2}/ft.)

Nv and Nh = Membrane force in the horizontal and vertical direction due to loads other than earthquake, wind, and tornado (such as pressure and dead load).

N_{ve} = Membrane force in the vertical direction due to earthquake, wind, or tornado loading. When considering earthquake loading, the force is based on the square root of the sum of the squares of two horizontal and one vertical component of earthquake loading. When considering wind or tornado load, the force is based on the absolute sum of the horizontal and vertical components of loading. The force is always considered as positive.

N_{he} = Membrane force in the horizontal direction due to earthquake, wind, or tornado loading. The forces are determined on the same basis as N_{ve}. The force is always considered as positive.

f_y = Specified tensile yield strength of reinforcing steel, psi.

V_u = Maximum tangential shear at the section under consideration.

V_c = Tangential shear force carried by the concrete. The strain compatibility of the concrete and reinforcing system along the minor principal axis (concrete compression strut) may be used in verifying that the strain in the tension diagonal does not exceed the strain allowable of $2E_y$.

5) When diagonal reinforcing is not required, the following equations are used for factored load combinations presented in Table 3.8.1-2.

$$A_{sh} = \frac{N_h + N_{he} + V_u}{0.9f_y} \qquad (9a)$$

$$A_{sv} = \frac{N_v + N_{ve} + V_u}{0.9f_y} \qquad (9b)$$

6) For service load combinations presented in Table 3.8.1-2, the equations (6) through (9) are used to design the meridional hoop, and inclined reinforcing steel, but 0.9 f_y is replaced by the reinforcing stress allowable listed in Section 3.8.1.5.2.2 and V_u is replaced by V, the applied shear load at the section under consideration.

b) Radial Shear - An example of this type of shear is the shear force caused by self-constraint of a cylinder and base slab during pressurization of the Containment, V_u.

1) Factored Load Design - The nominal shear stress, v_u, is computed by:

$$v_u = \frac{V_u}{0.85bd} \qquad (10)$$

where: d = Distance from the extreme compression fiber to the centroid of the tension reinforcement, in.

b = Unit length of section.

When shear reinforcement perpendicular to the containment surface is used, the required area of shear reinforcement is not less than:

$$A_v = \frac{(v_u - v_c)bs}{fy} \qquad (11)$$

where: s = Spacing of shear reinforcement in a direction parallel to the longitudinal reinforcement. The perpendicular shear reinforcement is not spaced further apart than 0.50d.

v_c = Nominal permissible shear stress carried by concrete, psi, as defined in Section 3.8.1.5.1.1 (C).

When inclined stirrups are used, the required area is not less than

$$A_v = \frac{(v_u - v_c)bs}{f_y\ (\sin\alpha \pm \cos\alpha)} \qquad (12)$$

When shear reinforcement consists of a single bar or a single group of parallel bars, all bent upward at the same distance from the support, the required area is not less than

$$A_v = \frac{(v_u - v_c)bs}{f_y\ (\sin\alpha)} \qquad (13)$$

in which $(v_u - v_c)$ does not exceed $3\sqrt{f'_c}$

Amendment No. 1

where f'_c is the specified compressive strength of concrete.

When shear reinforcement consists of a series of parallel bent-up bars, or groups of parallel bent-up bars at different distances from the support, the required area is not less than that computed by equation (13).

Only the center three fourths of the inclined portions of any longitudinal bar that is bent is considered effective for shear reinforcement.

Where more than one type of shear reinforcement is used to reinforce the same portion of the web, the required area is computed as the sum of the various types separately. In such computations, vc is included only once. The value of $(v_u - v_c)$ does not exceed $8 \sqrt{f'_c}$.

Inclined stirrups and bent bars are spaced so that every 45-degree line extending toward the reaction from the mid-depth of the member, 0.50d, to the longitudinal tension bars are crossed by at least one line of shear reinforcement.

Shear reinforcement extends to at least a distance, d, from the extreme compression fiber and is anchored at both ends to develop the design yield strength of the reinforcement.

2) Service Load Design - The same requirements stated in Section 3.8.1.5.1.1C)2) are used to design shear reinforcement for service loads, with the following modifications:

2 a) Equation (10) is replaced by $v_c = \frac{V_e}{bd}$ (14)

b) The reinforcement steel allowable stress from ASME Code Section III, Division 2/ACI 359 Code CC-3032.1 replaces f_y in Equations (11), (12), and (13).

3.8.1.4.4.7 Variation in Physical Material Properties

The basic assumptions used in the static analysis are in accordance with the ASME Section III, Division 2/ACI 359 Code. Quality control assures that material properties are within the ranges of values anticipated by the analysis and the ASME Section III, Division 2/ACI 359 Code.

In addition, the safety factors included in the allowable stresses provide a safeguard against small adverse variations in material properties and strength.

The effects of the penetrations of the containment shell are taken into account by utilizing a finite element technique to determine the increased forces and moments of the shell in the area of the penetrations. The redistribution of stresses due to containment concrete cracking is also investigated.

Variations in the foundation rock parameters have a negligible effect on the overall analysis of the structure for combined loads since the seismic loads used in the analysis are based on the most critical rock properties.

Concrete temperatures do not exceed the values indicated in the ASME Code Section III, Division 2/ACI 359 Code, Section CC-3440 (a), for normal operation and Section CC-3440 (b) for accident condition.

3.8.1.4.4.8 Treatment of Large Thickened Penetration Regions

Large openings are provided for the equipment hatch, personnel airlocks, main steam penetrations, and feedwater penetrations. In all of these areas, the thickness of the wall is increased from 4 ft. 6 in. to 6 ft. 6 in. in order to accomodate the concentration of stresses and to allow the introduction of additional reinforcement required by special analysis.

All of the large penetrations are incorporated into a three-dimensional finite element model in which a finer mesh around the penetrations is provided in order to obtain reliable stress information. The effect of eccentricity due to the fact that the increase of wall thickness is extended only on the outside face of the wall is considered. The STARDYNE computer program is used for this analysis and the investigation is performed for all load combinations listed in Table 3.8.1-2.

As described in Section 3.8.2, the interaction between the cylindrical concrete wall and steel penetrations is considered and the interaction forces are introduced at the nodal points around the openings.

To account for the effects of concrete cracking, the cracking pattern determined in the finite element analysis described in Section 3.8.1.4.4.4 is used as an input in the finite element analysis used for the large openings.

The results of the analysis include biaxial bending moments and shears, axial force, and torsion. These are used in the design of the reinforced concrete around the penetration openings. Conventional reinforcement, consisting of circular bars around the openings for moments and tensions and stirrups for shear and torsion, is provided.

3.8.1.4.4.9 Liner Plate Analysis and Liner Anchorage System

The purpose of the liner plate is to provide a leak-tight membrane. As such, it is not designed as a component of the Containment to resist design loads, but the stresses and strains in the liner are determined considering the wall and liner as a composite section to assure that the leak-tight integrity of the Containment is not jeopardized.

An anchorage system consisting of headed studs is used to retain the liner and concrete shell as a composite section. The studs are fusion-welded to the liner plate. The headed studs are 5/8 in. diameter by 4 in. long. The mat liner is anchored by welding it to embedded steel members which are anchored in the concrete mat. At the mat-wall intersection, the vertical wall liner is continuously welded to the mat liner.

The liner is analyzed for the loads and load combinations shown in Table 3.8.1-2, except that all load factors in all factored load combinations are equal to 1.0. The calculated stresses and strains do not exceed the values shown in Table 3.8.1-3.

The size and spacing of liner anchorages are chosen such that the response of the liner will be predictable for all of the loads and load combinations shown in Table 3.8.1-2. The anchorage system is designed to accommodate the design in-plane shear loads or deformations exerted by the liner and loads applied normal to the liner surface. The forces and displacements do not exceed the allowables listed in Table 3.8.1-3. The containment vacuum load of 2 psi, with a load factor of 1.0, is considered in combination with other loads. Liner anchorages and welds are designed to withstand this load condition.

In general, the design of the liner is not fatigue-controlled, since most stress and strain changes will occur only a small number of times and produce only minor stress-strain fluctuations. Earthquake and design basis accident strains occur too infrequently, and with too few cycles, to generally be controlling. Nevertheless, because of the critical nature of the liner, the design assures the suitability of the liner for the following specific operating conditions involving cyclic applications of load and the thermal condition specified in the design specification for the Containment.

The fatigue evaluation of the liner considers the following cyclic loading conditions:

a) Thermal cycling due to variations of temperature between cold shutdown and operating conditions of the reactor. The number of cycles is postulated as 500 in 40 years.

b) Thermal cycling due to variations of temperature between summer and winter operating conditions. The number of events for 40 years was considered.

c) Thermal cycling associated with the postulated LOCA is one event.

The fatigue methods and limits established by ASME Boiler and Pressure Vessel Code Section III, Division 1, Article NB 3222.4 apply.

Since the liner is anchored at relatively close intervals compared to its diameter, the analysis is based on plate or beam theory, as appropriate.

The anchor studs are analyzed assuming that the liner remains elastic under all conditions, that the liner strains are converted to stresses using Hook's Law, and that the modulus of elasticity and Poisson's ratio do not exceed yield.

The anchor design and analysis considers the effects of the following:

a) The unbalanced loads resulting from variations of liner curvature. Some areas of the liner may have inward curvature between the anchors, whereas other areas may have outward curvature. The variations result in shear load and displacement at the anchor;

b) Liner thicker than nominal due to the rolling tolerances given in SA-20. The thicker plate may impose greater forces and displacements on the anchorage system than a nominal thickness liner;

c) Yield strength higher than the minimum specified due to the rolling processes and biaxial loading;

d) Weld offset, structural discontinuities, and concrete voids behind the liner;

e) Variation in anchor spacing;

f) Variation in anchor stiffness due to variations of the concrete modulus;

g) Local concrete crushing in the anchor zone; and

h) Stud anchors that are designed to fail before tearing the liner.

Due to the nature of the loading and the types of components, the allowable capacity of the components is specified in terms of stresses and strains for liner plate and in terms of forces and displacements for the concrete anchorages.

In order to determine the ultimate capacity (force and displacement) and the spring constants of the anchorages, which are required in the analysis of the liner and anchorage, tests were performed at Lehigh University's Fritz Engineering Laboratory in Bethlehem, Pennsylvania. The anchor studs were embedded into a concrete disc, which was subjected to bending in order to create biaxial tension similar to the actual state of stresses that would exist in the actual containment wall during an accident condition. The anchorages were tested in tension and shear both in the region where there is biaxial tension and in the region where there are no stresses.

The results of the tests are shown on Figures 3.8.1-32 through 3.8.1-35. Figures 3.8.1-32 and 3.8.1-33 show the results for studs subjected to tension and shear, respectively, with concrete in biaxial tension; Figures 3.8.1-34 and 3.8.1-35 show the results for studs subjected to tension and shear, respectively, with the concrete unloaded. The tests show that the ultimate capacity of the anchorages is not influenced by biaxial tension. The slope of the curve for the anchorages tested in the region with biaxial tension is smaller than the slope of curve for the anchorages tested in the region with no stresses. Although the ultimate force and displacement capacity is not changed for concrete in biaxial tension or unloaded, the biaxial tension state has an important impact on the analysis, since the slope of the load deformation curve determines the spring constants used in the analysis.

The results of the test for the biaxial tension state was included in a finite element model to determine the behavior of the liner anchorages interaction. Figure 3.8.1-36 shows the finite element model used for the analysis of the liner plate.

To minimize stresses and strains at the junction between the mat liner plate and the cylindrical wall liner plate, an unanchored 90 degree, free-standing welded connection was selected, as described in Section 3.8.1.1.3.2. The analysis of this connection is performed using a finite element model as shown on Figure 3.8.1-37. The ANSYS computer program is used for this investigation. The results of the investigation are shown on Figure 3.8.1-38.

The 1 in. thick continuous liner plate which supports the crane brackets is anchored to the concrete containment with anchor bars, plates, and headed steel studs. To determine the behavior of the liner plate in this region and the forces induced in different types of anchorages with different structural rigidity, a finite element model is used, as shown on Figure 3.8.1-39. The external loads used as an input in the finite element analysis are the output forces obtained from the special investigation of the crane girder-cylindrical wall interaction described in Section 3.8.1.4.4.12.

Temporary or permanent brackets and attachments connected to the containment liner plate to support mechanical pipe systems or small equipment are designed to resist the design loads without loss of liner integrity due to excessive deformation from bracket or attachment loads. In order to accommodate the additional loads, the liner plate is locally reinforced with additional studs in the area of surface attachments.

Brackets and attachments connected to the liner are designed and analyzed by using accepted techniques in accordance with the AISC Manual for Steel Construction, Part 5, "Design, Fabrication, and Erection of Structural Steel for Buildings." The design allowable stresses for mechanical loads in the
5 construction, test, and normal load categories for brackets and attachments are in accordance with the AISC Manual. For all other categories of loading, brackets and attachments have been sized for the required section strength as specified in Section 3.8.3.3.3. For brackets and attachments which resist external mechanical loads and are not continuous through the liner, the liner stress in the through-thickness direction is taken as one-half of that in the as-rolled direction.

Due to the internal pressure and/or accident differential temperature between the liner surface and concrete wall, the liner plate may be subjected to membrane stresses (tension during the test pressure and compression during accident conditions). As shown on Figure 3.8.1-40, the connection of the pads for mechanical supports induces additional bending stresses into the liner plate. Additional bending stresses are also induced by the locked-in stresses produced during placement of fresh concrete during construction, if the liner plate is used as an internal form. All of the combined membrane and bending stresses are calculated and superimposed in order to verify that the stresses and strains are within the ASME Section III, Division 2/ACI 359 Code limits.

The yield strength of the liner is not exceeded during the test pressure load combination. However, in accident conditions, even without mechanical loads, the combined membrane and bending stresses due to a LOCA associated with the SSE exceed the yield strength capacity of the liner plate.

If the yield strength capacity of the liner is not exceeded, the analysis is a linear problem and the superposition principle is valid. Therefore the stresses induced into the liner plate by the containment structure loading and the stresses induced by the mechanical loads are determined separately and superimposed after that.

When the yield strength capacity of the liner is exceeded, the analysis becomes a nonliner problem and superposition of stresses is not allowed; a unique nonlinear analysis is performed by combining the loads from the containment structure with the mechanical loads induced into the liner, using the plastic theory method.

Using the common practice procedures of plastic design for combined axial load and bending, special diagrams, such as membrane force versus strain, bending moment versus strain, and axial force versus moment capacity were developed for various moment-force ratios (eccentricities). Figures 3.8.1-41 through 3.8.1-43 show the diagrams used in the design of the liner in the inelastic range.

3.8.1.4.4.10 Containment Penetrations Analysis

The penetration assemblies are analyzed using the same techniques and procedures used for metal containments, as described in ASME "Boiler and Pressure Vessel Code" Section III, Division 1, Subsection NE, "Class MC Components". The analysis considers concrete confinement of the penetration sleeves, as described in Section 3.8.2.4.1.

Each penetration is provided with an anchorage system capable of transferring pressure loads and other mechanical loads, such as piping restraints, into the concrete. The design allowables for the penetrations are the same as those used in ASME Section III, Division 1. For penetration nozzles which are not continuous through the liner, the liner stress in the through-thickness direction is taken as one-half of that in the as-rolled direction.

The analysis of containment penetrations, designated as Class MC Components, is presented in Section 3.8.2.

3.8.1.4.4.11 General Design Considerations

Design details of the Concrete Containment Structure for flexure, axial, and shear loads, reinforcing steel design requirements (splicing, development length, and anchorages), reinforcing steel fabrication and construction requirements (spacing, cover, tolerances, and bending), and concrete crack control are in accordance with the requirements of ASME Code Section III, Division 2/ACI 359 Code.

3.8.1.4.4.12 Special Investigations

a) Cylindrical Wall - Crane Girder Interaction - The polar crane girder is supported on brackets attached to the liner plate. The concrete cylindrical wall and the steel crane girder, which have different thermal expansion coefficients and different thermal gradients during various load combinations, could have differential displacements which could induce large bending moments into the concrete wall and excessive stresses into the crane girder. In order to minimize the interaction forces, moments, and thermal stresses, the crane girder is segmented. The supports of the girder are designed to allow free movement of the crane girder due to the differential thermal gradients and to provide seismic restraint at the same time.

A three-dimensional finite element analysis is performed to investigate the interaction between the concrete wall and the steel crane girder and to determine the interactive forces, moments, and shears developed in the crane girder, brackets, liner, liner anchorages, and concrete wall. The ANSYS computer program is used for this analysis.

b) Dome Construction Sequence - During construction of the concrete containment dome, the liner plate is used as a form, sustaining the weight of the reinforcing steel and fresh concrete without additional support.

As shown in Figure 3.8.1-23, placement of the concrete proceeds in successive lifts of 4 to 5 ft. of concrete, with pours up to 20 in. applied symmetrically and continuously around the entire circumference of the dome. The next placement of concrete is added only when the concrete previously placed is strong enough to work in conjunction with the liner plate as a composite section to take the additional construction loads.

As a result of this construction procedure, additional construction locked-in stresses and displacements occur. In order to account for all of the additive stresses and displacements during construction, and to verify that the allowable stresses and strains in the liner are not exceeded, a finite element analysis is performed, using the NASTRAN computer program. In this finite element analysis, each placement of concrete is modeled in order to account for stresses and strains associated with the additional concrete placed.

3.8.1.5 <u>Structural Acceptance Criteria</u>

The Containment is designed to perform within the elastic range for service loads and is essentially elastic under factored loads.

In order to keep the Containment elastic under service loads and below the range of general yield under factored loads, the allowable stresses and strains specified below are used.

Yield strength reduction factors are used to provide stress margins in order to allow for small variations in homogeneity of material and workmanship.

The tabulated values (ϕ) of yield strength reduction factors, contained in Table 3.8.1-4, are defined as non-dimensional stress limits which are used for designing the containment shell structure against those load combinations specified in Table 3.8.1-2, for both service and factored load combinations.

3.8.1.5.1 Allowable Stresses for the Factored Load Category

3.8.1.5.1.1 Concrete Allowable Stresses

a) Concrete Compressive Stresses

1) Primary compressive stresses:

Membrane stress = $0.6\ f'_c$

Membrane plus bending = $0.75\ f'_c$

2) Primary-plus-secondary compressive stresses:

Membrane stress = $0.75\ f'_c$

Membrane plus bending = $0.85\ f'_c$ with a limit of 0.002 strain

The stresses given above in items 1 and 2 are reduced, if necessary, to maintain structural stability.

b) Concrete Tensile Stresses - Concrete tensile strength is not relied upon to resist flexural and membrane tension.

Table 3.8.1-4 shows the strength reduction factors for concrete.

c) Concrete Shear Stresses - Radial, tangential, peripheral, and torsional shears are considered in the design of the containment structure.

1) Radial Shear - An example of the shear caused by self-constraint of the cylinder and base slab during pressurization of the Containment.

(a) The nominal shear stress, v_c, does not exceed the lesser of:

$$v_c = \sqrt{3.5 f'_c} \qquad (15a)$$

$$v_c = 1.9\ \rho\sqrt{f'_c}\ /0.15 + 2500\rho\ (V_u d/M_u) \qquad (15b)$$

for $\rho < 0.015$

where Mu is the applied design load moment at the section under construction.

$$v_c = 1.9\sqrt{f'_c} + 2500\rho\ (V_u d/M_u) \qquad (15c)$$

for $\rho \geq 0.015$

where $(V_u d/M_u)$ does not exceed 1.0

(b) For sections subjected to membrane compression, either Eq (16) or (17) are used, but v_c shall not be larger than the value given by (18):

$$v_c = 1.9\sqrt{f'_c} + 2500p\,(V_u d/M') \qquad (16)$$

where $M' = M_u - N_u\,[(4t-d)/8]$ then M' shall be less than $V_u d$. If M is negative, Eq (18) is used

$$v_c = 2(1 + 0.0005\,Nu/Ag)\sqrt{f'_c} \qquad (17)$$

1 $$v_c = 3.5\sqrt{f'_c}\sqrt{(1 + 0.002\,N_u/Ag)} \qquad (18)$$

When Nu = the axial design load normal to the cross section occuring simultaneously with Vu

Ag = gross area of section

The units for N_u /Ag are psi.

(c) For sections subjected to membrane tension, Eq (19) is used with N_u negative for tension:

$$v_c = 2.0\sqrt{f'_c}\,(1 + 0.002\,N_u/Ag) \qquad (19)$$

2) Tangential Shear - An example is the shear force resulting when the Containment is subjected to earthquake motion.

The allowable tangential shear force is:

$$V_c = v_c bt \qquad (20)$$

Where: t = thickness of concrete section

a) The tangential shear stress, vc, carried by the concrete does not exceed 40 psi and 60 psi for load combinations 11 and 14 respectively, presented in Table 3.8.1-2. When V_u exceed V_c, diagonal reinforcing is provided.

b) The tangential shear stress v_c, carried by the concrete does not exceed 160 psi for load combinations (6) through (9), presented in Table 3.8.1-2. For these load combinations, a meridional and hoop reinforcing system may be used provided that V_u does not exceed $8.5\sqrt{f'_c}$. If V_u exceeds this limit a diagonal reinforcing system is provided.

c) The tangential shear stress, V_c, carried by the concrete does not exceed 80 psi for service load combinations presented in Table 3.8.1-2. For these load combinations a meridional and hoop reinforcing system may be used provided that V_u does not exceed $4.2\sqrt{f'_c}$. If V_u exceeds this limit a diagonal reinforcing is provided.

3) Peripheral Shear

(a) The peripheral or punching shear stress taken by the concrete on the assumed failure surface does not exceed vc as obtained below:

$$v_{ch} = 4\sqrt{f'_c}\ \sqrt{1 + (f_m/4\sqrt{f'_c})} \qquad (21)$$

$$v_{cm} = 4\sqrt{f'_c}\ \sqrt{1 + (f_h/4\sqrt{f'_c})} \qquad (22)$$ 2

where:

V_{ch} = the allowable shear stress on a failure surface perpendicular to a meridional line.

V_{cm} = the allowable shear stress on a meridional failure surface perpendicular to the plane of the shell.

f_m = membrane stress in the meridional direction, compression is positive.

f_h = membrane stress in the hoop direction, compression is positive.

(b) The value of v_c is calculated as a weighted average of v_{ch} and v_{cm}. For a circular failure surface, v_c is the average of v_{ch} and v_{cm}.

The failure surface for peripheral shear is considered to be perpendicular to the surface of the Containment and located so that its periphery is at a distance d/2 from the periphery of the concentrated load or reaction area.

For failure due to impact loads, local areas for missile impact are defined as having a maximum diameter equal to 10 times the mean diameter of the impacting missile, or $5\sqrt{t}$ plus the mean diameter of the impacting missile (where t is defined as the total section thickness in feet), whichever is smaller.

4) Torsion - The shear stress taken by the concrete resulting from pure torsion does not exceed v_{ct} as calculated from the following equation:

1 $$v_{ct} = 6\sqrt{f'_c}\sqrt{1 + \frac{fh+fm}{6\sqrt{f'_c}} - \frac{fm\ fh}{6\sqrt{f'_c}}} \qquad (23)$$

5) Brackets and Corbels - These provisions apply to brackets and corbels having a shear span to depth ratio, a/d, of unity or less. The distance d is measured at a section adjacent to the face of the support but is not taken greater than twice the depth of the corbel or bracket at the outside edge of the bearing area.

(a) The shear stress does not exceed:

1 $$v_u = \left[6.5 - 5.1\sqrt{(N_u/V_u)}\right]\ [1 - 0.5\ (a/d)]\ \times \left\{1 + \left[64 + 160\sqrt{(N_u/V_u)^3}\right]\rho\right\}\sqrt{f'_c} \qquad (24)$$

where $\rho = A_s/B_d$ does not exceed $0.13\ f'_c/f_y$ and N_u/V_u is not taken less than 0.20, and where N_u is the design tensile force on a bracket or corbel acting simultaneously.

(b) When provisions are made to prevent tension due to restrained shrinkage and creep so that the member is subject to shear and moment only, the shear stress does not exceed

1 $$v_u = 6.5\ [1 - 0.5\ (a/d)]\ [1 + 64\ \rho v]\ \sqrt{f'_c} \qquad (25)$$

where $\rho v = (A_s + A_{vh})/bd$ but is not greater than $0.20\ f'_c/f_y$, and A_{vh} does not exceed A_s.

(c) Closed stirrups or ties that are parallel to the main tension reinforcement and have a total cross-sectional area A_{vh} not less than $0.50A_s$ are uniformally distributed within two-thirds of the effective depth and adjacent to the main tension reinforcement.

1 (d) The ratio $\rho = A_s/bd$ is not less than $0.04\ f'_c/f_y$.

d) Concrete Bearing Stresses - Bearing stresses do not exceed $0.6\ f'_c$ except as provided below:

1) When the supporting surface (A_2) is wider on all sides than the loaded area (A_1), the permissible bearing stress on the loaded area may be multiplied by $\sqrt{A_2/A_1}$, but this factor may not exceed two.

2) When the supporting surface is sloped or stepped, A_2 is taken as the area of the lower base of the largest frustum of a right

pyramid or cone contained wholly within the support, with its upper base as the loaded area and side slopes of one vertical to two horizontal.

3.8.1.5.1.2 Reinforcing Steel Allowable Stresses

a) Reinforcing Steel Tensile Stresses

1) Average tensile stress is 0.9 f_y.

2) The design yield strength of the reinforcement is 60,000 psi.

3) The tensile strain may exceed yield when the effects of thermal gradients through the concrete section are included, provided that the temperature-induced forces and moments reduce as yielding in the reinforcement occurs and the increased concrete cracking does not cause deterioration of the Containment. Maximum tensile strain is limited to twice the corresponding yield strain.

b) Reinforcing Steel Compressive Stresses

1) For load-resisting purposes, the allowable stress is 0.9 f_y.

2) The strains may exceed yield when acting in conjunction with the concrete if the concrete requires strains larger than the reinforcing yield to develop its capacity.

Table 3.8.1-4 shows the allowable stresses for reinforcing steel.

3.8.1.5.2 Allowable Stresses for the Service Load Category

3.8.1.5.2.1 Concrete Allowable Stresses

a) Concrete Compressive Stresses

1) Primary compressive stresses (as defined in Section CC-3136 of the ASME Code Section III, Division 2/ACI 359 code)

Membrane stress = $0.3 f'_c$

Membrane stress for load combinations including wind or earthquake = $0.40 f'_c$

2) Primary-plus-secondary compressive stresses (as defined in Section CC-3136 of the ASME Code Section III, Division 2/ACI 359 code)

Membrane stress = $0.45 f'_c$

Membrane plus bending = $0.6 f'_c$

3) Local compression at discontinuities and in the vicinity of liner anchors = $0.6 f'_c$

b) Concrete Tensile Stresses - Concrete tensile strength is not relied upon to resist flexural and membrane tension.

c) Concrete Shear Stresses - The allowable concrete stresses and the limiting maximum stresses for shear and torsion are 50 percent of the values given for factored loads, except for the following, in which 67 percent of the values given for factored loads are used:

1) temporary pressure loads during test conditions.

2) thermal loads combined with other loads, provided that the section thus required is not less than that required for the combination of the other loads in the loading combination.

The computed membrane stress on the gross section resulting from service loads are multiplied by 2 and substituted for N_u/A_g, f_m, or f_h in invoking the provisions of Sections 3.8.1.5.1.1 (C_1, C_3, and C_4).

d) Concrete Bearing Stresses - The allowable stresses for bearing are 35 percent of the stresses given in Section 3.8.1.5.1.1d).

3.8.1.5.2.2 Reinforcing Steel Allowable Stresses

a) Reinforcing Steel Tensile Stresses

1) average tensile stress= $0.5f_y$

The values given above may be increased by 33-1/3 percent when temperature effects or temporary pressure loads during test conditions are combined with other loads.

b) Reinforcing Steel Compression Stresses

1) For load-resisting purposes, the allowable stress is $0.5\ f_y$.

2) The stress may exceed that given in Item b.1 for compatibility with the concrete, but this stress may not be used for load resistance.

3.8.1.5.3 Allowable Stresses and Strains for Liner Plate and Anchorages

3.8.1.5.3.1 Liner Plate Allowable

The allowable stresses and strains of the liner plate for construction, service, and factored loads are presented in Table 3.8.1-3.

3.8.1.5.3.2 Liner Anchors Allowable

The allowable forces and displacements of the liner anchors for service and factored load combinations are presented in Table 3.8.1-3.

3.8.1.5.4 Concrete Containment Design Considerations

Assumptions, details, and procedures used in the design for flexure, axial, and shear loads are in accordance with the requirements of ASME Code

Reinforcing steel requirements regarding splices, development length, hooks, anchorages, and cover are in accordance with the requirements of ASME Code Section III, Division 2/ACI 359 Code, Section CC 3530.

The requirements for crack control are in accordance with Section CC 3534 of ASME Code Section III, Division 2/ACI 359 Code.

Concrete temperatures do not exceed the values indicated in the ASME Code Section III, Division 2/ACI 359 Code Section CC 3430(b) for accident or short-term loading.

Corrosion protection for the reinforcing steel in the containment structure is provided by positioning reinforcing steel to allow clearance between the steel and any concrete face on the containment wall in accordance with ASME Code Section III, Division 2/ACI 359 Code. The alkaline environment of the concrete adequately protects embedded steel parts from corrosion.

Exposed surfaces of the liner walls, domes, air lock, and hatch are protected against corrosion. After suitable surface preparation, a rust-inhibiting base coat is applied. Finish coats are nonmetallic with smooth nonporous surfaces suitable for loss of coolant accident conditions. Surfaces in contact with concrete are not painted because of the alkaline environment of the concrete.

The radiation sources used for design and analysis of the shielding requirements are based on the core power level (2900 MW) for each Unit. These | 2
are given in Section 12.2.1 and include radiation sources for all phases of plant operation including full power operation, shutdown conditions, and refueling operations, and for various postulated accidents. They include the neutron and gamma fluxes outside the reactor vessel, the reactor coolant activation, fission and corrosion product activities, deposited corrosion product sources on reactor coolant equipment surfaces, spent fuel handling sources, and postulated core meltdown sources. In addition, radiation sources for various auxiliary systems are also tabulated.

The Containment is a reinforced concrete structure with a cylindrical wall 4-1/2 feet thick and a 2-1/2 feet thick dome. In conjunction with the primary and secondary shields, the concrete containment structure limits the radiation level outside the Containment from all sources inside the Containment to no more than 0.25 mrem/hr. at full power operation.

The concrete containment structure provides protection to plant personnel from radiation sources inside the Containment following a Design Basis Accident (DBA).

3.8.1.6 Materials, Quality Assurance, and Special Construction Techniques

3.8.1.6.1 Materials

The materials for the Concrete Containment Structure and foundation mat are in accordance with Article CC-2000 of the ASME Code Section III, Division 2/ACI 359 Code, and as specified hereunder. The materials are selected so that they are compatible with both the normal operating environment and the post accident conditions described in Section 3.11.1. Exceptions to the ASME Code Section III, Division 2/ACI 359 Code are listed in Appendix 3.8A.

a) Cement - Cement conforms to the requirements of ASTM C150, Specifications for Portland Cement, Type II, with the exceptions listed in Appendix 3.8A. Cement is produced and tested by the manufacturer at intervals in accordance with ASTM C-150.

In addition to the tests required of the cement manufacturers, the following tests are performed by CP&L, or an organization designated by CP&L, once every six months:

1) ASTM C-114 - Chemical Analysis

2) ASTM C-115 - Fineness of Portland Cement by the Turbidimeter or ASTM C-204 - Fineness of Portland Cement by Air Permeability Apparatus

3) ASTM C-151 - Autoclave Expansion of Portland Cement

4) ASTM C-191 - Time of Setting of Hydraulic Cement by Vicat Needle

5) ASTM C-109 - Compressive Strength of Hydraulic Cement Mortars

6) ASTM C-190 - Tensile Strength of Hydraulic Cement Mortars

During construction, if cement has been in storage at the site for 6 months, the following tests are performed by CP&L prior to further use of the cement to check storage environment effects on the cement characteristics:

7) ASTM C-191 - Time of Setting of Hydraulic Cement by Vicat Needle.

8) ASTM C-109 - Compressive Strength of Hydraulic Cement Mortars (using 2 in. (50 mm) cube specimens)

Table 3.8.1-5 shows the summary of in-process test results for the cement.

b) Aggregates - Aggregates conform to the requirements of ASTM C-33, Specifications for Concrete Aggregate, with the exceptions listed in Appendix 3.8A.

The aggregate is tested by the supplier for gradation and fineness modulus every 500 tons and for specific gravity and absorption every 5000 tons. In addition, aggregate used for concrete for the Concrete Containment Structure is tested by CP&L, or an organization designated by CP&L,

during concrete production for the requirements and respective frequencies tabulated below:

	Requirements	Test Method	Frequency
1)	Gradation	ASTM C136	Once daily during production (*)
2)	Moisture Content	ASTM C566	Twice daily during production
3)	Material finer than #200 Sieve	ASTM C117	Daily during production
4)	Organic Impurities	ASTM C40	Daily during production
5)	Friable Particles	ASTM C142	Monthly during production
6)	Lightweight Particles	ASTM C123	Monthly during production
7)	Specific Gravity and Absorption	ASTM C127 and/or ASTM C128	Monthly during production
8)	Los Angeles Abrasion	ASTM C131 or ASTM C535	Every 6 months
9)	Potential Reactivity	ASTM C289	Every 6 months
10)	Soundness	ASTM C88	Every 6 months
11)	Water Soluble Chlorides	ASTM D1411	Monthly during production

2

(*) Twice daily during production if more than 200 cu.yds. of concrete are placed.

Summaries of in-process test results of aggregate appear in Tables 3.8.1-6 through 3.8.1-11.

c) Water - Mixing water conforms to the requirements of Article CC 2223 of the ASME Section III, Division 2/ACI-359 Code. 2

Water used in concrete mixing is sampled, tested, and analyzed initially for use in trial mixes and monthly thereafter for use in production concrete by CP&L, or an organization designated by CP&L, to assure conformance with the following limits and tests:

1) The mixing water, including that contained as free water in aggregate, does not exceed more than 250 ppm of chlorides as Cl- as

determined by ASTM D512, "Chloride Ion in Industrial Water and Industrial Waste Water." The water-soluable chloride content of the aggregate is established by the methods described in ASTM D-1411, "Water Soluble Chlorides Present as Admixes in Graded Aggregate Road Mixes."

2) Sulfates 1000 ppm Maximum

3) The total solids content of the mixing water does not exceed 2000 ppm as measured by American Public Health Association "Standard Method for Determination of Total Solids."

4) In addition to the above, the water is tested monthly in accordance with the indicated tests.

Test Method	Requirement
ASTM C109	Effect on Compressive Strength
ASTM C191	Setting Time
ASTM C151	Soundness
ASTM D512	Chlorides
APHA 208*	Total Solids

* Standard Methods 14th Edition, 1975, American Public Health Association.

Table 3.8.1-12 shows the summary of in-process test results for water.

d) Admixtures - Where necessary, admixtures are added to entrain air and increase workability, while reducing the water-cement ratio and retarding the initial set time. The particular admixtures utilized are determined by conducting tests to ensure compliance with Article CC 2224 of ASME Code Section III, Division 2/ACI 359 Code.

Admixtures are used for all concrete construction in accordance with the following requirements and tested by the supplier at intervals conforming with ASTM C-260 and ASTM C-494.

1) Air Entraining Agents - Air entraining agents conform to ASTM C-260 and are used in proportions so that air-entrainment specified in ACI-318 is produced, as determined by ASTM C-138, C-233, and C-173 or C-231. In order that proportions may be adjusted to produce the specified percentage of air under varying conditions, the agent is not combined with the cement or other admixtures prior to batching.

2) Water Reducing Agents - Water reducing agents used in the concrete conform to ASTM C-494. Final approval of the admixture is contingent upon satisfactory tests with the cement and aggregates

used in the work. A set retarding, water reducing agent is used during hot weather in accordance with ACI-305.

Flyash, if used in concrete, conforms to ASTM-C-618, Class F, and is tested in accordance with ASTM C-311 for every 100 tons of flyash utilized. Flyash does not exceed 25 percent, by weight, of cement in the final mix. Concrete produced with flyash meets all of the requirements specified for standard concrete.

Table 3.8.1-13 shows a summary of in-process test results for preliminary acceptance tests of the admixtures.

e) Cement Grout - Cement, aggregate, water, and admixtures for grout conform to the requirements stipulated above. The proportions of materials are based upon trial mixes using the same type and brand of ingredients as is used for construction to meet the specified requirements of consistency, shrinkage, and compressive strength. The tests are performed in accordance with ASTM C-109 and Corps of Engineers methods CRD-C-79 and CRD-C-588-76.

f) Concrete - Structural concrete for the Containment and foundation mat is specified to have a minimum design compressive strength of 5000 psi (Class X), or 4000 psi (Class AA), at 28 days after placing. The concrete mixes yield a unit air-dry weight of at least 137 lb. per cu. ft. at 28 days, in accordance with ASTM C-642.

The design of concrete mixes is in accordance with ACI 211.1-74 "Recommended Practice for Selecting Proportions for Normal and Heavy Weight Concrete," and in accordance with Article CC-2232 of the ASME Code Section III, Division 2/ACI 359 Code. The previously specified ingredients are used to obtain material proportions for the specified concrete.

During construction, minor modifications of design mixes may be necessitated by variations in aggregate gradation or moisture content.

Concrete construction procedures, including stockpiling, storing, batching, mixing, conveying, depositing, consolidating, curing, and construction joint preparation are in accordance with the provisions of Article CC-4200 of the ASME Code Section III, Division 2/ACI 359 Code. SHNPP complies with the requirements of NRC Regulatory Guide 1.55, with the clarifications described in Section 1.8.

g) Reinforcing Steel

1) Reinforcing Bars - Reinforcing bars are new billet steel in accordance with ASTM A-615 Grade 60 (60,000 psi minimum yield strength). When called for on the design drawings, weldable grade reinforcing steel in accordance with ASTM A706 is used. The reinforcing steel and Cadweld splice material conforms to the requirements of Article CC-2300 of ASME Code Section III, Division 2/ACI 359 Code, with the exceptions listed in Appendix 3.8A.

Placing and splicing of No. 11 and smaller bars meet the requirements of Article CC-4330 of ASME Code, Section III, Division 2/ACI 359 Code.

At least one full diameter reinforcing steel sample of each bar size is tested by the reinforcing steel supplier for each 50 tons or fraction thereof of reinforcing bars produced from each heat. No specific method of sample selection is imposed upon the reinforcing steel supplier. These samples are tested based upon ASTM A-615 specifications. All requirements of NRC Regulatory Guide 1.15 are complied with and the material also conforms to ASME Section III, Division 2/ACI 359 Code, except as noted in Appendix 3.8A.

All samples are tested for:

- Tensile yield strength
- Tensile ultimate strength
- Elongation in 8 in.
- Unit Weight

Inspections are performed as necessary to verify compliance with specifications.

2) Mechanical Splicing - No. 18 reinforcing bars are spliced with mechanical (Cadweld) splices in accordance with the requirements of NRC
2 Regulatory Guide 1.10, with the clarification described in Section 1.8. The Cadweld inspection program is also in conformance with NRC Regulatory Guide 1.10.

The average tensile strength of the splices are equal to or greater than the specified ultimate tensile strength of the rebar. The minimum acceptable tensile strength of any splice is 125 percent of the specified minimum yield strength for the particular bar size and ASTM specification.

All completed splices are visually inspected at both ends of the splice sleeve and at the tap hole in the center of the splice sleeve. Splices that fail to pass the visual inspection are discarded and replaced, or repaired by welding. Splices that have been discarded are not used for tensile testing.

The splice samples are either production or sister splices for straight bars, and straight sister splices for all curved bars. Selected splices are tested in accordance with the following schedule for each position, bar size and grade of bar and for each splicing crew as follows:

(a) Test frequency where only production splices are tested:

(1) 1 out of first 10 splices

(2) 1 out of next 90 splices

(3) 2 out of the next and each subsequent unit of 100 splices

(b) Test frequency where combinations of sister and production splices are tested:

(1) 1 production splice of the first 10 production splices

(2) 1 production and 3 sister splices for the next 90 production splices

(3) 1 splice, either a production or sister splice, for the next and subsequent units of 33 splices. At least one-fourth of the total number of splices tested are production splices.

Straight sister splices are substituted for production samples for splicing sleeves arc welded to structural steel elements.

To be acceptable, sound nonporous filler metal must be visible for the full circumference at both ends of the splice sleeve and at the tap hole in the center of the splice sleeve. Filler metal is usually recessed 1/4 in. from the end of the sleeve due to the packing material. Such indentation is not considered as a poor fill.

The following reasons constitute cause for visual rejection of splices:

1) Slag in the tap hole where the slag exceeds the thickness of the sleeve's wall.

2) Spongy appearance of the filler metal caused by gas blowout.

3) Void areas for each end of splices in any position exceeding the allowable values tabulated below:

Bar Size	Allowable Void Area (Sq. In.)
9	1.02
10	1.03
11	1.53
14	2.15
18	3.00

Joints which do not meet the visual acceptance standards are rejected and either completely removed and replaced, or repaired by welding.

Welding is done by the manual shielded metal arc (SMA) process.

The welding electrode for joining the reinforcing bar to the
1 splice sleeve conforms to AWS Specification A.5.5 Classification
E 8018-B2 1/8" or 5/32" diameter. Electrodes for joining the splice sleeves to structural steel components conform to AWS A5.1 Classification E 7018.

All rust, scale, oil, grease, dirt, or other foreign substances are removed from the areas to be welded. All degreasing is done by swabbing the weld area with acetone or other approved solvent or cleaner. No residual cleaning compounds are left on the surface prior to welding.

The welding current is direct current with the electrode positive (reverse polarity). The base material is preheated to 300 F minimum and an interpass temperature of 300 F minimum is maintained during welding.

Amperages and voltages are in accordance with electrode manufacturer's recommendation.

All slag, flux, or foreign materials remaining on any bead of welding are removed before laying down the next or successive bead. Stress relieving is not required.

After completion of welding, a visual inspection is made for the presence of cracks, surface porosity, slag inclusions, undercut, and inadequate weld size.

For test sample splices from the Containment Building that fail to meet the tensile test acceptance standards, the following procedures are used:

1) If any production or sister splice used for testing fails to meet the strength requirements and failure occurs in the bar, the cause of the bar break is investigated. Any necessary corrective action affecting splice samples are implemented prior to continuing the testing frequency.

2) If the running average tensile strength of 15 consecutive samples fails to meet the tensile requirements, splicing is halted. The cause(s) of the failure are investigated and the necessary corrective action(s) are taken. When splicing is resumed, the splicing test frequency is started anew.

3) Welded Splices - Welded Splices, if used, comply with Regulatory Guide 1.94.

h) Steel Liner Plate - The fabrication, testing, and examination of the steel liner is in conformance with Articles CC-4500 and CC-5500 of ASME Code Section III, Division 2/ACI 359 Code, with the exceptions listed in Appendix 3.8A.

The steel liner plate is carbon steel conforming to ASTM A 516 Grade 70. This steel has a minimum yield strength of 38,000 psi and a minimum ultimate strength of 70,000 psi with minimum elongation of 21 percent. Liner plates comply with the requirements of the applicable ASME Code material specification for low temperature service. The impact testing minimum requirement is as follows: 2

1) As specified in ASME Code Section III, Division 1, paragraph NE-2320, for procurement performed prior to April 29, 1977. 2

2) As specified in ASME, Section III, Division 2/ACI 359 Code, paragraph CC 2520, for procurement performed after April 29, 1977.

Charpy V-notch specimens (SA-370 Figure 11 - Type A) are used for all impact testing at a maximum temperature of 0 F.

Welding materials (electrodes, filler metals, and/or inserts) are selected in conformance with the code requirements. Only those types of low hydrogen electrodes and combinations of wire and flux that produce welds that at least meet the impact values of the parent material, as specified, are permitted in the construction.

All welding materials are certified (Actual Test Results) to meet the impact test requirements of ASME SFA-5.1. Weld metal test plates are certified to meet impact tests in accordance with the applicable Subsection of the ASME Code Section III, Division 2/ACI 359 Code, employing a maximum temperature of 0 F and using the same material and thickness range as defined by the ASME Code Section III, Division 2/ACI-359 Code. 2

In manual shielded metal arc-welding, the electrodes are of the low-hydrogen type, are analytically compatible with the base metal, and are such that the mechanical properties of the resulting welds meet the full requirements for mechanical properties of the base metal. Electrodes conforming to ASME SFA 5.5, Classification E 7010, are permitted for making test channel attachment welds only. All low-hydrogen electrodes are stored in ovens at 200 to 300 F for approximately 8 hours immediately prior to use. Electrodes removed from storage ovens are not exposed to ambient temperature for more than 4 hours. Electrodes removed from ovens and not used within a 4 hour period are returned to the ovens for 8 hours of redrying at 200 to 300 temperature. The electrode manufacturer's recommended practices are acceptable as an alternate, provided they are proven to yield a moisture content of less than 0.6 percent for E 7018 electrodes when they are consumed.

The procedures, design, methods, and sequence of welding are reviewed prior to performance of welding. All full penetration groove welds made without backing have the root layer gouged, chipped, or ground to sound metal prior to welding the second side. All vertical welding proceeds uphill, except for the following, which can be welded either uphill or downhill:

1) Capping or wash passes

2) Shielded metal arc welding using E 7010 electrodes

3) Double-welded groove joints in the containment liner

4) The remaining weld layers beyond the root of single-welded groove joints in the containment liner.

Prior to welding, all surfaces are properly prepared to be free of oil, grease, rust, pitting, scale, and deleterious matter to ensure satisfactory welding. All protective coatings, if present, are chemically or mechanically removed from all areas within 2 in. of a seam to be welded. Weldable primers, such as Deoxaluminite, need not be removed when welding is performed according to procedures which are qualified for welding over such coatings.

All automatic welding is done by the submerged arc process or the externally supplied gas-shielded arc process. The welds are analytically compatible with the base metal and have mechanical properties that meet the full requirements of the mechanical properties of the base metal.

Preheat at 200 F minimum is applied to all material whose thickness exceeds 1-1/4 in. For material whose thickness is less than 1-1/4 in. preheat at 100 F is applied if the base metal temperature falls below 50 F. The above requirements are minimum unless otherwise specified in ASME Code Section III, Division 2, ACI 359 Code, Table CC-4552-2.

Thermal post weld heat treatment is performed as required by, and in accordance with, the ASME Code Section III, Division 2/ACI 359 Code. Post weld heat treatment procedures are reviewed by the Architect-Engineer. Parts of the liner furnished prior to April 29, 1977, comply with the post-weld heat treatment requirements of the ASME Boiler and Pressure Vessel Code, Section III, "Nuclear Power Plant Components," Subsection NE, Winter 1971 Addenda, which requirements are equal to or greater than those of Division 2 of the ASME Code Section III, Division 2/ACI 359 Code.

All longitudinal and circumferential welds in the liner are full penetration bevel butt type. All welders, welder operators, and welding procedures are qualified in accordance with and meet the requirements of, Section IX of the ASME Code. All accessible seam welds are subject to spot radiographic inspection in accordance with ASME Section III, Division 2/ACI 359 Code, paragraph CC-5531. Butt welds are examined per ASME Code Section III Division 2/ACI 359 Code, paragraph CC-5521. Radiographic examination is performed in accordance with the techniques prescribed in Section V, Article 2 of the ASME Boiler and Pressure Vessel Code, Winter 1971 Addenda for services rendered prior to April 29, 1977, such as the shear key and sump pit assemblies of Units 1 and 2. For services rendered subsequent to April 29, 1977, radiographic examination is performed in accordance with the techniques prescribed in Section V, Article 2 of the ASME Boiler and Pressure Vessel Code, Winter 1975 Addenda.

In addition to seam welds with back-up bars, all non-butt and attachment welds to the Containment, except those welds for the leak chase system, non-load bearing plates, and temporary erection attachments, are examined by the magnetic particle or liquid penetrant test per ASME Code Section III, Division 2/ACI 359 Code, paragraphs CC-5521, CC-5522, and CC-5523. For magnetic particle or liquid penetrant inspections performed prior to April 29, 1977, the procedures and acceptance criteria conform to Appendix VI and VIII of ASME Boiler and Pressure Vessel Code, Winter 1971 Addenda. For magnetic particle or liquid penetrant inspections performed after April 29, 1977, the procedures conform to Section V, Articles 7 and 6, respectively, ASME Boiler and Pressure Vessel Code Winter 1975 Addenda. Acceptance criteria for the magnetic particle or liquid penetrant examination is in accordance with ASME Code Section III, Division 2/ACI 359 Code, Paragraphs CC-5545 and CC-5544, respectively.

The root pass and final weld layer for attachments to the Containment using full penetration tee welds are examined by the magnetic particle or liquid penetrant method. In addition, the completed tee weld, where accessible, is ultrasonically inspected in accordance with ASME Section III, Division 1, Paragraphs NE-5111 and NE-5330.

Those areas of liner plates which are loaded during service by load bearing plates (loaded in the through thickness direction as defined in Paragraphs CC-3740 and CC-3750 of Section III, Division 2) are examined by the straight-beam ultrasonic method in accordance with SA-578 and ASME Code Section III, Division 2/ACI 359 Code, Paragraph CC-2533.

The criteria for workmanship and visual quality of welds is in accordance with code requirements, as well as the following:

1) Each weld has the minimum specified size throughout its full length. Each weld is free of linear defects such as slag, cracks, pinholes, and excessive undercut and rounded indications such as pinholes which exceed the acceptable limit as permitted by Paragraph CL-5544.2. In addition, the layer of welds is free of coarse ripples, arc strikes, irregular surface, non-uniform bead pattern, high crown, and deep ridges or valleys between beads. Controlled peening, except for the root pass and final weld bead layer, has been reviewed and approved.

2) Butt welds are multipass construction, slightly convex, of uniform height, and full penetration.

3) Fillet welds are of the specified size, with full throat and legs of uniform length.

Vacuum box testing of the liner is performed in accordance with the applicable requirements of ASME Code Section III, Division 2/ACI 359 Code, paragraph CC-5000. After completion of a successful vacuum box and radiography tests, and subsequent repair and retesting of any defects found, the welds are covered by test channels as indicated on the design drawings. A test channel strength and simultaneous leakage (pressure decay) test is then performed by applying 51.75 psig air pressure to the test channels for at least two hours, after which all welds are solution film tested. For those cases where a vacuum box test is performed on the liner seam welds, these welds are not solution film tested a second time. Where there is any indicated loss of channel test pressure within the two hour period, not allowed by accepted test procedures, the channel sections under test are determined to contain defects. Such defects are repaired. Compensation for change in ambient air temperature is made if necessary. Leak testing is performed in accordance with the requirements of ASME Code Section III, Division 2/ACI 359 Code, paragraph CC-5535.2.

All testing connections and accessories, as applicable, are permanently left in place with all connections properly sealed.

After fabrication, surfaces are cleaned in accordance with SSPC-SP-1 "Solvent Cleaning" to remove oil, grease, dirt, loose rust, loose mill scale, and other foreign substances if necessary before mechanical cleaning is started.

A shop coating of 7107 Epoxy White Primer as manufactured by Keeler & Long, Waterbury, Conn. is applied by the liner manufacturer to a dry film thickness of 2 to 5 mils, according to the paint manufacturer's instructions, over steel which has been prepared for coating by commercial Blast Cleaning SSPC-SP-6 as described by the Steel Structures Painting Council. In certain instances, SSPC-SP10 "Near White Blast Cleaning" has been permitted in lieu of SSPC-SP-6.

The corresponding topcoat for this primer is applied in the field and consists of Keeler and Long 7475 Epoxy Enamel Topcoat at a dry film thickness of 2.5 to 5.5 mils.

The above coating system meets the criteria outlined in ANSI Standard N512-1974, "Protective Coatings (Paints)for the Nuclear Industry" and ANSI Standard N101.2, 1972 "Protective Coatings (Paints) for Light Water Nuclear Reactor Containment Facilities."

Application of the above coating system meets the intent of ANSI N101.4 "Quality Assurance for Protective Coatings Applied to Nuclear Facilities" and Reg. Guide 1.54 "Quality Assurance Requirements for Protection Coatings Applied to Water-Cooled Nuclear Power Plants."

Test results, as indicated in the following documents, were utilized in the selection of these paint systems:

(a) ORNL-3589 "Gamma Radiation Damage and Decontamination Evaluation of Protective Coatings," By G. A. West and C. D. Watson February, 1965.

(b) ORNL Log Book A7562, 6/27/77.

(c) ORNL-TM-2412 "Design Consideration of Reactor Containment Spray Systems - Part V, Protective Coating Systems," J. C. Griess, T. H. Roco, et al, October, 1970.

(d) Keeler and Long, Inc., Publication 78-0810-1.

The areas in which the above coatings meet specified criteria are as follows:

1) Radiation Resistance - The protective coating system used on the containment liner is resistant to radiation exposures which would result from 40 years of normal plant operation followed by the radiation exposure resulting from a postulated Loss of Coolant Accident with TID-14844 source terms assumed. ANSI Standard N-512-1974 Table 2.1 lists as a guide more than $4.5x10^9$ rads for "severe exposure" radiation resistance. Test results submitted by the above mentioned manufacturer indicate that their referenced coatings have radiation resistances which fall in these ranges.

2) Decontamination Ability - A total decontamination factor of 440 with a percentage activity removal of 99.8 (Ref: ORNL A7562) was achieved by the protective coating system used for the containment liner. Coating systems indicated above meet this criteria using appropriate procedures.

3) Heat Transfer Characteristics - Protective coating systems are required to have a heat transfer coefficient range of 1,000 to 3,000 BTU-mil./hr.-ft.^{2} F. The systems indicated above meet this requirement. Effects of the liner coating systems on containment post-LOCA transients are not significant.

4) Hydrogen Generation - Coating systems indicated above have no zinc in their composition. Consequently no hydrogen generation will result from contact between the containment spray solution and the coatings.

5) Temperature, Pressure, and Humidity Conditions - Qualification testing of the coating systems are performed for the coating manufacturer by an independent laboratory. The procedures used in the qualification tests and the evaluation standards applied to the test are specified in ANSI Standard N-101.2-1972.

The tests performed meet the temperature, pressure, and humidity conditions calculated for the Shearon Harris Nuclear Power Plant post-accident containment.

The completed liner is constructed to the following tolerances:

1) The difference between the maximum and the minimum diameter at a specified elevation does not exceed 0.65 ft. and the radius from the theoretical centerline of the Containment does not have a minus dimension in excess of 2-1/4 in. or a plus dimension in excess of 3 in. These measurements are taken in at least 26 different points at the top of the wind girder. Wind girders are located approximately at the center of each course of plate and are not located more than 10'-0 apart in the vertical direction unless approved by the Architect Engineer.

2) Deviation from a 10 ft. straight edge placed in the vertical direction between circumferential seams does not exceed 3/4 in. Measurements are taken no closer than 12 in. from a welded seam.

3) The maximum deviation from a straight line or from a true circular or spherical form, measured anywhere on the liner in any direction, does not exceed $\pm$ 1/4 in. in a 14 in. span.

4) Elevations are maintained to within 2 in. of theoretical elevations up to and including the spring line of the dome. Penetration positions are within $\pm$ 1 in. tolerances.

5) Flat spots or local out-of-roundness do not exceed 2 in. in 15 ft.

i) Liner Plate Anchorages - Concrete anchor studs for attachment of the liner plate are Nelson studs of low carbon steel ASTM A108 of a grade suitable for end welding to the liner plate, with automatically timed welding equipment.

Welding details, qualifications, and procedures for steel welding are in accordance with AWS D2.0 for requirements for stud welding for services provided prior to April 29, 1977. For services rendered after this date, stud welding meets the requirements of the ASME Code Section III, Division 2/ACI 359 Code.

In order to determine the tensile and shear capacities of the anchors and the stiffness required for the analysis of the liner plate and its anchorages, a test was performed at Lehigh University, by Fritz Engineering Laboratory, Bethlehem, Pennsylvania, Report No. 200.77.477.1. The results of these tests are discussed in Section 3.8.1.4.

The concrete anchor studs used for the connection of the bottom liner plate are bent Nelson studs 3/8 in. diameter x 4 in. long. The concrete anchor studs used for the connection of the cylindrical wall and dome liner are headed Nelson studs 5/8 in. diameter x 4 in. long.

During construction, the following requirements for testing and inspection are observed:

1) Prior to the start of the stud welding operation, two studs are welded in the same general position to separate pieces of material that are of similar thickness and material as the member. After cooling, each stud is bent at an angle of 30 degrees from its original axis by striking the stud with a hammer. If failure occurs in the weld zone of either stud, the procedure is corrected and two additional studs are successfully welded and tested before any studs are welded to the member. The foregoing testing is performed after any change in the welding procedure. If failure occurs in the stud shank, an investigation is made to ascertain and correct the cause before more studs are welded.

2) Studs bent in testing that show no signs of failure are straightened by hammer blows without heating. Studs attached to the embedded angles, structural tees, and liner plate forming the bottom section of the liner are not straightened after being bent for testing.

3) Studs on which a full 360 degrees weld is not obtained are repaired by adding a 3/16 in. fillet weld in place of the lack of weld, using the shielded metal arc process with low-hydrogen welding electrodes.

4) If the reduction in the height of studs as they are welded becomes less than normal, welding is stopped immediately and not resumed until the cause has been corrected.

5) If visual inspection reveals any stud in which the reduction in height due to welding is less than normal, such stud is struck with a lead hammer, or an appproved alternate method, and bent 15 degrees off vertical. Studs that crack in the weld, the base metal, or the shank, under inspection or subsequent straightening, are replaced.

6) For studs fastened to penetration sleeves, the first two studs welded to each sleeve, after being allowed to cool, are bent 30 degrees by striking the stud with a lead hammer or an approved alternate method. If failure occurs in the weld zone of either stud, the stud is removed, the procedure is corrected, and two additional studs are successfully welded and tested on a sister plate before further studs are attached to the sleeve. Two consecutive studs are then welded to the member, tested, and found satisfactory before any more production studs are welded to the sleeves. Subsequently, a 10 percent random sample of the studs on each sleeve are bend tested.

j) Penetration Anchorages and Attachments - For all Type II and Type III penetration sleeves designed in accordance with ASME Code Section III, Division 2/ACI 359 Code, in the portion backed by concrete, the concrete anchorages used to connect the sleeve into the concrete wall are double headed Nelson studs 7/8 in. diameter by 8 in. long.

Fabrication, welding details, and welding qualification procedures for stud welding are in accordance with ASME Code, Section III, Division 2/ACI 359 Code, except as noted ir Appendix 3.8A.

Special anchorages are used for all Type I penetration sleeves and components, such as the equipment hatch, personnel air locks and emergency air locks, designed in accordance with ASME Code, Section III, Division 1, Subsection NE. The special anchorages are fabricated from SA 105 materials using accepted manual welding procedures. Fabrication welding details, qualification, and procedures for welding anchorages in accordance with ASME Code, Section III, Division 1, Subsection NE.

In order to determine the tensile and shear capacity of the concrete anchorages and the stiffness required for analysis of the concrete containment interaction with steel penetrations, a test was performed at Lehigh University, by Fritz Engineering Laboratory, Bethlehem, Pennsylvania, Report No. 200.77.477.2. The results of these tests are discussed in Section 3.8.2.4.

Special attachments are used for the main steam and feedwater penetration sleeves, which are subjected to excessive rupture loads and which are designed in accordance with ASME Code, Section III, Division 1, Subsection NE. The special attachments are fabricated from material similar to the material used for the penetration sleeves. Accepted manual welding procedures are employed.

Fabrication, welding details, and welding qualification procedures for attachment welding are in accordance with ASME Code, Section III, Division 1, Subsection NE.

In order to determine the tensile and shear capacity of the concrete attachments and the stiffnesses required for the analysis of the concrete containment penetration sleeves interaction, tests were performed at Lehigh University, Fritz Engineering Laboratory, Bethlehem Pennsylvania, Report No. 200.77.477.3. The results of these tests are discussed in Section 3.8.2.4.

k) Structural Steel Members and Attachments - Material for liner plate attachments (load bearing), crane brackets, and structural steel members which are attached to the containment liner are in accordance with the ASME Code Section III, Division 2/ACI 359 Code, as described in Appendix 3.8A.

Crane girders, structural steel, stiffener plates, and similar applications not within the scope of the ASME Code conform to the following:

1) Plate material ASTM-A36 or ASTM-A516 GR70

2) Structural Steel ASTM-A36

The following welding inspections are made:

1) All full penetration butt welds are 100 percent radiographed.

2) All full penetration tee welds are tested by magnetic particle or liquid penetrant test of root pass and final weld layer; ultrasonic tests are performed on completed welds where accessible.

3) Fillet welds joining structural members in which either member is greater than 5/8 in. nominal thickness are inspected by liquid penetrant or magnetic particle methods after the final weld layer is applied. All other fillet welds are inspected visually for unacceptable defects using 5X magnification.

4) The above examinations are performed in accordance with the AWS Code specified in Section 3.8.1.2. As an alternate, the above required examination may be performed in accordance with the ASME Code, as follows:

(a) Radiographic, magnetic particle, and/or liquid penetrant examinations may be performed in accordance with the requirements of the ASME Code, Section V and Section III, Division 2/ACI 359 Code, as specified in Section 3.8.1.6, for services after April 29, 1977.

(b) Ultrasonic examination may be performed in accordance with the requirements of the ASME Code, Section III, Divisions 1 and 2 as described in 3.8.1.6.1 h)2) above.

5) All welders, welder operators, and welding procedures are qualified in accordance with either the requirements of the AWS Code or the ASME Code, Section IX, whichever is applicable.

3.8.1.6.2 Quality Assurance

The overall quality assurance program is in accordance with the Engineering and Construction QA program which was approved by the NRC during the Construction Permit review. Materials testing, fabrication, construction, and construction testing and examination are in accordance with applicable provisions of Articles CC-4000 and CC-5000 of the ASME Code Section III, Division 2/ACI 359 Code. The test methods and frequency of testing for concrete and concrete ingredients conform to the requirements stipulated in the ASME Code Section III, Division 2/ACI 359 Code, with the exceptions listed in Appendix 3.8A.

The services of an independent laboratory were obtained prior to commencing concrete work. This laboratory or CP&L produced control mixes with consistencies satisfactory for the work, using the proposed materials, in order to determine suitable mix proportions that are necessary to produce concrete conforming to the specified type and strength requirements.

Proportions for concrete mixes are based on laboratory or CP&L trial batches made of materials specifically approved for use and from which individual water/cement ratio curves were developed. Mix proportions were selected to ensure maximum workability and conformance with the concrete compressive strength requirements.

2 Proportions for the laboratory or CP&L trial batches and the subsequent mix adjustments were in accordance with ACI 211.1, "Recommended Practice for Normal and Heavyweight Concrete."

2 Initially, concrete mix proportions were selected from the appropriate water/cement ratio curves, so that the average compressive strength exceeded f'c, i.e., 5000 psi (Class X) and 4,000 psi (Class A), by 1,200 psi. In addition, proportions were selected so that the air-dried hardened unit weight would not be less than 137 $lb./ft.^3$ and the slump and air content would be 4 in. and 4 to 8 percent, respectively. A maximum slump of 8 in. is permitted if superplasticizer mix is used.

The initial mix proportions were used until sufficient test data (concrete cylinders tested in accordance with ASTM C39) became available and an over-design considerably less than 1,200 psi could be established.

New mix proportions were selected based on the water-to-cement ratio curves modified by field tests and newly established over-design strength so that the requirements of Sub-Subparagraph CC-2232.2(b) of the ASME Code Section III, Division 2/ACI 359 Code are complied with.

Tables 3.8.1-14 through 3.8.1-17 show a summary of the in-process test results for concrete with compression strengths of 5000, 4000, 3000, and 2000 psi.

For concrete used in the Containment Structure, the properties tabulated below are measured - prior to construction - in accordance with the respective specifications and the applicable conditions noted below:

	Property	Specification	Age of Sample(Days)	Temperature (°F)
1.	Slump	ASTM C143	NA	NA
2.	Compressive Strength	ASTM C39	3, 7, & 28	As per ASTM C39
3.	Flexural Strength	ASTM C78	28	As per ASTM C78
4.	Splitting Tensile Strength	ASTM C496	28	As per ASTM C496
5.	Static Modulus of Elasticity	ASTM C469	28	As per ASTM C469
6.	Poisson's Ratio	ASTM C469	28	As per ASTM C469
7.	Coefficient of Thermal Conductivity	CRD-C44	28	As per CRD-C44

8.	Coefficient of Thermal Expansion	CRD-C39	28	As per CRD-C39
9.	Creep of Concrete in Compression (*)	ASTM C512	2,7,28,90 days & 1 yr.	As per ASTM C512
10.	Shrinkage(*) Coefficient (Length change of cement mortar and concrete)	ASTM C157	4,7,14, & 28 days & 8,16, 32, & 64 weeks	As per ASTM C157
11.	Density (Specific Gravity)	ASTM C642	28	As per ASTM C642

* These tests are concurrent with construction. 2

Concrete slump, temperature, air content, and mechanical properties examinations are performed on a common sample to establish conformance with the provisions listed above.

Concrete is sampled at the point of delivery into the forms.

The methods used in sampling, making, curing, and testing the concrete samples, either in the field or in the laboratory, are in accordance with the appropriate ASTM Standards and include, but are not necessarily restricted to, the following standards:

ASTM C172 - Standard method of Sampling Fresh Concrete

ASTM C 31 - Standard method of Making and Curing Concrete Compressive and Flexural Test Specimens in the Field.

ASTM C192 - Standard Method of Making and Curing Concrete Test Specimens in the Laboratory.

ASTM C39 - Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens.

ASTM C567 - Standard Method of Test for Unit Weight of Structural Lightweight Concrete.

ASTM C138 - Tentative Method of Test for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete.

Three-day, seven-day, and 28-day tests are made on 6 x 12 in. cylinders. For each design mix, a correlation between three-day, seven-day, and 28-day strengths is made in the laboratory. Soon after a job starts, a similar correlation evolves for samples of concrete taken from the mixer. After that correlation has been established, the results of the 7-day tests may be used as an indicator of the compressive strengths which should be expected at 28 days. If 7-day tests show compressive strengths that are too low, corrective measures are taken at once without waiting for the results of the 28-day tests.

The number of test cylinders made under various conditions are as follows:

		Min. No. of Test Breaks			
	Cylinders	3-Day	7-Day	28-Day	Extra
1) Until final determination of each design mix for each class of concrete placed in any one day.*					
Each 100 cu. yd. or fraction thereof	14	4	4	4	2
2) For each class of concrete of determined mix placed in any one day					
Each 100 cu. yd. or fraction thereof	4	-	1	2	1

* This is intended to cover only those new design mixes, created by modification of determined design mixes, which have not been proven by the lab tests prior to their placement. The number of cylinders may be reduced to a minimum of four per set if a sufficient number of cylinders (e.g. 100) for the modified design mix has proven the mix to be acceptable.

The extra cylinders are tested if it is necessary to substantiate 7 or 28 day test results.

The concrete cylinders are tested for compressive strength in accordance with ASTM C39. The strength level of the concrete is considered satisfactory if:

a) No individual strength test results falls more than 500 psi below the required class strength at 28 days.

b) The averages of all sets of three consecutive strength test results equal or exceed the required class strength at 28 days.

Each 28-day strength test result is the average of two cylinders from the same sample. The variation between the two cylinders must be not more than five percent of their average. A greater variation requires testing of the third (spare) cylinder to determine the average strength. If the third cylinder strength variation is also greater than five percent of the average, CP&L determines the reason for such a wide variation in test results and rectifies it.

The coefficient of variation for the tests on each mix, as determined in accordance with ACI 214, must not be greater than 15 percent. A greater variation will require a review of concrete batching, mixing, transporting facilities, and procedures to assure a reduction in this coefficient to the required 15 percent or lower.

The slump tests are performed as follows:

a) One slump test is performed for the first batch placed each day, and thereafter for each 50 cubic yards of each class of concrete placed.

b) Slump tests are made on each concrete batch used for test cylinders.

c) Slump tests are made at any time the inspector has reason to suspect that the concrete slumps are not within the allowable tolerances.

The concrete air entrainment content and temperature is taken with each slump test.

The concrete unit weight is determined daily during production, in addition to the slump, air content, and temperature.

The batch plant scales are calibrated to ASTM C 94 standard on a monthly basis.

Mixer uniformity tests to the ASTM C 94 standard are performed initially and every six months.

The evaluation of the test results for concrete are in accordance with ACI 214 and ASME Code Section III, Division 2/ACI 359 Code.

During concrete operations, inspectors at the batch plant witness the mix proportions of each batch delivered to construction, and periodically sample and test the concrete ingredients. The inspectors ensure that a ticket is provided for each batch, which documents the time loaded, actual proportions of the mix, amount of concrete, and the concrete design strength. The cleanliness of trucks and the handling and storage of aggregate are checked by the batch plant inspectors. The concrete batch plant complies in all respects, including provisions for storage and precision of measurements, with ASTM C-94, and National Ready Mixed Concrete Association (NRMCA) - Certification of Ready Mixed Concrete Production Facilities. Water and ice additions, if necessary, are modified as required based on measurements of the moisture content and gradation changes of the aggregate.

Other inspectors at the construction site inspect reinforcing and form placement, make slump tests, make test cylinders, check air content, check concrete temperatures, record weather conditions, and inspect concrete placing and curing. The requirements of Regulatory Guide 1.55 are followed with clarifications described in Section 1.8.

The reinforcing steel bars comply with the requirements of Articles CC-4300 and CC-5300 of ASME Section III, Division 2/ACI 359 Code, with the exceptions listed in Appendix 3.8A. The requirements of Regulatory Guides 1.10 and 1.15, with clarifications in Section 1.8 and Appendix 3.8A, are also followed. 2

The following inspections are performed:

a) Visual inspection of fabricate reinforcement is periodically performed to ascertain dimensional conformance with specifications and drawings.

b) Visual monitoring of in-place reinforcement is periodically performed by the placing inspector to assure dimensional and locational conformance with drawings and specifications.

3.8.1.6.3 Special Construction Techniques

The recommendations of Regulatory Guide 1.107, "Qualifications for Cement Grouting for Prestressing Tendons in Containment Structures," are not applicable to the Shearon Harris containment. The Concrete Containment Structure (CCS) is a steel lined reinforced concrete structure in the form of
2 a vertical right cylinder with a hemispherical dome and a flat base with a recess beneath the reactor vessel, as described in Section 3.8.1.1. No prestressing system is employed in the containment design and construction. However, the following special construction techniques were followed.

a) Concrete construction practices, including stockpiling, storing, batching, mixing, conveying, depositing, consolidating, curing, and the preparation of formwork and construction joints, are in accordance with the provisions of Section CC-4200 of the ASME Code Section III,
2 Division 2/ACI 359 Code with the exceptions listed in Appendix 3.A. The requirements of RG 1.55, with the clarification described in Section 1.8, are also followed. No special construction techniques are utilized in the concrete construction.

In general, concrete lifts in the wall and dome of the containment structure are placed in approximately 10 ft. and 4 ft. high lifts, respectively. Each lift is constructed in not more than 20 in. layers placed at such a rate that concrete surfaces do not reach their initial set before additional concrete is placed. Past experience indicates that the use of properly controlled concrete mixes and placements not exceeding 20 in. high layers, as described above, followed by careful curing at each lift, controls shrinkage sufficiently to provide the necessary stability in the finished concrete.

The cylindrical wall liner is used as an interior form for placing of concrete in the wall. The liner is connected to the exterior form as shown on Figure 3.8.1-44. Additional vertical channels are provided on the inside face to minimize liner stresses due to the placement of fresh concrete.

Calculations are made in order to determine the stresses induced into the liner during construction. These "locked-in" stresses are super-imposed on all other mechanical and thermal stresses induced into the liner, using the load combinations from Table 3.8.1-2.

The dome liner is used as the sole support for placing reinforcing steel and concrete in the dome. Calculations are made and a sequence of operations are devised to allow this practice with assurance that the liner will not be in jeopardy of buckling. The sequence of concrete placement in the dome is shown on Figure 3.8.1-23.

b) Temporary Construction Openings - Temporary construction openings are provided in the cylindrical wall of the containment structure. Construction joints are provided around the openings and the concrete surface is sufficiently roughened for proper interlocking of the concrete. The reinforcement extends into the opening for sufficient length to enable splicing of bars.

The wall around the opening is designed to provide the necessary reinforced concrete beam section to span the opening, and to provide the necessary column section on either side of the opening to transfer the loads to the foundation mat.

3.8.1.7 Testing and In-Service Surveillance Requirements

3.8.1.7.1 Structural Integrity Pressure Test

The Concrete Containment Structure is subjected to a preoperational structural proof test after the Containment is complete, with liner, concrete structures, all electrical and piping penetrations, equipment hatch, and personnel locks in place.

While the SHNPP's Containment is a non-prototype Containment, the structural acceptance test is performed in accordance with the procedures outlined in Article CC-6000 of the ASME Code Section III, Division 2/ACI 359 Code for a prototype Containment as augmented by the provisions delineated in Regulatory Guide 1.18. 15

The internal test pressure is increased from atmospheric pressure to 1.15 times the containment design pressure in five approximately equal pressure increments. The Containment is depressurized in the same number of increments. Measurements are recorded at atmosphric pressures and at each pressure level of the pressurization and depressurization cycles. Concrete crack patterns are recorded at atmospheric pressures and at each pressure level of the pressurization and depressurization cycles. Concrete crack patterns are recorded at atmospheric pressure both before and immediately after the test and at the maximum pressure level achieved during the test. Instrumentation for these tests consists of taut wire extensometers for longer distance and LVDT (linear variable differential transducers) for shorter distances, with automatic data logging systems to measure deflections. Vertical displacements are measured with Invar tapes. The environmental conditions during the test are measured in a manner and to an extent that permits evaluation of their contributions to the response of the Containment. The test is not conducted under extreme weather conditions such as snow, heavy rain, or strong winds.

In order to determine the complete picture of the overall deflection pattern of the Containment, radial and vertical deflections of the Containment are measured in accordance with ASME Code Section III, Division 2/ACI 359 Code, Article CC-6232. The radial deflections are measured at several points along four meridians spaced around the Containment, including locations with varying stiffness characteristics. Vertical deflections of the Containment are measured at the apex and the springline of the dome.

Figure 3.8.1-45 shows the radial displacement measurement locations and Figure 3.8.1-46 shows the vertical displacement measurement locations.

The radial and tangential deflections of the containment wall adjacent to the equipment hatch opening are measured at twelve points, as shown on Figure 3.8.1-45.

The pattern of cracks that exceed 0.01 inch in width before, during, or after the test are mapped in accordance with ASME Code Section III, Division 2/ACI 359 Code, Article CC-6233 near the base-wall intersection, at the midheight of the wall, at the springline of the dome, around the equipment hatch opening, and at main steam and feedwater penetrations, as shown on Figure 3.8.1-50.

Strain measurements in the concrete sufficient to permit a complete evaluation of strain distribution are determined in accordance with the requirements of Regulatory Guide 1.18, and as shown on Figures 3.8.1-47, 3.8.1-48, and 3.8.1-49.

As a minimum, the following responses of the Concrete Containment Structure to pressurization are established by the tests:

a) Yielding of conventional reinforcement does not develop, as determined from analysis of crack width, strain gage, or deflection data.

b) No visible signs of permanent damage to either the concrete structure or the steel liner that can be detected.

c) The deflection recovery 24 hours after complete depressurization is 70 percent or more.

d) The measured maximum deflections at points of maximum predicted deflection does not exceed predicted values by more than 30 percent. This requirement is waived if the 24-hour recovery is greater than 90 percent.

3.8.1.7.2 Initial and In-Service Leakage Rate Tests

Initial and in-service leakage rate tests are discussed in Section 6.2.6.

TABLE 3.8.1-1

DESIGN, PROCUREMENT, FABRICATION AND ERECTION STATUS

OF CONTAINMENT COMPONENTS, PARTS AND APPURTENANCES

		Design	Procurement, Fabrication, Erection Prior to April 29, 1977	Procurement, Fabrication, Erection After April 29, 1977	Ebasco Construction Specification	Stamp Required ASME Sect III Div.2/ACI 359	Report: Data Report Div.2	Report: Stress Report Div.1
Components	Reinforced Concrete Mat	ASME Sect III Div.2/ACI 359	NA	ASME SECT III Div 2/ACI 359	CAR-SH-CH-6	NA	Yes	
	Reinforced Concrete Wall	ASME Sect III Div.2/ACI 359	NA	ASME Sect III Div 2/ACI 359	CAR-SH-CH-6	NA	Yes	
	Reinforced Concrete Dome	ASME Sect III Div.2/ACI 359	NA	ASME Sect III Div 2/ACI 359	CAR-SH-CH-6	NA	Yes	
Parts	Steel Liner	ASME Sect III Div.2/ACI 359	*	**	CAR-SH-AS-1	***	Yes	
	Anchor Studs	ASME Sect III Div.2/ACI 359	*	**	CAR-SH-AS-1	NA	Yes	
	Crane Supports & Brackets	AISC 1970	AISC 1970	**	CAR-SH-AS-1	NA	Yes	
	Equipment Hatch	ASME Sect III Div.1 Subsect NE	*	**	CAR-SH-AS-1	***		Yes
	Personnel Air Lock	ASME Sect III Div.1 Subsect NE	*	**	CAR-SH-AS-1	***		Yes
	Emergency Air Lock	ASME Sect III Div.1 Subsect NE	*	**	CAR-SH-AS-1	***		Yes

2

TABLE 3.8.1-1 (Cont'd)

DESIGN, PROCUREMENT, FABRICATION AND ERECTION STATUS

OF CONTAINMENT COMPONENTS, PARTS AND APPURTENANCES

		Design	Procurement, Fabrication, Erection Prior to April 29, 1977	Procurement, Fabrication, Erection After April 29, 1977	Ebasco Construction Specifcation	Stamp Required ASME Sect III Div.2/ACI 359	Report Data Report Div.2	Report Stress Report Div.1
Parts (cont'd)	Valve Chamber	ASME Sect III Div.1 Subsect NE	*	**	CAR-SH-AS-1	***		Yes
	Type I Penetration Sleeves	ASME Sect III Div.1 Subsect NE	NA	ASME Sect III Subsect NE	CAR-SH-M-54	***		Yes
	Type II Penetration Sleeves	ASME Sect III Div.1 Subsect NE	*	**	CAR-SH-AS-1	***	Yes	
	Type III Penetration Sleeves	ASME Sect III Div.1 Subsect NE	*	**	CAR-SH-AS-1	***	Yes	
	Electrical Penetrations	ASME Sect III Div.1 Subsect NE	NA	ASME Sect III Div.I Subsect NE	CAR-SH-E-28	***		Yes
	Fuel Transfer Tube Penetration Sleeve	ASME Sect III Div.1 Subsect NE	*	**	CAR-SH-AS-1	***	Yes	Yes

TABLE 3.8.1-1 (Cont'd)

DESIGN, PROCUREMENT, FABRICATION AND ERECTION STATUS

OF CONTAINMENT COMPONENTS, PARTS AND APPURTENANCES

		Design	Procurement, Fabrication, Erection Prior to April 29, 1977	Procurement, Fabrication, Erection After April 29, 1977	Ebasco Construction Specification	Stamp Required ASME Sect III Div.2/ACI 359	Report: Data Report Div.2	Report: Stress Report Div.1
	Sump Recircul. RHR Sleeve (Sleeve Nos. 47 & 48)	ASME Sect III Div.1 Subsect NE	*	**	CAR-SH-AS-1	***	Yes	
	Sump Recircul. Cont Spray Sleeve (Sleeve Nos. 49 & 50)	ASME Sect III Div.1 Subsect NE	*	**	CAR-SH-AS-1	***	Yes	
Attachments to Liner	Spray Piping, HVAC Pads	AISC 1970	AISC 1970	**	CAR-SH-AS-1			
	Test Channels and Angles	AISC 1970	AISC 1970	**	CAR-SH-AS-1			
Materials	Concrete	NA	ACI 318-71	ASME Sect III Div.2/ACI 359	CAR-SH-CH-6	Produced and certified in accordance with CC 2000 with exceptions listed in Appendix 3.8A		
Materials (cont'd)	Reinforcing Steel	NA	ACI 318-71	ASME Sect III Div 2/ASI 359	CAR-SH-CH-7A			
	Concrete Embedments	NA	Manuf.Recomm.	ASME Sect III Div 2/ACI 359	CAR-SH-AS-7 and CAR-SH-CH-16			

2

2

TABLE 3.8.1-1 (Cont'd)

DESIGN, PROCUREMENT, FABRICATION AND ERECTION STATUS

OF CONTAINMENT COMPONENTS, PARTS AND APPURTENANCES

	Design	Procurement, Fabrication, Erection: Prior to April 29, 1977	Procurement, Fabrication, Erection: After April 29, 1977	Ebasco Construction Specification	Stamp Required ASME Sect III Div.2/ACI 359	Report: Data Report Div.2	Report: Stress Report Div.1
Measuring Devices (strain, stress, etc.)	NA	Manuf.Recomm.	Manuf.Recomm.				
Waterproofing Membrane	NA	Manuf.Recomm.	Manuf.Recomm.	CAR-SH-CH-12			
Water Stops	NA	Manuf.Recomm.	Manuf.Recomm.	CAR-SH-CH-13			
Mechanical Splices	NA	Manuf.Recomm.	ASME Sect III Div.2/ACI 359	CAR-SH-CH-15			

* Prior to April 29, 1977 for all these items the procurement, fabrication and erection were performed in accordance with ASME Code Sect III Div.1, Subsection NE, Winter 1971 Addendum.

** After April 29, 1977 for all these items the procurement, shipping, erection, shop painting, testing and inspection are performed in accordance with ASME Code Sect. III Div.2/ACI 359, Winter 75 Addendum and Associated Sections of the ASME Code Sect. III Div.1, Winter 75 Addendum.

*** No Stamp; Acceptance based on the Structural Integrity Test. Materials, fabrication and construction, testing and examination in accordance with the Engineering and Construction QA program which was approved by the NRC during the construction permit review.

For status of procurement, fabrication and erection of parts as of April 29, 1977 see Table 3.8A-1 in Appendix 3.8A.

TABLE 3.8.1-2

CONTAINMENT STRUCTURE LOAD COMBINATIONS AND LOAD FACTORS

a) Service Load Combinations

1) Test Pressure

$C = 1.0\ (D + L + Pt + Tt)$

2) Construction

$C = 1.0\ (D + L + To + Hu)$

3) Normal Operating

$C = 1.0\ (D + L + To + Ro + Pv)$

4) Operating Basis Earthquake

$C = 1.0\ (D + L + To + Ro + E + Pv)$

5) Hurricane

$C = 1.0\ (D + L + To + Ro + Hu + Pv)$

b) Factored Load Combinations

6) Operating Basis Earthquake

$C = 1.0D + 1.3L + 1.0(To + Ro) + 1.5E + 1.0Pv$

7) Hurricane

$C = 1.0D + 1.3L + 1.0(To + Ro) + 1.5Hu + 1.0Pv$

8) Safe Shutdown Earthquake

$C = 1.0(D + L + To + Ro + E' + Pv)$

9) Tornado

$C = 1.0(D + L + To + Ro + W + Pv)$

10) Loss of Coolant Accident

a) $C = 1.0(D + L) + 1.5P + 1.0(Ta + Ra)$

b) $C = 1.0(D + L) + 1.0P + 1.0Ta + 1.25Ra$

TABLE 3.8.1-2 (Cont'd)

b) Factored Load Combinations (cont'd)

11) Loss of Coolant Accident with OBE

$C = 1.0(D + L) + 1.25P + 1.0(Ta + Ra) + 1.25E$

12) Loss of Coolant Accident with Hurricane

$C = 1.0(D + L) + 1.25P + 1.0(Ta + Ra) + 1.25Hu$

13) Operating Basis Earthquake, Hurricane, and Flooding

$C = 1.0(D + L + To + E + Hu + Hq)$

14) Loss of Coolant Accident with SSE

$C = 1.0(D + L + P + Ta + Ra + E' + Rr)$

In all combinations, the live load, L, is considered either with full value or completely absent.

In load combinations 10 through 14, the maximum values of P, Ta, Ra, and Rr, including an appropriate load factor to account for the dynamic nature of the load, are used or a time history is performed.

Load combinations 9, 10a, 10b, and 14 are first satisfied without the impulsive loads (P, Rrr, Rrj) or the impactive loads (Wm and Rrm); yield strain and displacement may be exceeded, providing that the energy absorption capability or the resistance function of the structure, limited by one-third or two-thirds of the ductility at failure, are not exceeded when considering the impulse or impact loads, respectively. 2

In all factored load combinations used for the analysis of the liner, all load factors are taken equal to 1.0. 2

TABLE 3.8.1-3

STRESS AND STRAIN ALLOWABLES FOR LINER AND LINER ANCHORS

LINER PLATE ALLOWABLES

	Stress/Strain Allowables*	
Load Combination	Membrane	Membrane Plus Bending
Construction	$f_{st}=f_{sc}=2/3\ f_{py}$	$f_{st}=f_{sc}=2/3\ f_{py}$
Service	$E_{st}=E_{sc}=0.002$ in/in	$E_{st}=E_{sc}=0.004$ in/in
Factored	$E_{sc}=0.005$ in/in $E_{st}=0.003$ in/in	$E_{sc}=0.014$ in/in $E_{st}=0.010$ in/in

* The types of strains limited by this table are strains induced by deformation or constraint.

LINER ANCHOR ALLOWABLES

	Force/Displacement Allowables	
Load Combinations in Table 3.8.1-2	Mechanical Loads**	Displacement Limited Loads***
1 through 9	Lesser of Fa= 0.67Fy Fa= 0.33Fu	$\delta a = 0.25\ \delta u$
10 through 14	Lesser of Fa= 0.9Fy Fa= 0.5Fu	$\delta a = 0.50\ \delta u$

2

** Mechanical loads are those which are not self-limiting or self-relieving with load application.

*** Displacement limited loads are those resulting from constraint of the structure or constraint of adjacent material and are self-limiting or self-relieving.

Legend:

f_{st}, f_{sc}	=	allowable liner plate tensile or compressive stress, respectively
f_{py}	=	specified tensile yield strength of liner
E_{st}, E_{sc}	=	allowable liner plate tensile or compressive strain, respectively
Fa	=	allowable liner anchor force capacity
Fy	=	liner anchor yield force capacity
Fu	=	liner anchor ultimate force capacity
δa	=	allowable displacement for liner anchors
δu	=	ultimate displacement capacity for liner anchors

2

TABLE 3.8.1-4

CONTAINMENT STRUCTURE STRENGTH REDUCTION FACTORS

Item and Stress Category		Service Load Combinations ϕ	Factored Load Combinations ϕ
Concrete Compressive Stress:			
Primary Loads:	Membrane	0.30 or 0.40*	0.60
	Membrane plus bending:	0.45	0.75
Primary Plus	Secondary:		
	Membrane	0.45	0.75
	Membrane plus bending (Note 1)	0.60	0.85
Concrete Tensile Stress:		0	0
Reinforcing Steel Tensile Stress		0.50 or 0.66**	0.90****
Reinforcing Steel Compressive Stress		0.50*** or 0.66**	0.90*****

* Applicable only to load combinations which include either Hu or E loads.

** For load combinations in which temporary pressure loads or temperature effects loads are combined with other loads.

*** The others may exceed 0.5 fy for compatibility with the concrete but this stress will not be used for load resistance.

**** The tensile strain may exceed yield when the effects of thermal gradients through the concrete section are included.

***** The strains may exceed yield when acting in conjunction with the concrete if the concrete requires strains larger than the reinforcing yield to develop its capacity.

NOTE:

(1) The maximum allowable primary-plus-secondary membrane and bending compressive stress of 0.85 f'_c coresponds to a limiting strain of 0.002 in./in. as required by ASME Section III, Division 2/ ACI 359 Code.

TABLE 3.8.1-5

SUMMARY OF IN-PROCESS TEST PRESULTS
CEMENT

Compound/Property	Range Max.	Range Min.	Avg.
Autoclave expansion %	+0.02	-0.02	0.00
Initial set hr.	3:54	1.53	2:43
Final set hr.	5:26	2:45	3:54
3-day strength psi	2930	1975	2453
7-day strength psi	4390	3290	3878
Air content of mortar %	9.0	7.6	8.3
Blaine min.	4040	3819	3864
SiO_2 %	22.8	21.3	21.8
Al_2O_3 %	4.35	3.37	4.05
Fe_2O_3 %	4.15	3.47	3.81
MgO %	2.02	1.28	1.65
SO_3 %	2.72	2.35	2.59
Loss on ignition %	1.64	0.90	1.23
Insol. residue %	0.48	0.27	0.38

1. Preliminary acceptance tests

TABLE 3.8.1-6

SUMMARY OF IN-PROCESS TEST RESULTS
SIEVE ANALYSIS AND FINENESS MODULUS
FINE AGGREGATE (SAND)

Sieve Size	Cumulative Percent Passing Coarsest	Finest	Average	Initial[1]
3/8	100	100	100	100
No. 4	96	100	100	100
No. 8	83	100	100	95
No. 16	51	89	77	70
No. 30	26	59	44	37
No. 50	5	28	12	10
No. 100	0	8	2	3.3
No. 200	.1	2.8	1.00	1.3
F.M.				2.85
Number of Tests				

[1]Preliminary acceptance test

TABLE 3.8.1-7

SUMMARY OF IN-PROCESS TEST RESULTS
FINE AGGREGATE (SAND)

Property	Range Max.	Range Min.	Avg.
Friable particles (%)	0.70	0.0	0.34
Lightweight particles (%)	0.70	0.0	0.16
Absorption (%)	0.96	0.44	0.66
Specific gravity (SSD)	2.67	2.57	2.62
Reduction in alkalinity mm./1	78.1	29.8	54.5
Dissolved silica mm./1	29.8	22.4	27.0
$NaSO_4$ soundness (%)	7.5	1.6	4.5

Five cycles

TABLE 3.8.1-8

SUMMARY OF IN-PROCESS TEST RESULTS
SIEVE ANALYSIS
COARSE AGGREGATES
(1-1/2" GRAVEL

			Cumulative Percent Passing
Sieve Size	Range	Average	Initial[1]
2"	100	100	100
1-1/2"	90-100	95	96
1"	20-50	46	35
3/4"	0-15	14	6
3/8"	0-5	3	1.5
Number of Tests			

[1]Preliminary acceptance tests

TABLE 3.8.1-9

SUMMARY OF IN-PROCESS TEST RESULTS
SIEVE ANALYSIS
COARSE AGGREGATES
(3/4" GRAVEL)

Sieve Size	Range	Average	Cumulative Percent Passing Initial[1]
1"	100	100	100
3/4"	90-100	94	98
3/8"	20-55	25	35
No. 4	0-10	8	8
No. 8	0-5	2	3.5
Number of Tests			

[1]Preliminary acceptance tests

TABLE 3.8.1-10

SUMMARY OF IN-PROCESS TEST RESULTS
AGGREGATE NO. 4 (1-1/4 IN. GRAVEL)

Property	Range Max.	Range Min.	Avg.	Standard Deviation	No. of Tests	Initial[4]
Flat and Elongated (%)	3.18	1.0	1.86	N/A	4	
Friable particles (%)	0.39	0.03	0.19	N/A	4	
Lightweight particles (%)	0.01	0.00	0.00	N/A	4	
Soft Particles (%)	0.90	0.06	0.61	N/A	4	
Absorption (%)	0.56	0.40	0.43	N/A	4	
Specific gravity (SSD)	2.75	2.67	2.70	N/A	4	
L. A. abrasion (%)						
Reduction in alkalinity						
Dissolved silica						
$MgSO_4$ soundness (%)[3]	N/A	N/A	N/A			
$NaSO_4$ soundness (%)[3]						

[1]Insufficient data

[2]Potentially deleterious

[3]Five cycles

[4]Preliminary acceptance tests

TABLE 3.8.1-11

SUMMARY OF IN-PROCESS TEST RESULTS
AGGREGATE NO. 67 (3/4 IN. GRAVEL)

Property	Range Max.	Range Min.	Avg.
Flat and Elongated (%)	22.0	1.0	3.8
Friable particles (%)	0.44	0.0	0.14
Lightweight particles (%)	0.23	0.0	0.02
Soft Particles (%)	1.00	0.0	0.71
Absorption (%)	0.98	0.29	0.57
Specific gravity (SSD)	2.85	2.67	2.76
L. A. abrasion (%)	15.0	11.6	13.5
Reduction in alkalinity	78.4	28.3	53.7
Dissolved silica	41.3	21.6	29.0
$NaSO_4$ soundness (%)	7.6	0.2	3.0

Five cycles

TABLE 3.8.1-12

SUMMARY OF IN-PROCESS TEST RESULTS
WATER

Property	Variance[1] Range Max.	Min.	Avg.	No. of Tests
Initial time of set, vicat (min)	9	1	5.5	20
Final time of set, vivicat (min)	21	0	11.6	20
Autoclave expansion	+0.02	-0.04	-0.01	24
7-day compressive strength (%)	16.8	0.2	3.6	20
	Range Max.	**Min.**	**Avg.**	**No. of Tests**
Chlorides (ppm)	243	0.2	43.9	40
Solids (ppm)	825	18	265.9	40
Sulfates (ppm)	55.6	0.8	6.3	40

[1] Comparison of test water with control water

TABLE 3.8.1-13

SUMMARY OF IN-PROCESS TEST RESULTS
ADMIXTURES

Property	Range Max.	Range Min.	Range Avg.	Range Max.	Range Min.	Range Avg.	Range Max.	Range Min.	Range Avg.
Solids (%)	42.8	41.7	42.2	43.3	40.6	42.2	27.3	27.2	27.3
Specific gravity	1.193	1.190	1.192	1.214	1.192	1.200	1.083	1.081	1.082
pH	7.6	7.2	7.4	7.4	6.4	6.8	12.9	12.8	12.9
Chloride (ppm)			3			6			2

[1] Preliminary acceptance tests

TABLE 3.8.1-14

SUMMARY OF IN-PROCESS TEST DATA
CONCRETE (5000 PSI)

MIX ID		M-72 CLASS AAA	M-71 CLASS AAA
No. Samples			
Plastic Data	Avg. Temp Range	60-85	A
	Avg. Slump Range	2 1/4-33/4	A
	Avg. Air Cont. Range	5.0-7.5	A
	Avg. Unit Wt. Range	141.8-147.2	A
7-Day Strength	Avg. Strength	4420	4810
	Avg. Range	A	A
	C/V Within	A	A
	Std. Deviation		
	C/V Overall	A	A
28-Day Strength	Avg. Strength		6530
	Avg. Range		
	C/V Within		
	Std. Deviation		
	C/V Overall		
90-Day Strength	No. of Test-		
	Avg. Strength		
	Avg. Range		
	C/V Within		
	Std. Deviation		
	C/F Overall		

A—Insufficient Data

TABLE 3.8.1-15

SUMMARY OF IN-PROCESS TEST DATA
CONCRETE (4000 PSI)

MIX ID		M-44 CLASS AA	M-41 CLASS AA	M-63 CLASS AA	M-56 CLASS AA	M-57 CLASS AA	M-58 CLASS AA
No. SAMPLES							
Plastic Data	Avg. Temp Range	60-85	A	A	57.86	76-86	A
	Avg. Slump Range	2 1/2-4 1/2	A	A	2 1/2-4 1/2	2-3 1/2	A
	Avg. Air Cont.Range	3-6.0	A	A	4-8	4.4-7.8	A
	Avg. Unit Wt.Range	14 1/4-148.6	A	A	141.8-148.2	142.3-148.3	A
7-Day Strength	Avg. Strength	3550	2950	3960	3810	3560	3270
28-Day Strength	Avg. Strength	5701	4600	5790	5565	5450	5220
	Avg. Range	166	A	A	165	A	A
	C/V Within	2.57	A	A	2.63	A	A
	C/V Overall	8	A	A	11	A	A

A. Mixes M-41, M-63, M-57 & M-58 are seldom used mixes. Although data on these mixes exists the quantity of data is too low to meaningfully calculate these values.

TABLE 3.8.1-16

SUMMARY OF IN-PROCESS TEST DATA
CONCRETE (3000 PSI)

MIX ID		M-54	M-55
No. Samples		Class B	Class B
Plastic Data	Avg. Temp	61.77	61-81
	Avg. Slump	1 1/2-3 1/2	2-4
	Avg. Air Cont. Range	4.8-7.8	4-6
	Avg. Unit Wt. Range	141.0-147.2	143.2-148.6
7-Day Strength	Avg. Strength	3300	3400
	Avg. Range	A	A
	C/V Within	A	A
	Std. Deviation		
	C/V Overall	A	A
28-Day Strength	Avg. Strength	4978	4905
	Avg. Range	126	136
	C/V Within	2.25	2.46
	Std. Deviation		
	C/V Overall	8	9
90-Day Strength	No. of Tests		
	Avg. Strength		
	Avg. Range		
	C/V Within		
	Std. Deviation		
	C/V Overall		

A---Insufficient Data

TABLE 3.8.1-17

SUMMARY OF IN-PROCESS TEST DATA
CONCRETE (2000 PSI)

MIX ID		M-45
No. Samples		Class D
Plastic Data	Avg. Temp Range	60-80
	Avg. Slump Range	2-3 3/4
	Avg. Air Cont. Range	4.5-7.8
	Avg. Unit Wt. Range	141.9-145.7
7-Day Strength	Avg. Strength	2400
	Avg. Range	A
	C/V Within	A
	Std. Deviation	
	C/V Overall	A
28-Day Strength	Avg. Strength	4156
	Avg. Range	118
	C/V Within	2.53
	Std. Deviation	12
	C/V Overall	
90-Day Strength	No. of Tests	
	Avg. Strength	
	Avg. Range	
	C/V Within	
	Std. Deviation	
	C/V Overall	

A---Insufficient Data

REFERENCES: SECTION 3.8

3.8.1-1 ASME Section III Division 2/ACI 359-75 "Code for Concrete Reactor Vessels and Containments."

3.8.1-2 ACI 318-71 "Building Code Requirements for Reinforced Concrete."

3.8.1-3 ACI 349-75 "Code Requirements for Nuclear Safety Related Concrete Structures" Appendix C "Special Provisions for Impulsive and Impactive Effects."

3.8.1-4 NRC Regulatory Guide 1.10 "Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures."

3.8.1-5 NRC Regulatory Guide 1.13 "Spent Fuel Storage Facility Design Basis." 2

3.8.1-6 NRC Regulatory Guide 1.15 "Testing of Reinforcing Bars for Category I Concrete Structures."

3.8.1-7 NRC Regulatory Guide 1.18 "Structural Acceptance Test for Concrete Primary Reactor Containments."

3.8.1-8 NRC Regulatory Guide 1.19 "Nondestructive Examination of Primary Containment Liner Welds."

3.8.1-9 NRC Regulatory Guide 1.54 "Quality Assurance Requirements for Coatings Applied to Water-Cooled Nuclear Power Plants."

3.8.1-10 NRC Regulatory Guide 1.55 "Concrete Placement in Category I Structures."

3.8.1-11 NRC Regulatory Guide 1.57 "Design-Limits and Loading Combinations for Metal Primary Reactor Containment System Components."

3.8.1-12 NRC Regulatory Guide 1.60 "Design Response Spectra for Seismic Design of Nuclear Power Plants," Rev. 1, Dec. 1973.

3.8.1-13 NRC Regulatory Guide 1.61 "Damping Values for Seismic Design of Nuclear Power Plants," Oct. 1973.

3.8.1-14 NRC Regulatory Guide 1.92 "Combining Model Responses and Spatial Components in Seismic Response Analysis" Rev 1 Feb. 1976.

3.8.1-15 NRC Regulatory Guide 1.63 "Electric Penetration Assemblies in Containment Structures for Water-Cooled Nuclear Power Plants."

3.8.1-16 NRC Regulatory Guide 1.76 "Design Basis Tornado for Nuclear Power Plants."

3.8.1-17 NRC Regulatory Guide 1.70 "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants.

2

3.8.1-18 NRC Regulatory Guide 1.94 "Quality Assurance Requirements for Installation, Inspection, and Testing of Structural Concrete and Structural Steel during the Construction Phase of Nuclear Power Plants.

3.8.1-19 NRC Regulatory Guide 1.122 "Development of Floor Design Response Spectra for Seismic Design of Floor Supported Equipment or Components."

3.8.1-20 NRC Branch Technical Position AAB-3-2 "Tornado Design Classification."

3.8.1-21 "Concrete Manual" - Bureau of Reclamation, 8th Edition, 1975, P. 45.

3.8.1-22 NRC Standard Review Plan Sec. 2.3.1 "Regional Climatology."

3.8.1-23 NRC Standard Review Plan Sec. 2.3.2 "Local Meteorology."

3.8.1-24 NRC Standard Review Plan Sec. 3.3.1 "Wind Loading."

3.8.1-25 NRC Standard Review Plan Sec. 3.3.2 "Tornado Loading."

3.8.1-26 NRC Standard Review Plan Sec. 3.5.1.4 "Missiles Generated by Natural Phenomena."

3.8.1-27 NRC Standard Review Plan Sec. 3.5.2. "Structures, Systems and Compononents to be Protected from Externally Generated Missiles."

3.8.1-28 NRC Standard Review Plan Sec. 3.5.3 "Barrier Design Procedures."

3.8.1-29 NRC Standard Review Plan Sec. 3.7.1 "Seismic Input."

3.8.1-30 NRC Standard Review Plan Sec. 3.7.2 "Seismic System Analysis."

3.8.1-31 NRC Standard Review Plan Sec. 3.7.3 "Seismic Subsystem Analysis."

3.8.1-32 NRC Standard Review Plan Sec. 3.7.4 "Seismic Instrumentation."

3.8.1-33 NRC Standard Review Plan Sec. 3.8.1 "Concrete Containment" (11/14/75).

3.8.1-34 NRC Standard Review Plan Sec. 3.8.2 "Steel Containment."

3.8.1-35 NRC Standard Review Plan Sec. 3.8.3 "Concrete and Steel Internal Structures of Steel or Concrete Containments (11/24/75).

3.8.1-36 NRC Standard Review Plan Sec. 3.8.4 "Other Seismic Category I Structures."

3.8.1-37 NRC Standard Review Plan Sec. 3.8.5 "Foundations."

3.8.1-38 ASCE "Manual of Standard Practices for Design of Nuclear Power Plants."

3.8.1-39 ANSI "Building Code Requirements for Minimum Design Loads in Buildings and Other Structures A58.1-1972".

3.8.1-40 "Wind Forces on Structures", Task Committee on Wind Forces, Transactions, ASCE, Vol. 126, Part 2, Paper No. 3269, 1961.

3.8.1-41 Maher, F. J., "Wind Loads on Dome-Cylinder and Dome-Cone Shapes," Journal of the Structural Division, ASCE, Vol. 92, No. ST5. Proc. Paper 4933, October 1966.

3.8.1-42 Timoshenko S. and Woinowsky-Krieger S., "Theory of Plates and Shells," McGraw Hill 1959.

3.8.1-43 Fugge W., "Stresses in Shells," Springer-Verlag 1960.

3.8.1-44 Billington D. P., "Thin Shell Concrete Structures," McGraw Hill 1965.

3.8.1-45 Hetenyi M., "Beam on Elastic Foundation," University of Michigan Press 1964.

3.8.1-46 Arshain Amiridian, "Design of Protective Structures," Bureau of Yards and Docks, Department of the Navy, Washington, D. C., August 1950.

3.8.1-47 Recht R. F. and Ipson T. W., "Ballistic Perforation Dynamics" Journal of Applied Mechanics, Transactions of ASME, Vol. 30, Series E, No. 3, September, 1963.

3.8.1-48 Biggs J. M.,"Introduction to Structural Dynamics," McGraw Hill 1964.

3.8.1-49 Norris C. H. et. al., "Structural Design for Dynamic Loads" McGraw Hill 1969.

3.8.1-50 Williamson R. A. and Alvy R. R., "Impact Effect of Fragments Striking Structural Elements" Holmes and Narver, Inc. 1973.

3.8.1-51 Suarez M. A.,"Impactive Dynamic Analysis" ASCE National Structural Engineering Meeting, Baltimore 1971.

3.8.1-52 Newmark N. M., "An Engineering Approach to Blast-Resistant Design" ASCE Transactions Paper No. 2786.

3.8.1-53 Blume T. A., Newmark, N. M., and Corning L. H., "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions" PCA 1961.

3.8.1-54 Kennedy R. P., "A Review of Procedures for the Analysis and Design of Concrete Structures to Resist Missile Impact Effects" Holmes and Narver, Inc. Anaheim, California.

3.8.1-55 Winter G., "Design of Concrete Structures" McGraw Hill 1964.

3.8.1-56 Wang C. K. and Salmon C. G., "Reinforced Concrete Design" Intext Educational Publishers N.Y.

3.8.1-57 Dunham C. W., "Advanced Reinforced Concrete" McGraw Hill 1964.

3.8.1-58 Peterka J. A. and Cormak J. E., "Adverse Wind Loading Induced by Adjacent Buildings" ASCE J. Structural Div. March 1976.

3.8.1-59 Doan P. L., "Tornado Considerations for Nuclear Power Plants Structures" Nuclear Safety Vol. 11, No. 4, 1970.

3.8.1-60 Dunlop F. A., and Wiedner K., "Nuclear Power Plant Tornado Design Considerations" ASCE J. Power Div. March 1971.

3.8.1-61 Burket F. E., "Effects of Tornado on Buildings" Civ. Eng. Jan. 1963.

3.8.1-62 ACI Publication SP 17 (1973) "Design Handbook in Accordance with the Strength Design Method of ACI 318-71."

3.8.1-63 Jacobsen, L. S. and Ayre R. S., "Engineering Vibrations" McGraw Hill 1958.

3.8.1-64 Harris C. M., Crede C. E. "Shock and Vibration Handbook" McGraw Hill 1961.

3.8.1-65 Langharr H. L., "Energy Methods in Applied Mechanics" John Wiley & Sons 1962.

3.8.1-66 TRW Nelson Division "Embedment Properties of Headed Studs".

3.8.1-67 TRW Nelson Division, "Construction Applications - Bent Concrete Anchors".

3.8.1-68 PCI "Manual on Design of Connections for Precast Prestressed Concrete".

3.8.1-69 AISC "Manual of Steel Construction" 7th Ed. 1970.

3.8.1-70 Fritz Engineering Laboratory-Lehigh Univ.," Load-Deformation Graphs".

3.8.1-71 Timoshenko-Goodier, "Theory of Elasticity" McGraw Hill 1951.

3.8.1-72 ASME Boiler & Pressure Vessel Code Sect. III Div. 1, Subsection NA 1975.

3.8.1-73 ASME Boiler & Pressure Vessel Code Sect. III Div. 1, Subsection NE 1975 Class MC components.

3.8.1-74 Savin B. N., "Stress Distribution Around Holes," NASA 1970.

3.8.1-75 Peterson, R. F., "Stress Concentration Design Factors," John Willey 1966.

3.8.1-76 Brush, Almroth, "Buckling of Bars, Plates, and Shells" McGraw Hill.

3.8.1-77 Doyle, Chu, "Some Structural Considerations in the Design of Nuclear Containment Liners" Nuclear Engineering and Design 1971.

3.8.1-78 Winstead, Burdett, and Armentront, "Linear Anchorage Analysis for Nuclear Containments" ASCE J. Structural Div., Oct. 1975.

3.8.1-79 Bofaut, Carreira, and Walser, "Creep and Shrinkage in Reactor Containment Shells" ASCE J. Structural Div., Oct. 1975.

3.8.1-80 Hardington, Parker, and Spruce, "Liner Design and Development for the Oldbury Vessels" Paper 56, Conference on Prestressed Concrete Pressure Vessels, London, 1967.

3.8.1-81 Young, Tate, "Design of Liner for Reactor Vessels" Paper J57, Conference on Prestressed Concrete Pressure Vessels, London, 1967.

3.8.1-82 Chapman, Carter, "Interaction Between a Pressure Vessel and Its Liner" Paper J58, Conference on Prestressed Concrete Pressure Vessels, London, 1967.

3.8.1-83 Bishop, Horseman, and White, "Linear Design and Construction" Paper J59, Conference on Prestressed Concrete Pressure Vessels, London, 1967.

3.8.1-84 Parker, "Stress Analysis of Liners for Prestressed Concrete Pressure Vessels" Proceedings of the 1st International Conference on Structural Mechanics in Reactor Technology, Paper H6/1, 1971.

3.8.1-85 Yang, "A Matrix Displacement Method on Pre and Post-Buckling Analysis of Liners for Reactor Vessels" Proceedings of the 1st International Conference on Structural Mechanics in Reactor Technology, Paper H6/2, 1971.

3.8.1-86 Salvatori, R., "Failure Angle of Disc" Westinghouse 1971.

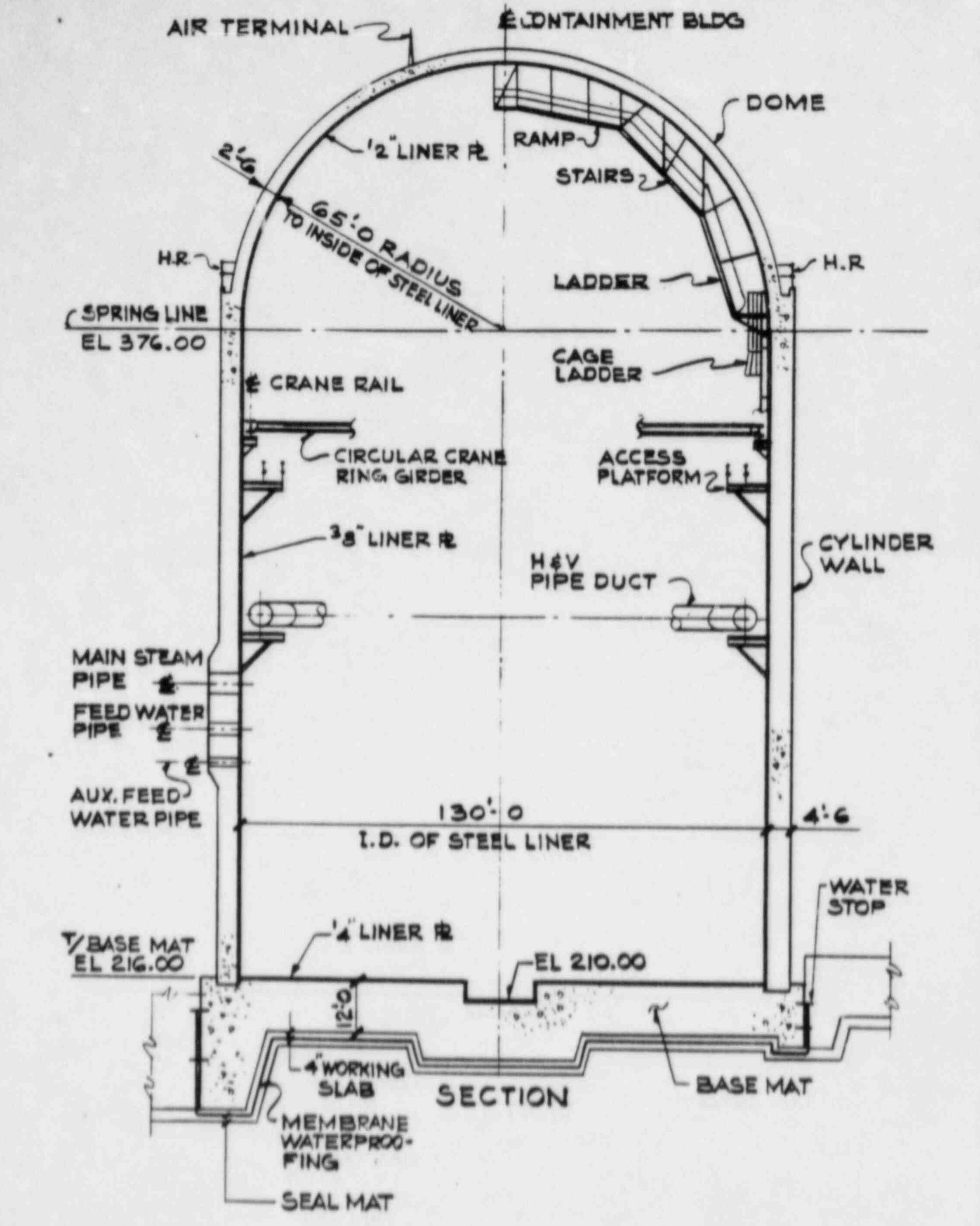
3.8.1-87 Bush, S., "Probability of Damage to Nuclear Components Due to Turbine Failure" Nuclear Safety 14, November 1973.

3.8.1-88 "Full-Scale Tornado Missile Impact Tests," EPRI NP-148, April 1976.

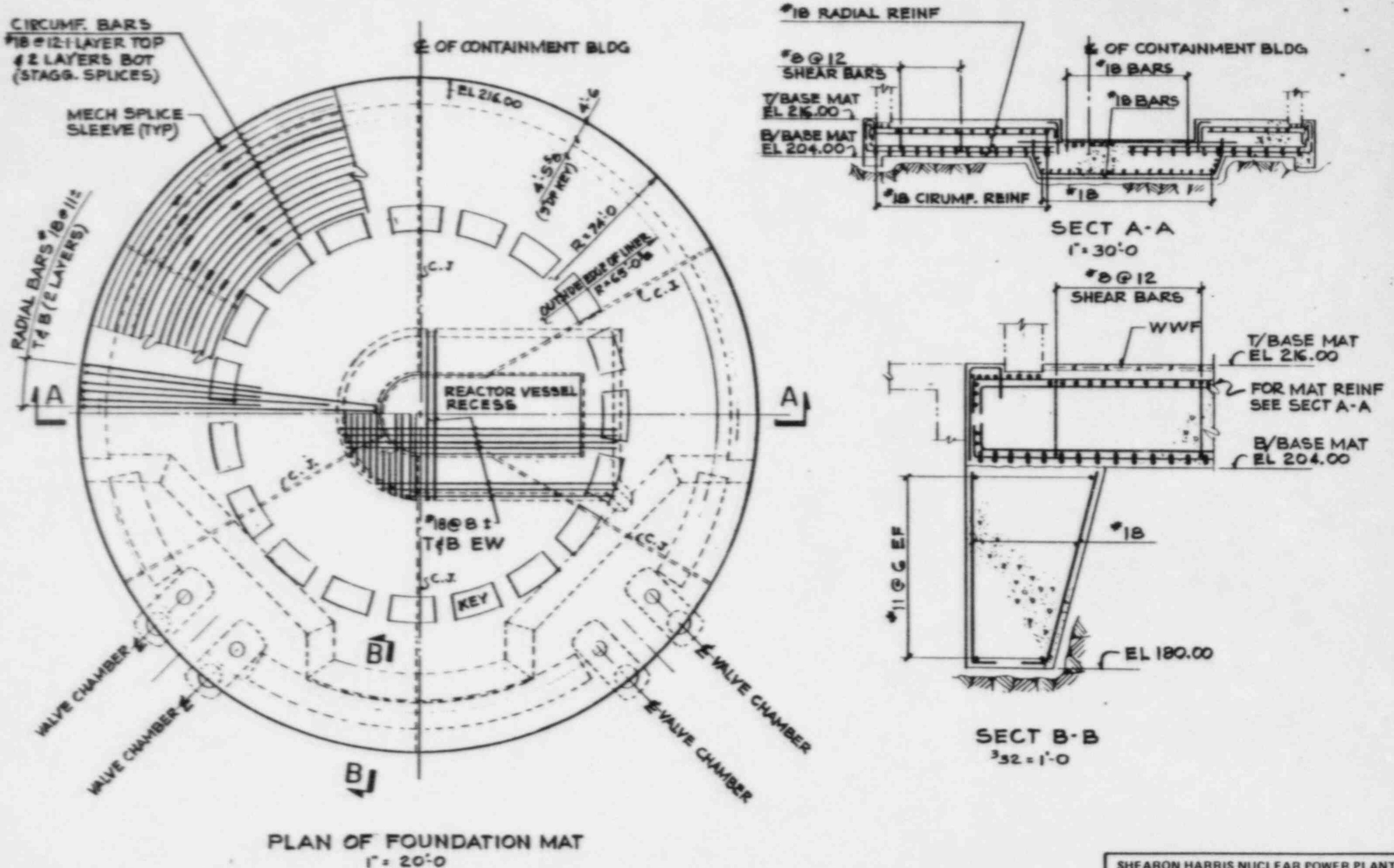
3.8.1-89 Russel, C. R., "Reactor Safeguards," Pergammon Press, 1962.

3.8.4-1 Winterhorn, H. F. and Fang, H., "Foundation and Engineering Handbook," Van Nostrand Reinhold, 1975. 5

3.8.5-1 TVA, Technical Report No. 13, "The Kentucky Project," Appendix D, Design of Kentucky Structures Against Earthquakes.



SHEARON HARRIS NUCLEAR POWER PLANT Carolina Power & Light Company FINAL SAFETY ANALYSIS REPORT	CONCRETE CONTAINMENT STRUCTURE – GENERAL ARRANGEMENT	FIGURE 3.8.1-1

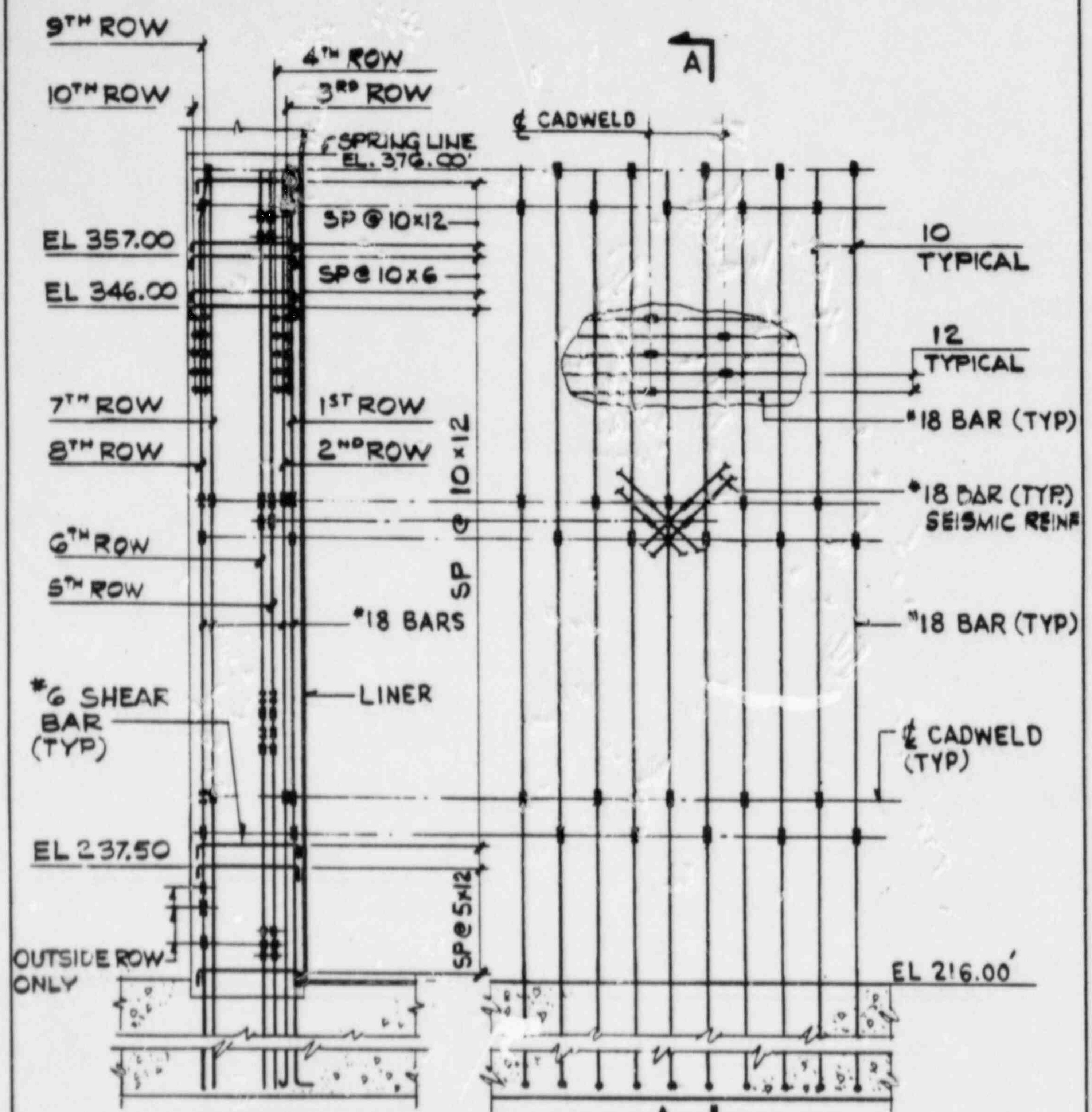


SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE – MAT, MASONRY & REINFORCING

FIGURE 3.8.1-2

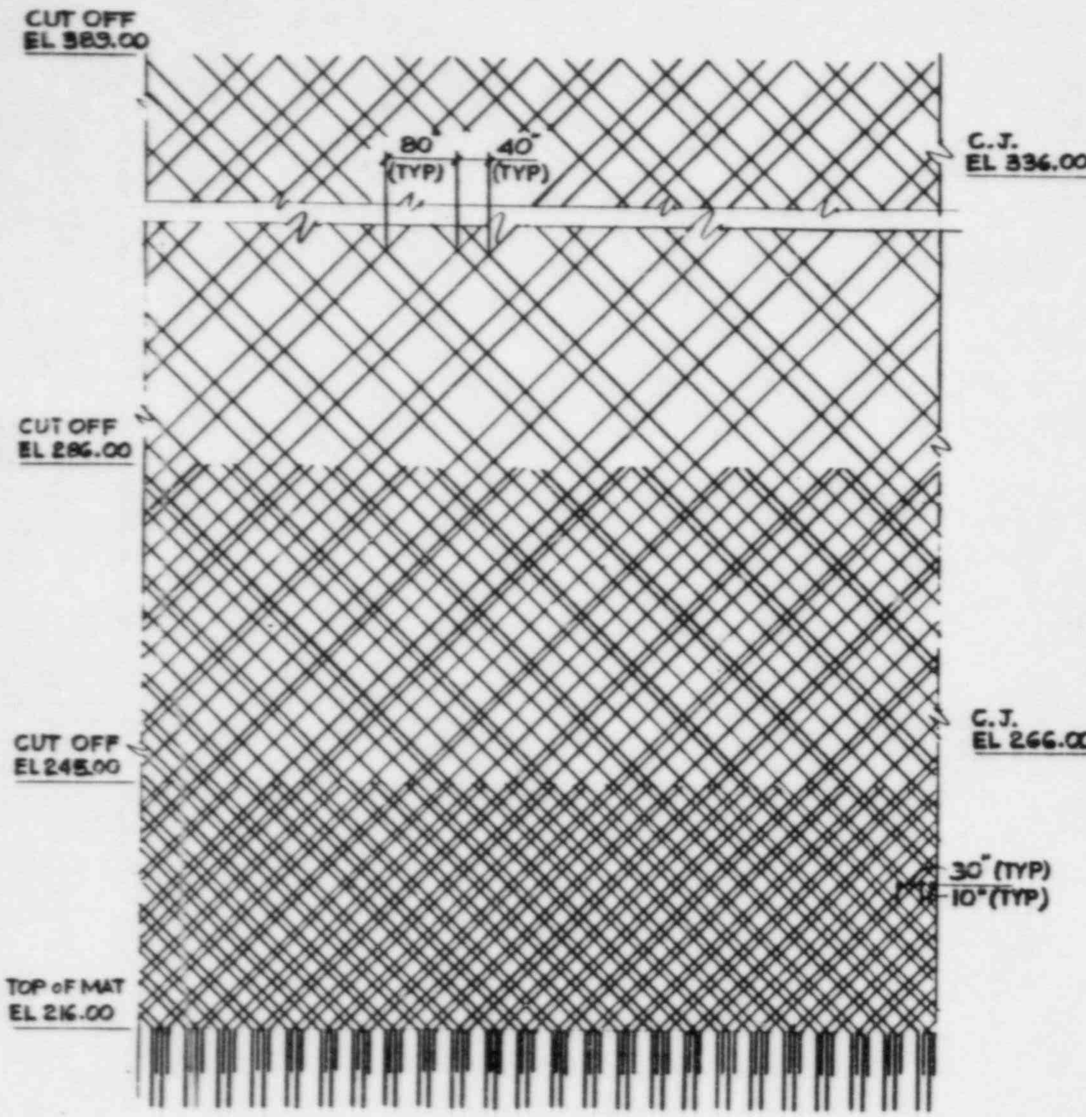
NOTE: CADWELD STAGGER SPACING VARIES.



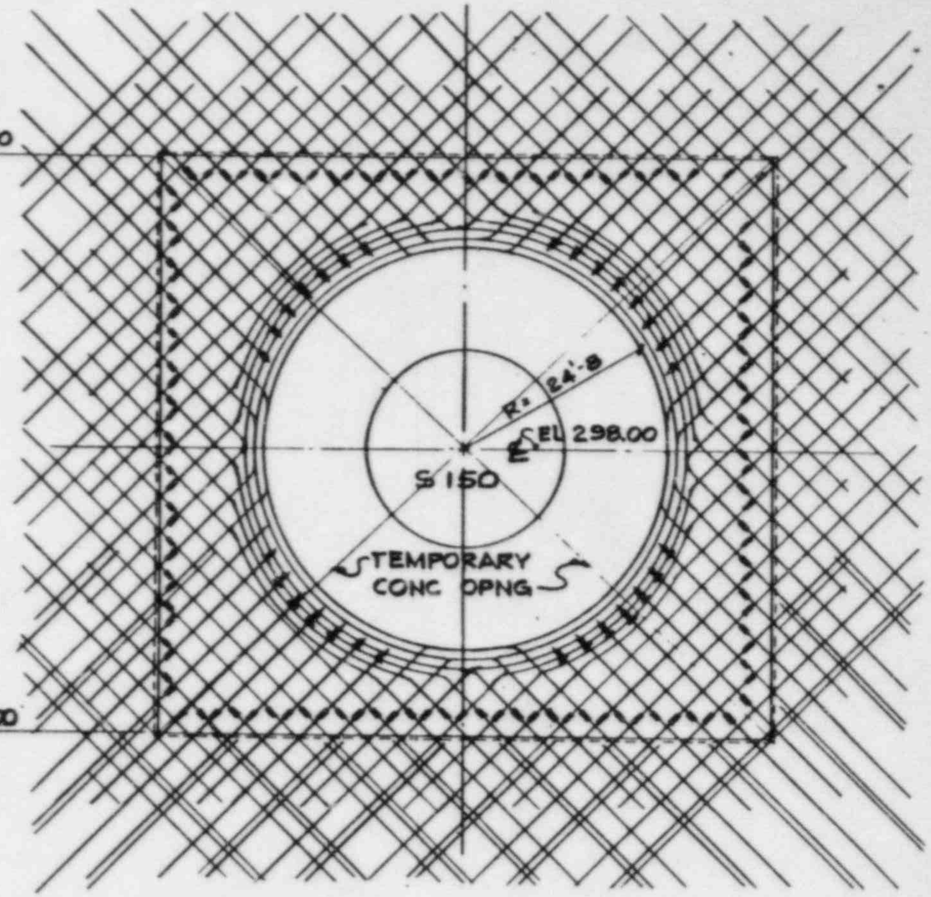
SECT A-A

CYL WALL DEVELOPED ELEV
NTS

SHEARON HARRIS NUCLEAR POWER PLANT Carolina Power & Light Company FINAL SAFETY ANALYSIS REPORT	CONCRETE CONTAINMENT STRUCTURE CYLINDER WALL REINFORCEMENT	FIGURE 3.8.1-3



DEVELOPED ELEVATION - TYPICAL AREA
(LOOKING FROM OUTSIDE)



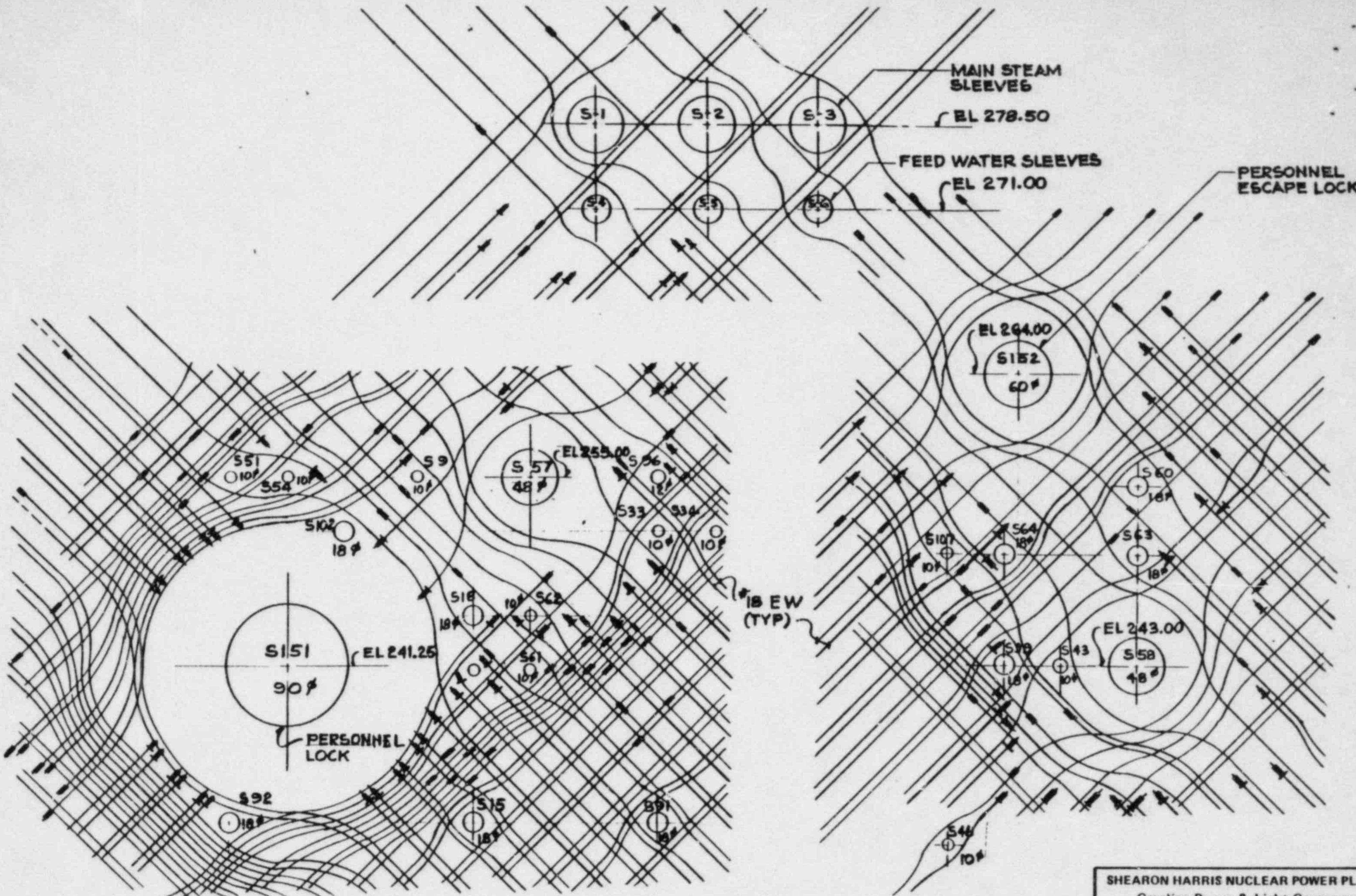
EQUIPMENT HATCH

NOTE: ALL REINFORCING BARS ARE #18

SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE – SEISMIC REINFORCEMENT

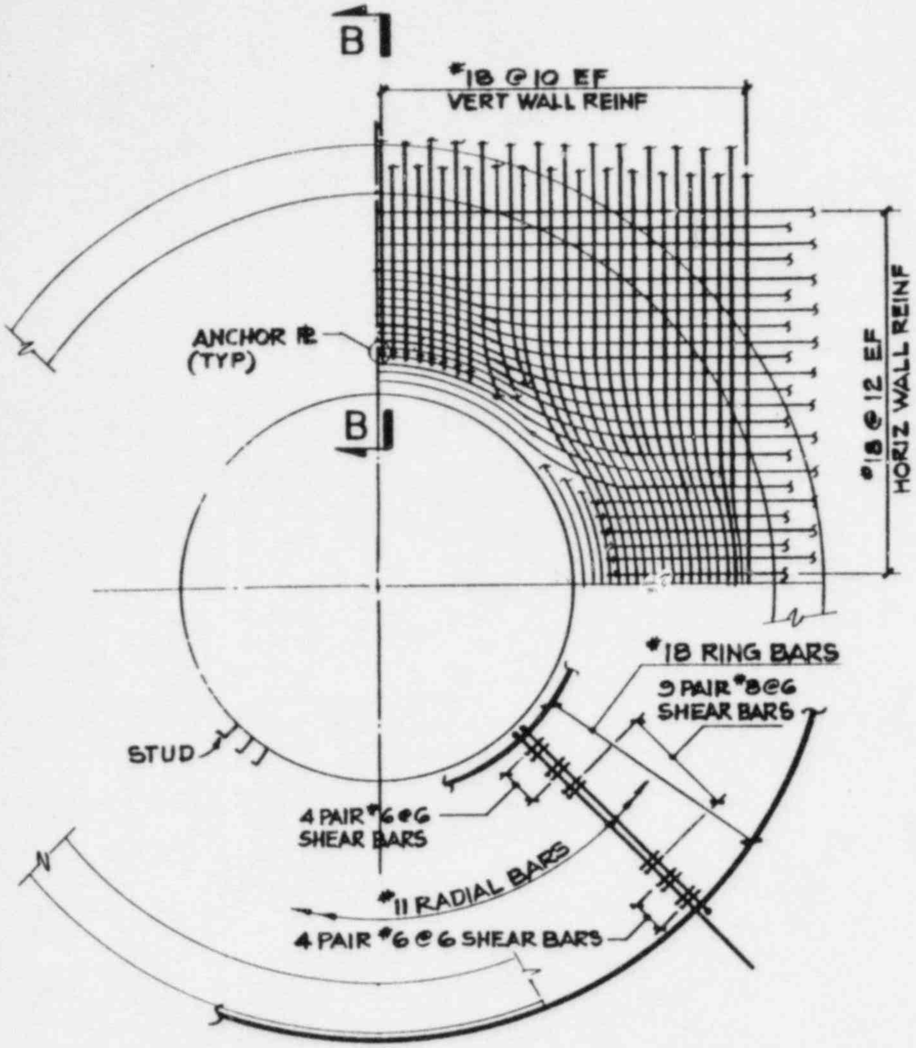
FIGURE 3.8.1-4



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

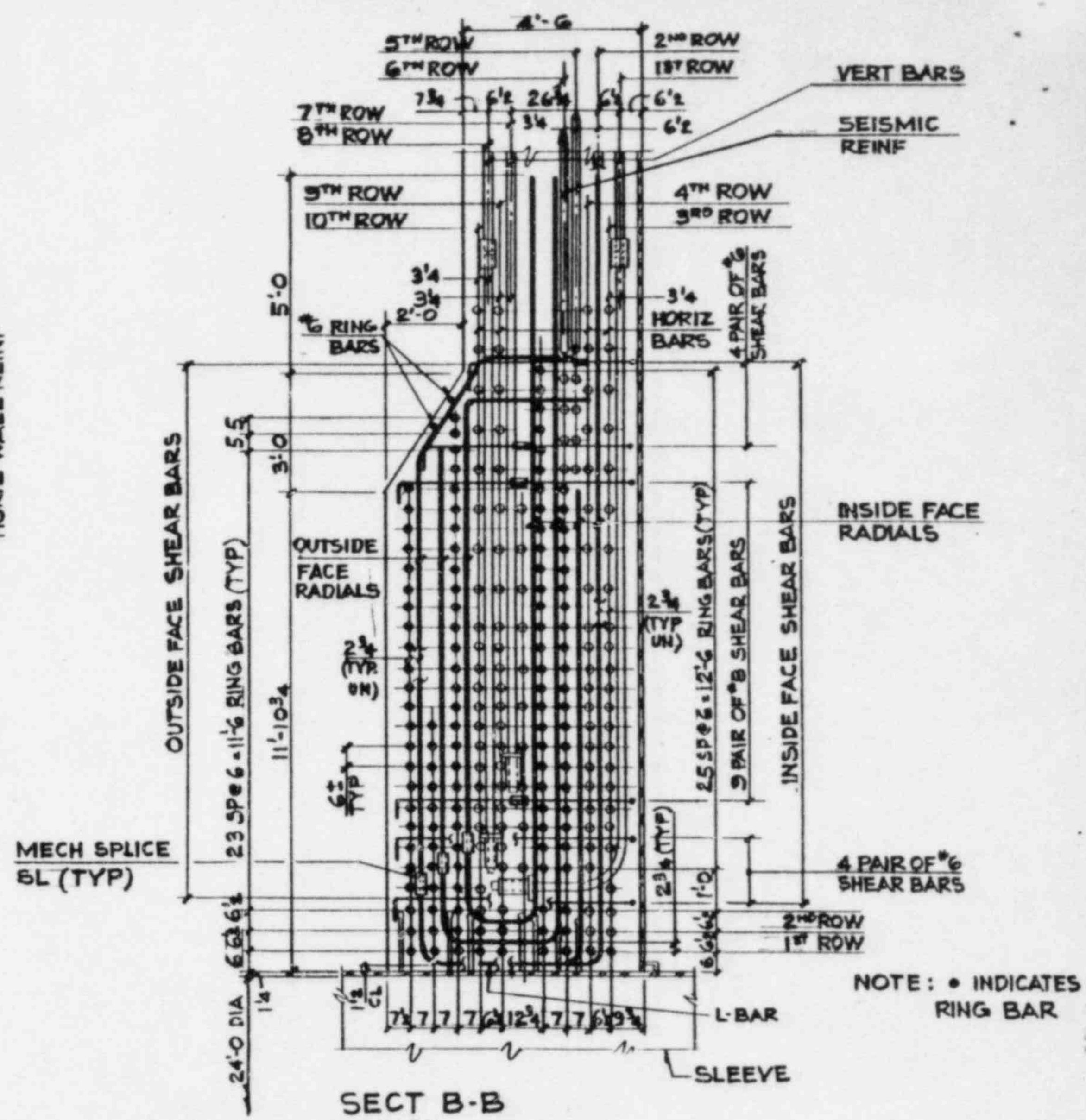
CONCRETE CONTAINMENT STRUCTURE – SEISMIC REINFORCEMENT

FIGURE 3.8.1-5



DEVELOPED ELEVATION

NTS



SECT B-B

NTS

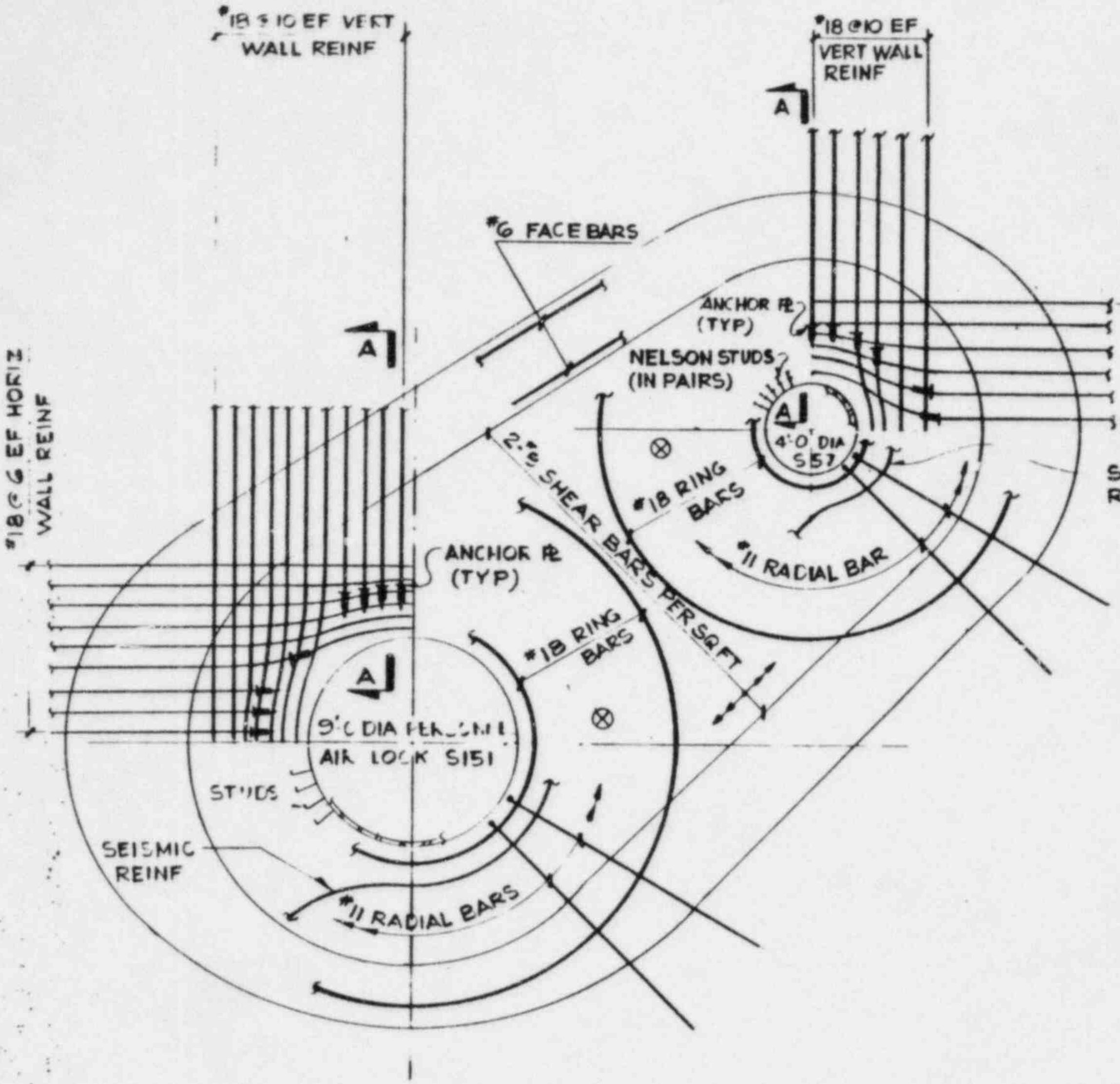
SHEARON HARRIS NUCLEAR POWER PLANT

Carolina Power & Light Company

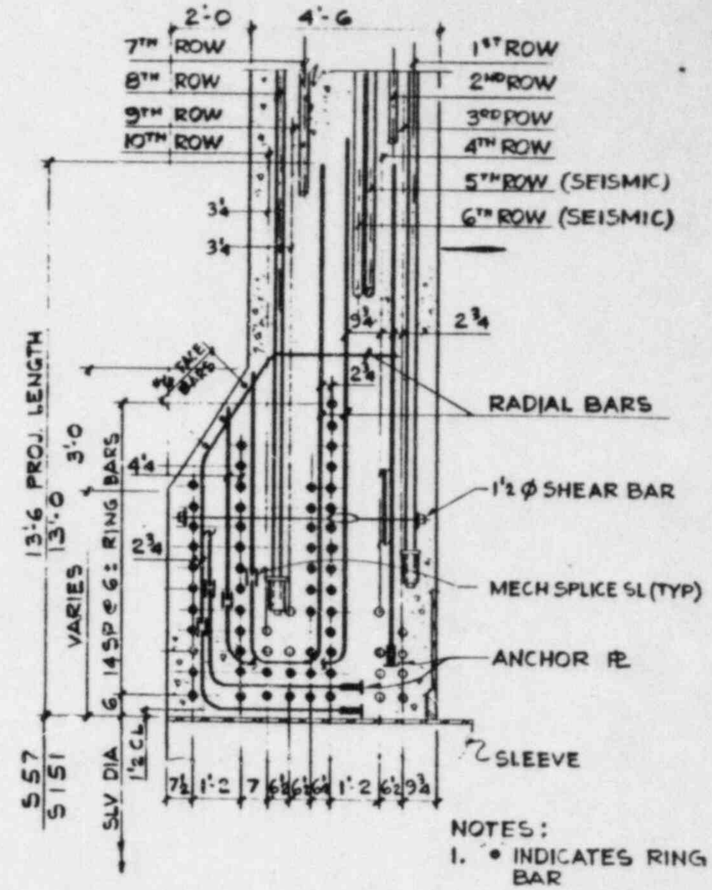
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE – EQUIPMENT HATCH REINFORCING

FIGURE 3.8.1-6



DEVELOPED ELEVATION (OUTSIDE FACE)
NTS



SECT A-A
NTS

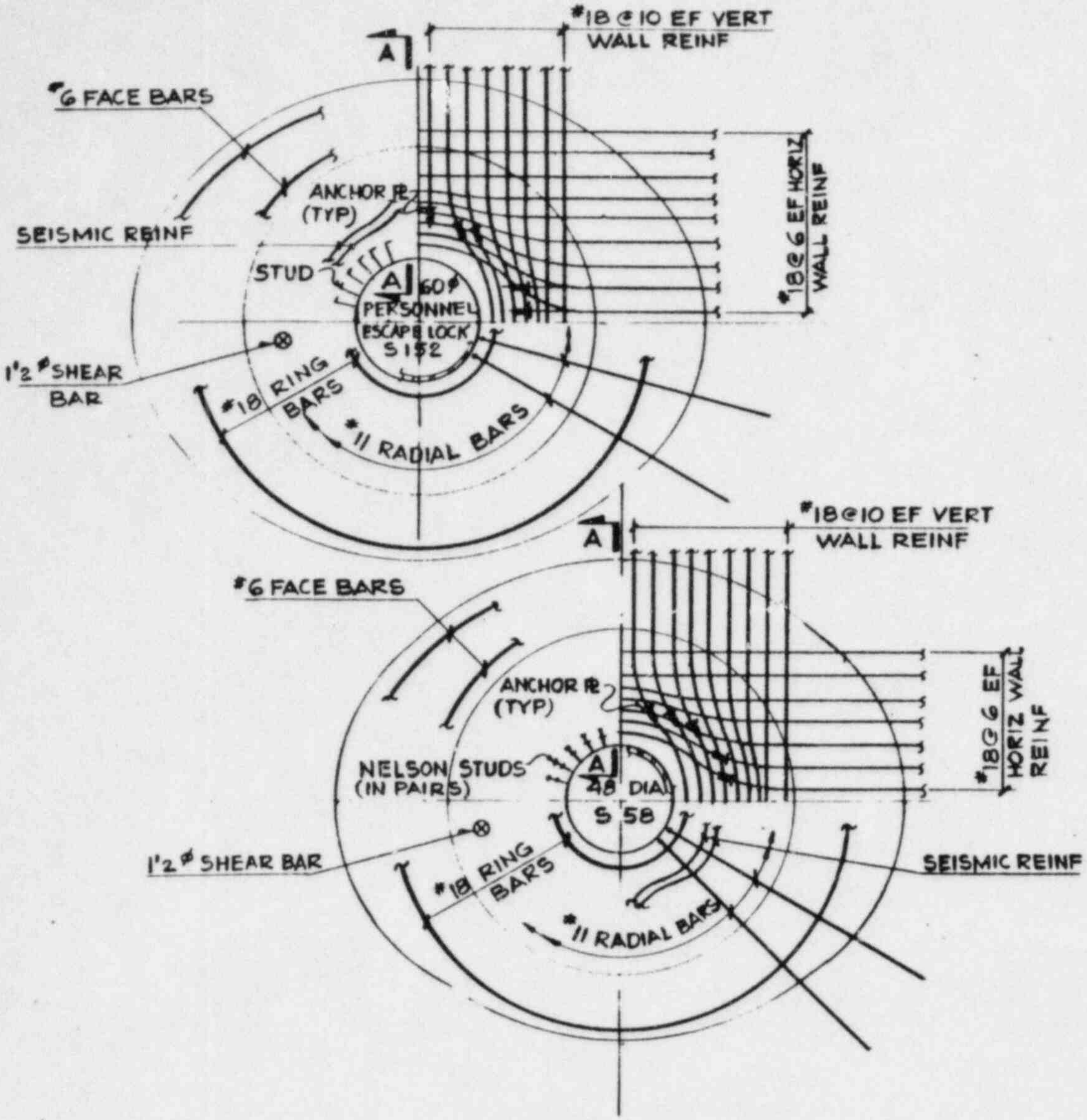
NOTES:

1. "•" INDICATES RING BAR
2. "o" INDICATES HORIZ BAR
3. STUDS OMITTED FOR CLARITY
4. ⊗ INDICATES 1'2 Ø SHEAR BAR

SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

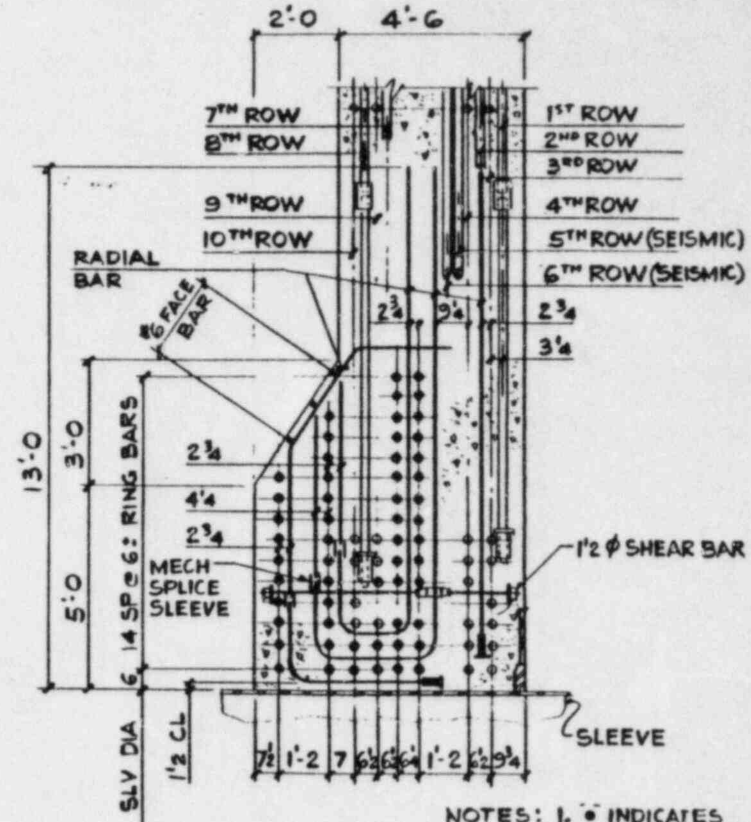
CONCRETE CONTAINMENT STRUCTURE
PERSONNEL AIR LOCK AND PEN. S57
REINFORCING

FIGURE 3.8.1-7



DEVELOPED ELEVATION (OUTSIDE FACE)

NTS



SECT A-A

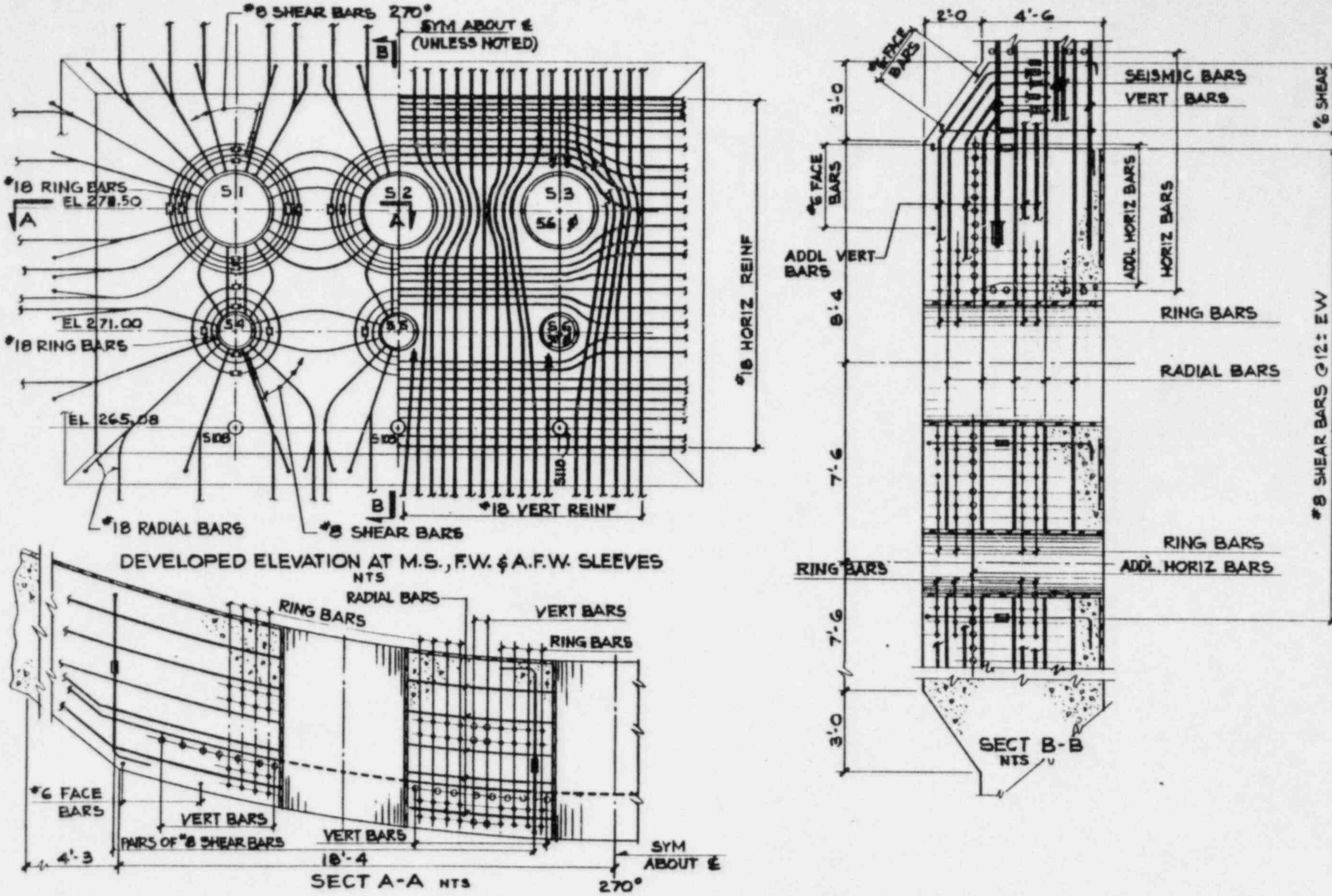
NTS

NOTES:
1. "•" INDICATES RING BARS
2. "o" INDICATES HORIZ BARS
3. STUDS OMITTED FOR CLARITY
4. ⊗ INDICATES 1'2 Ø SHEAR BAR

SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE
PERSONNEL ESCAPE LOCK AND PEN. S58
REINFORCING

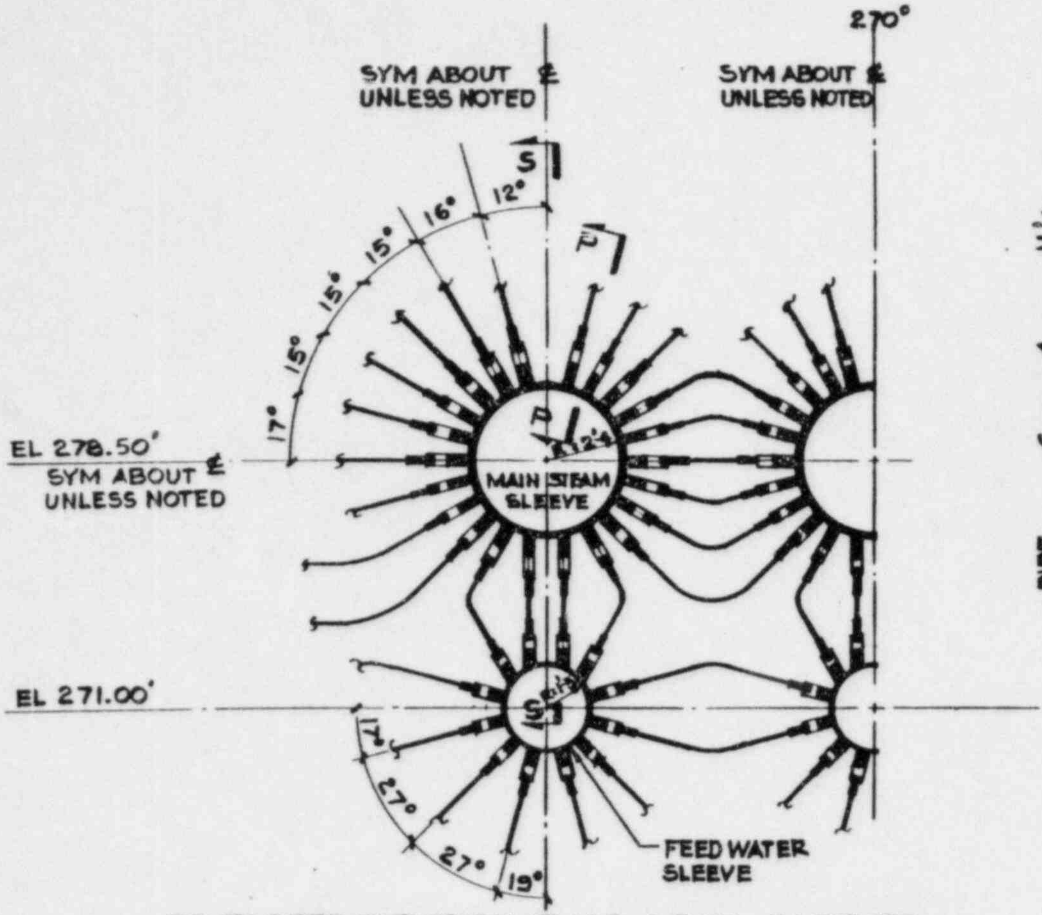
FIGURE 3.8.1-8



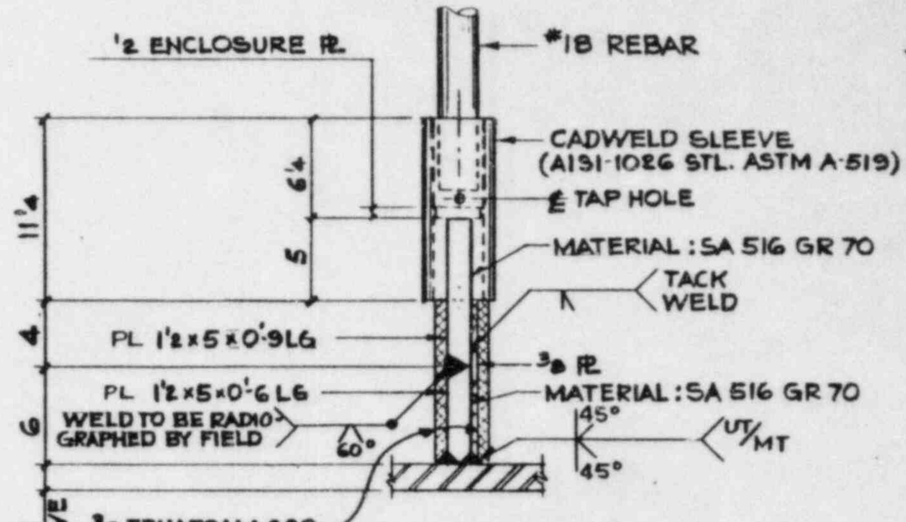
SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE – MS & FW PENETRATION REINFORCING

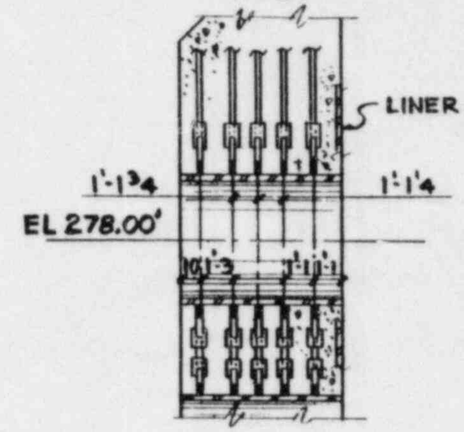
FIGURE 3.8.1-9



DEVELOPED ELEVATION AT M.S. & F.W. SLEEVES
NTS



SECT P-P
NTS

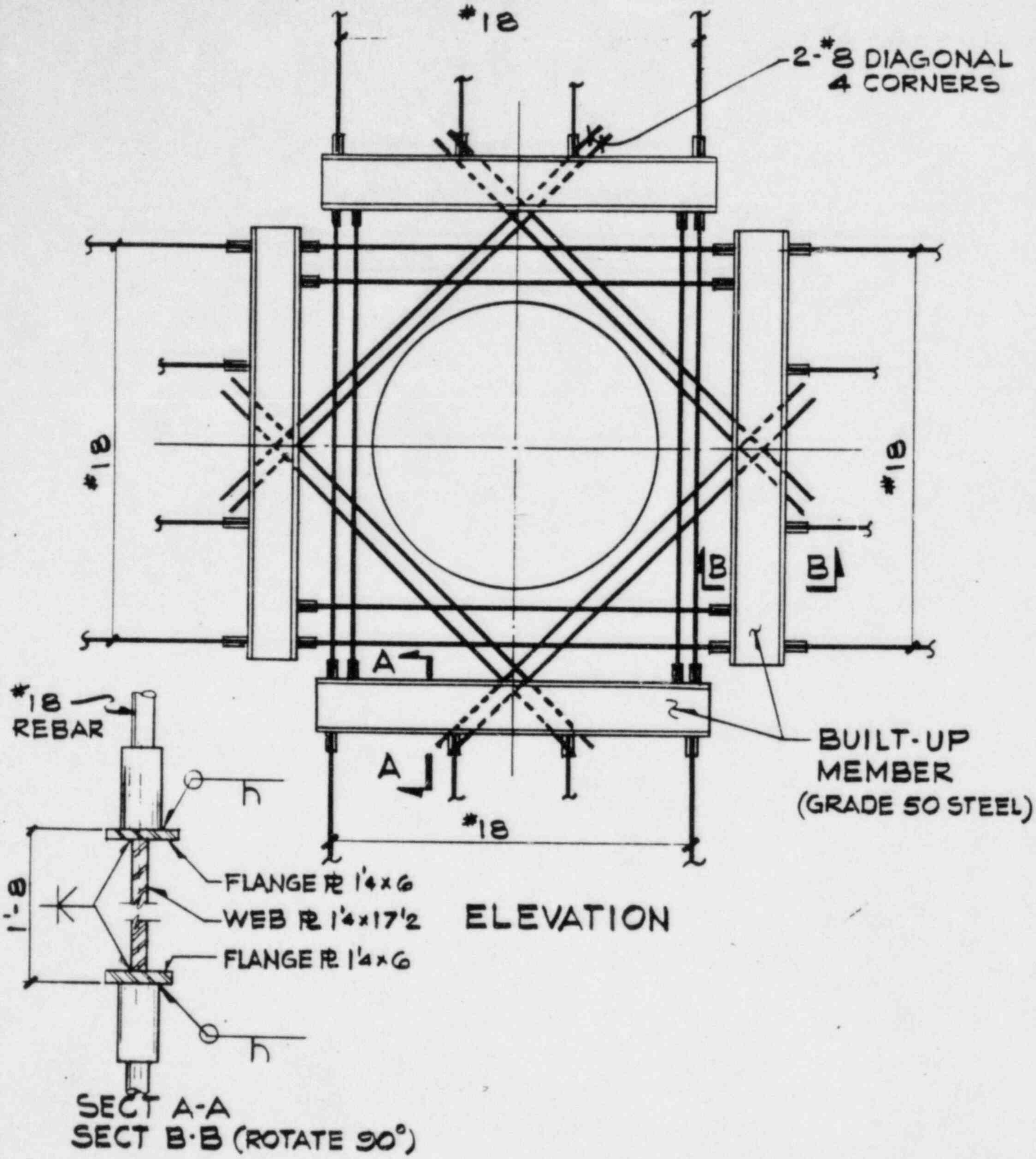


SECT S-S
NTS

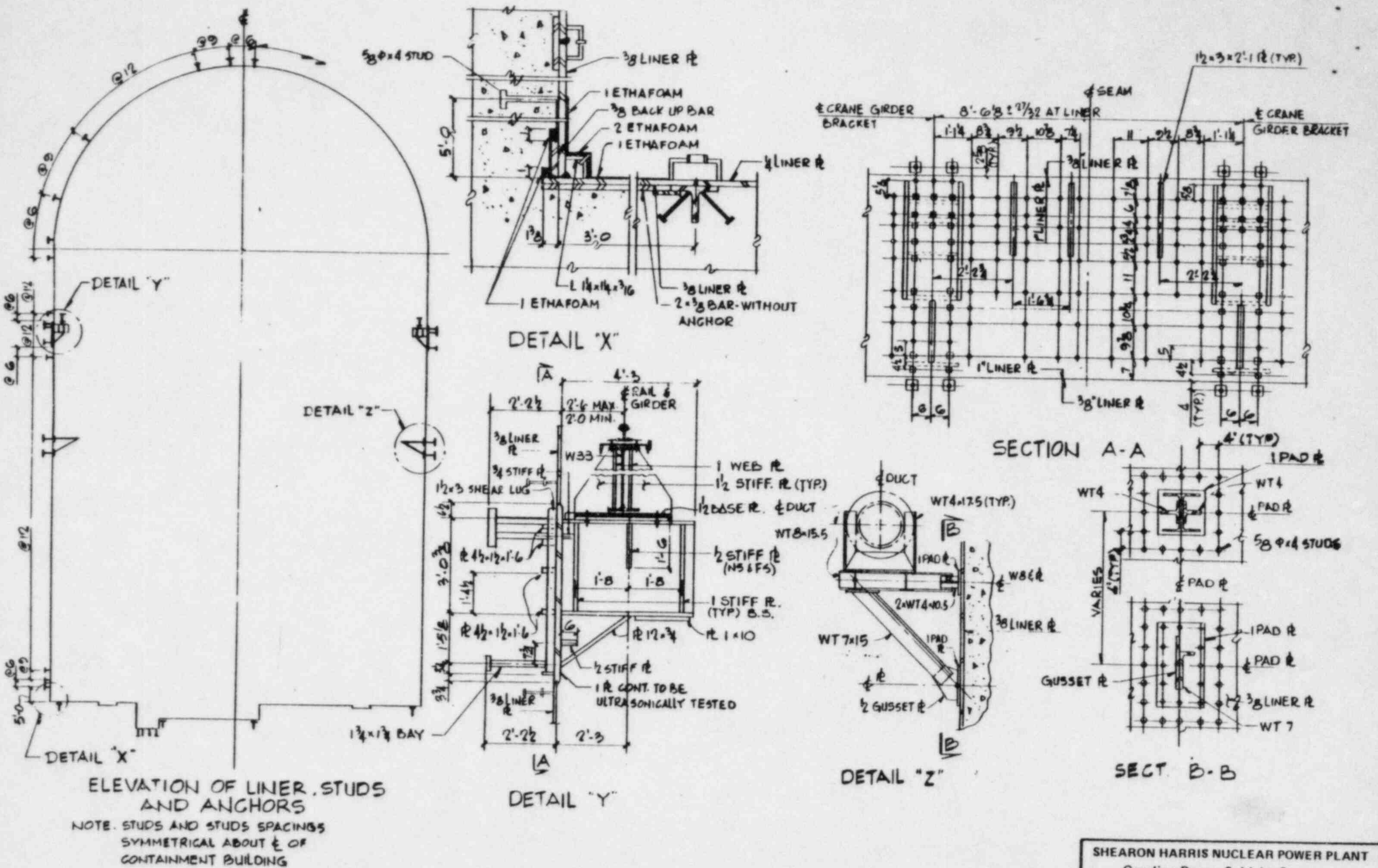
SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE – MS & FW PENETRATION ATTACHMENT

FIGURE 3.8.1-10



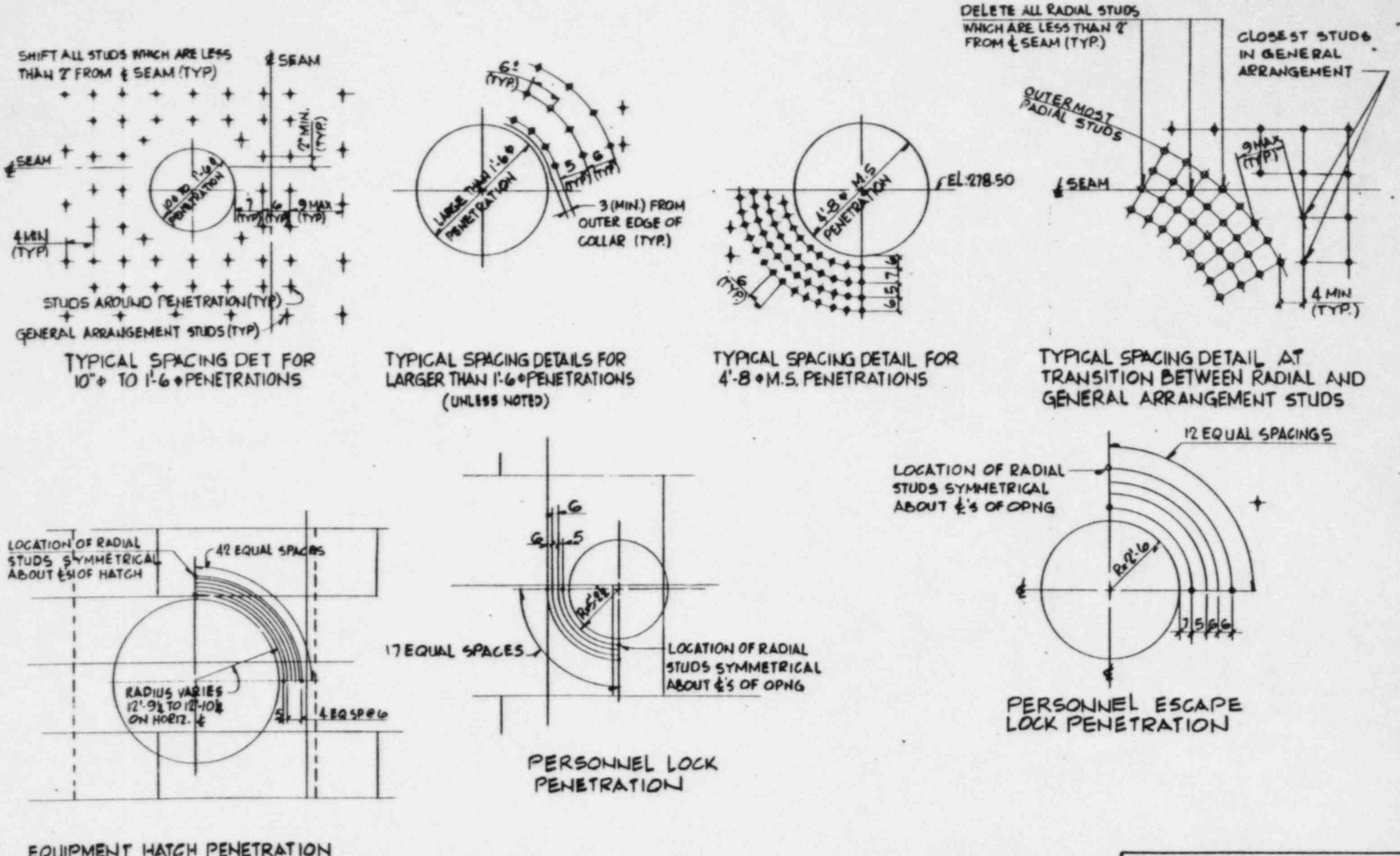
SHEARON HARRIS NUCLEAR POWER PLANT Carolina Power & Light Company FINAL SAFETY ANALYSIS REPORT	CONCRETE CONTAINMENT STRUCTURES – SMALL PENETRATION REINFORCING	FIGURE 3.8.1-11



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE – LINER DETAIL

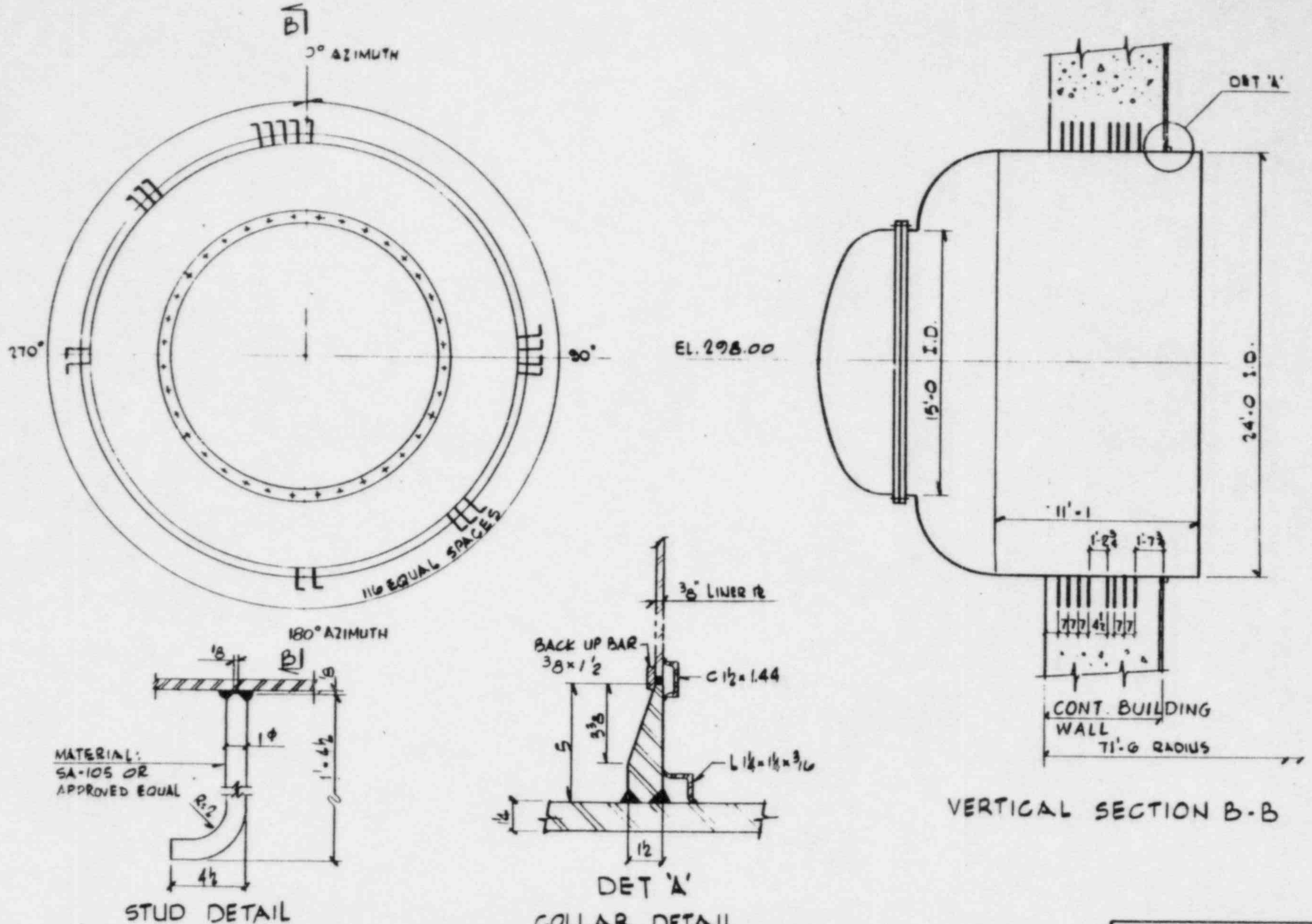
FIGURE 3.8.1-12



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE – LINER DETAILS

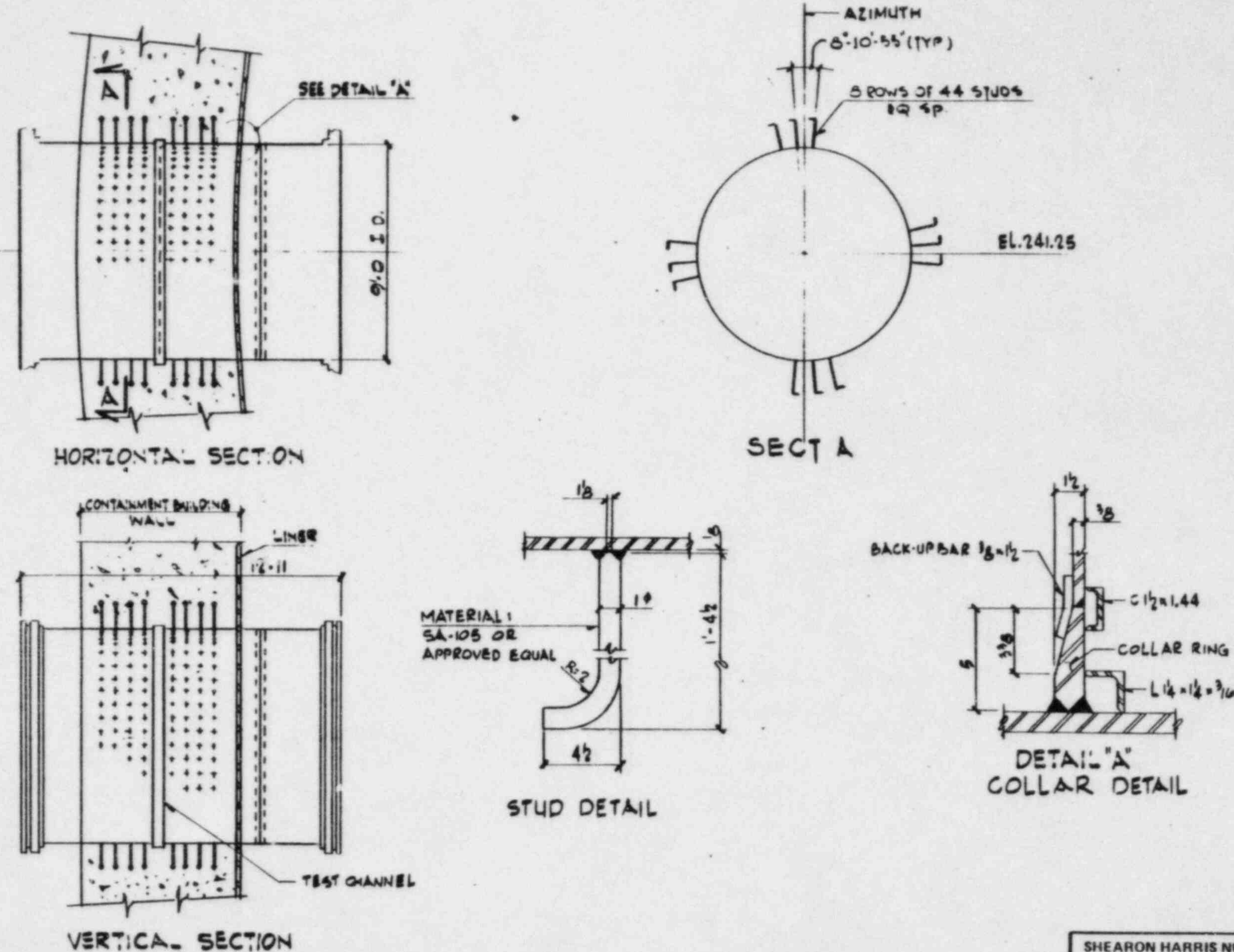
FIGURE 3.8.1-13



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE
EQUIPMENT HATCH PENETRATION

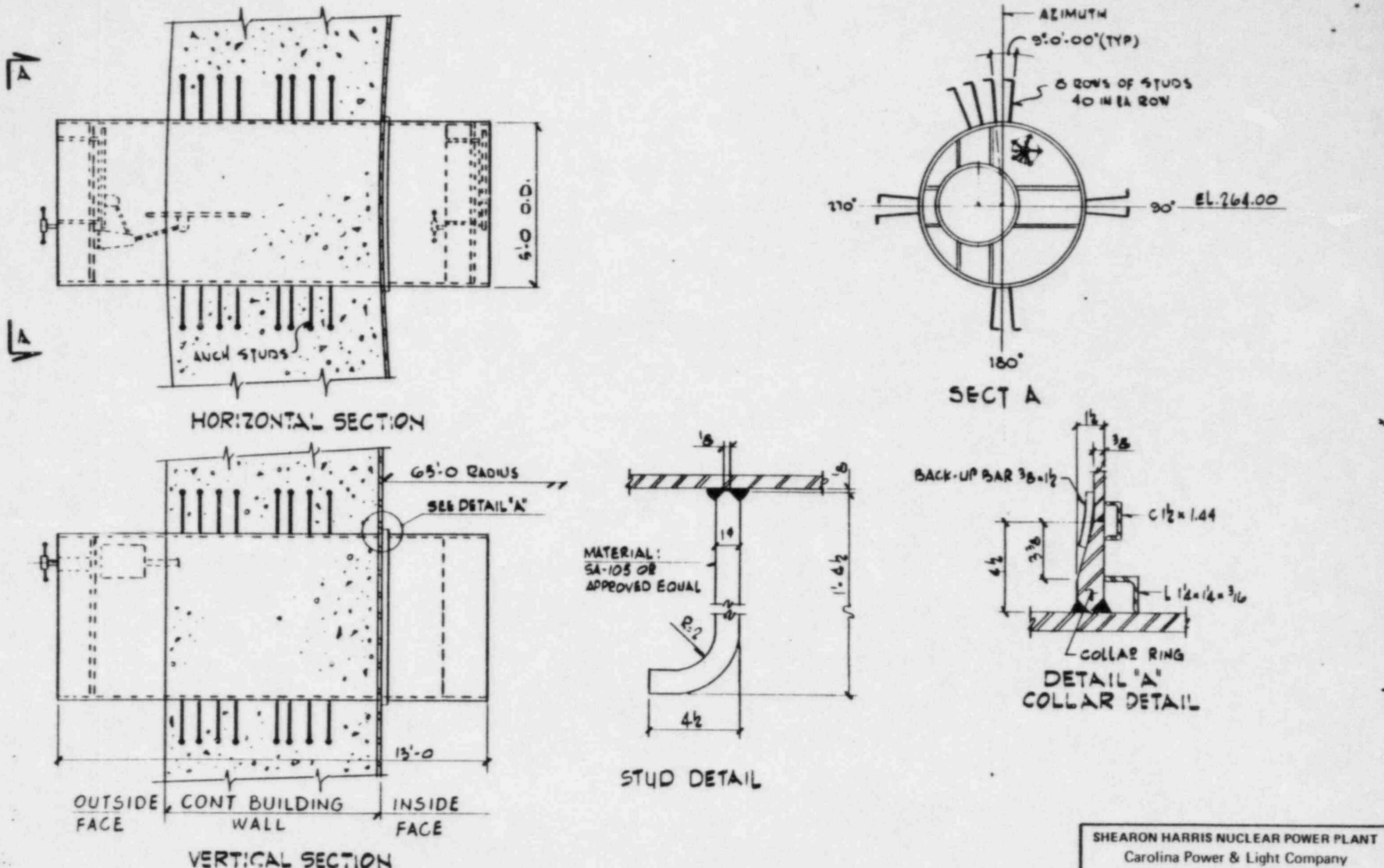
FIGURE 3.8.1-14



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE
PERSONNEL LOCK PENETRATION

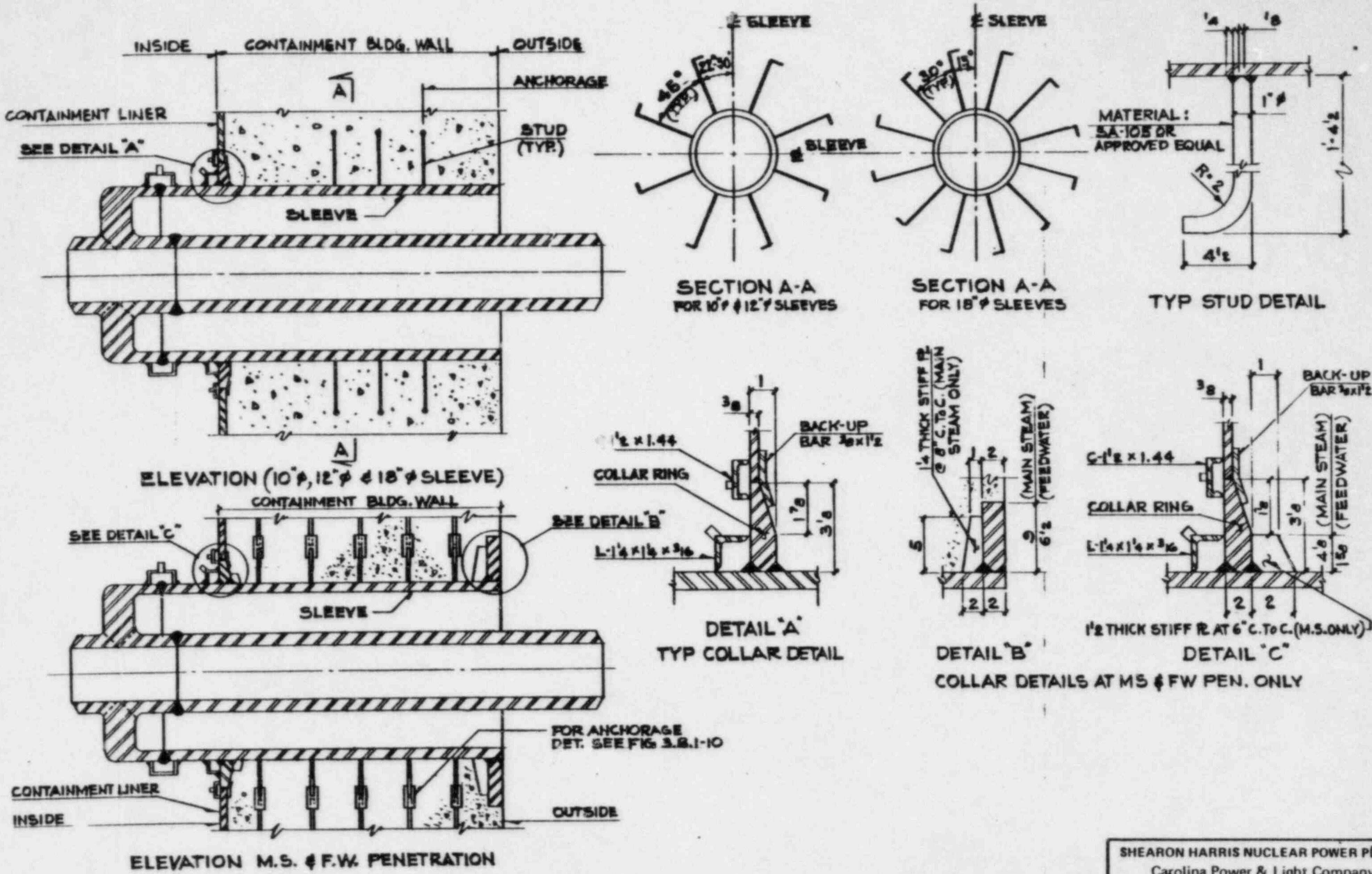
FIGURE 3.8.1-15



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

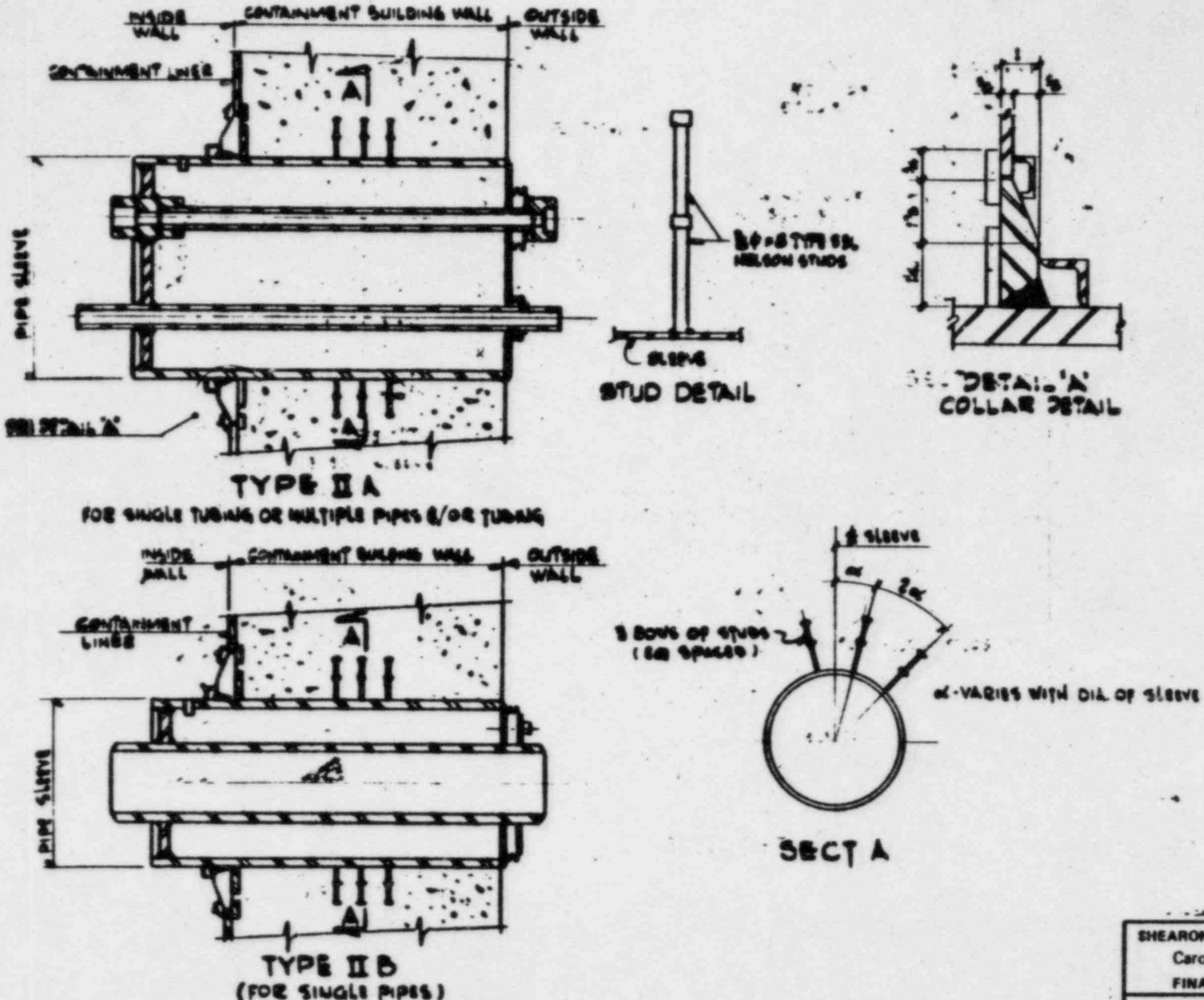
CONCRETE CONTAINMENT STRUCTURE
ESCAPE LOCK PENETRATION

FIGURE 3.8.1-16



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

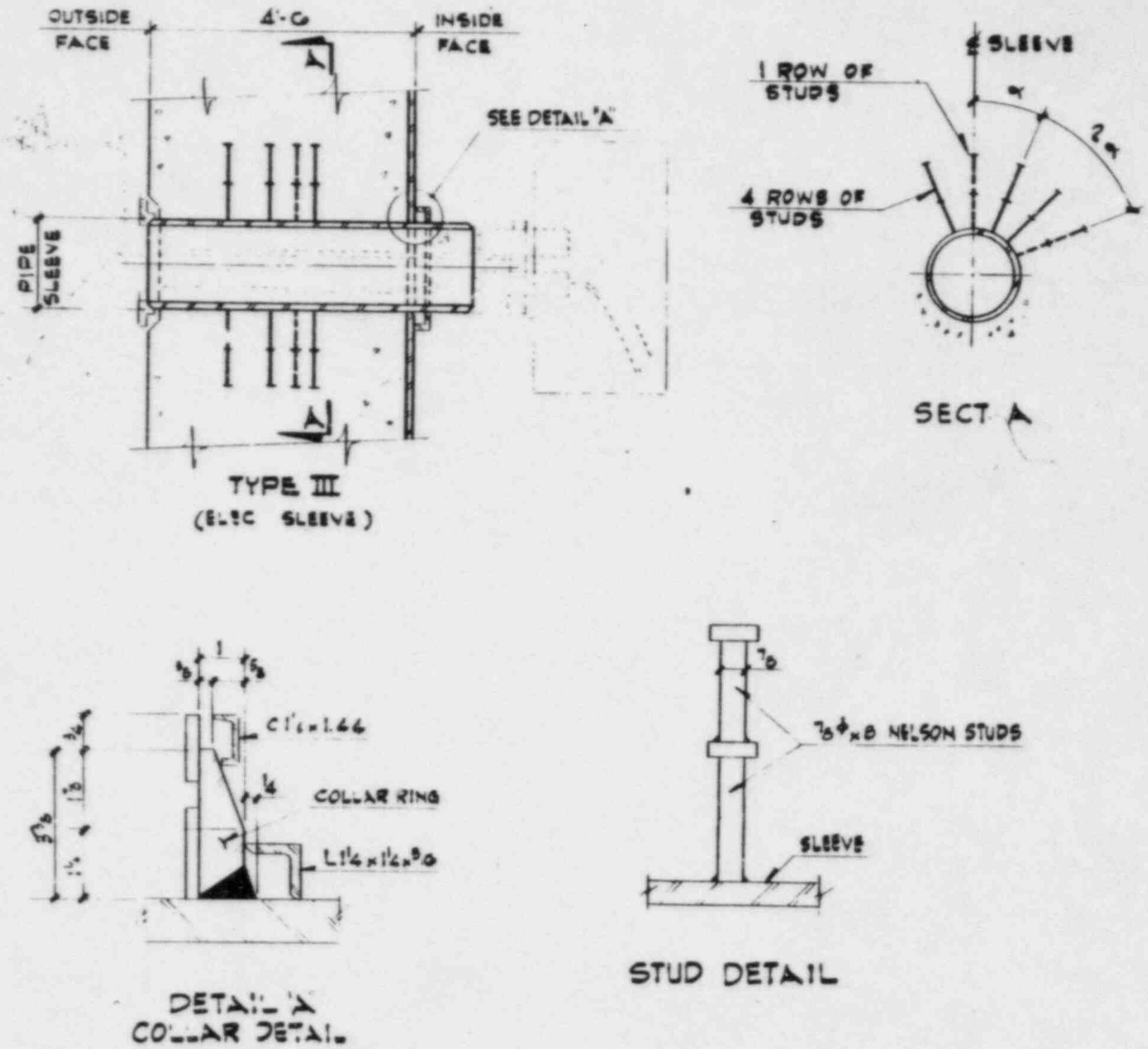
CONCRETE CONTAINMENT BUILDING
MECHANICAL TYPE I PENETRATION
FIGURE 3.8.1-17



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT BUILDING
MECHANICAL TYPE II PENETRATION

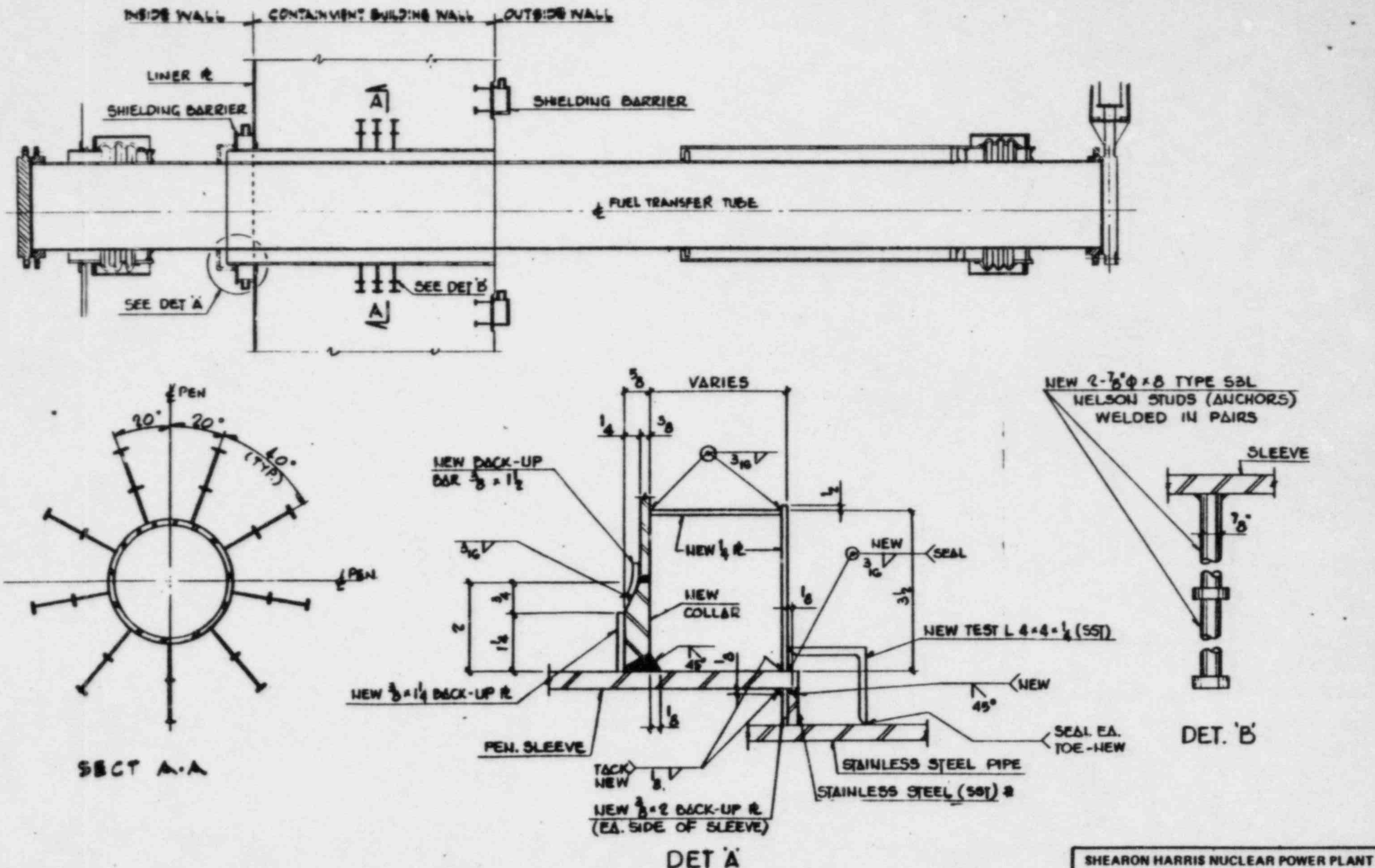
FIGURE 3.8.1-18



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT BUILDING
ELECTRICAL TYPE III PENETRATION

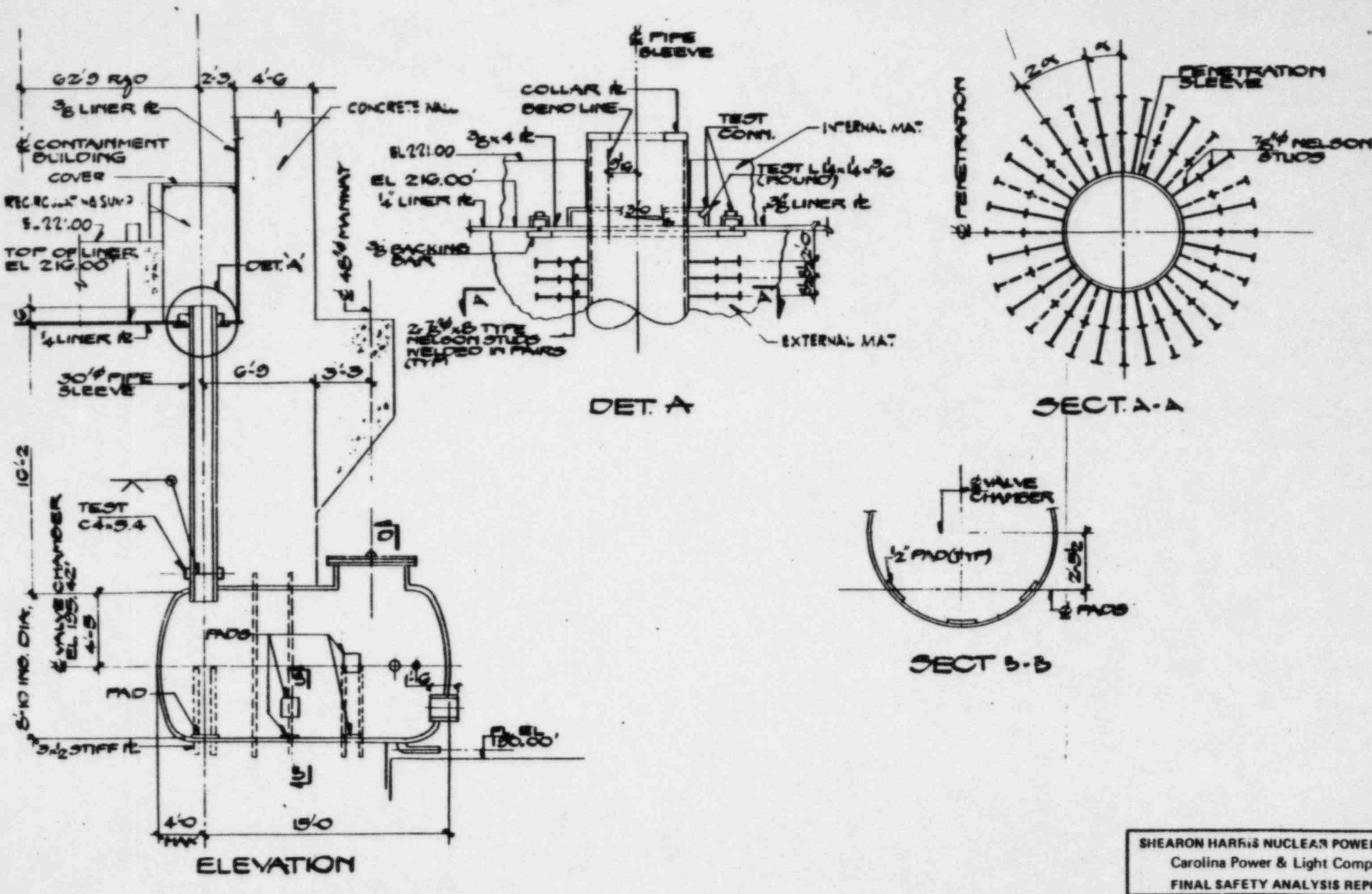
FIGURE 3.8.1-19



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

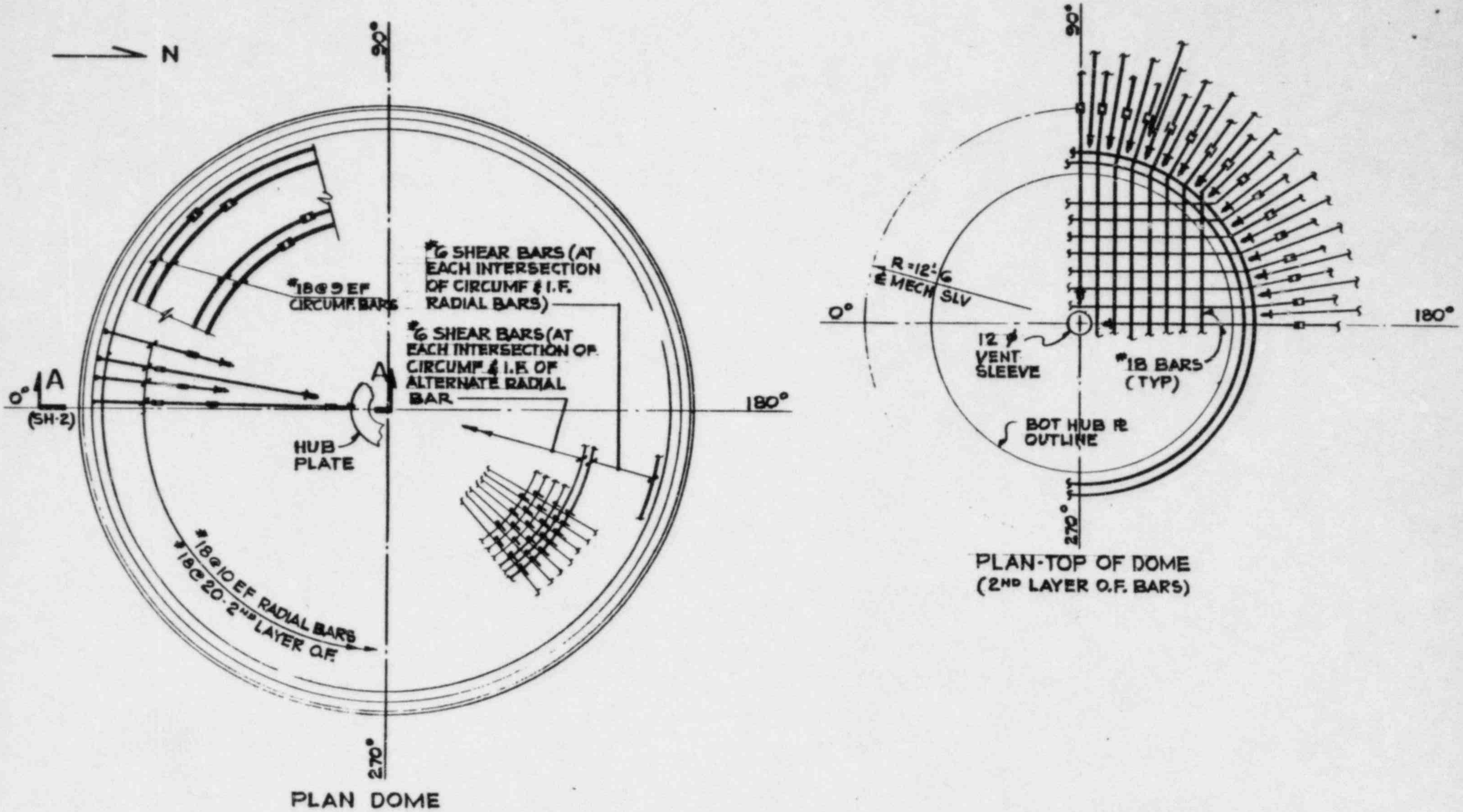
CONCRETE CONTAINMENT BUILDING
FUEL TRANSFER TUBE PENETRATION

FIGURE 3.8.1-20



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

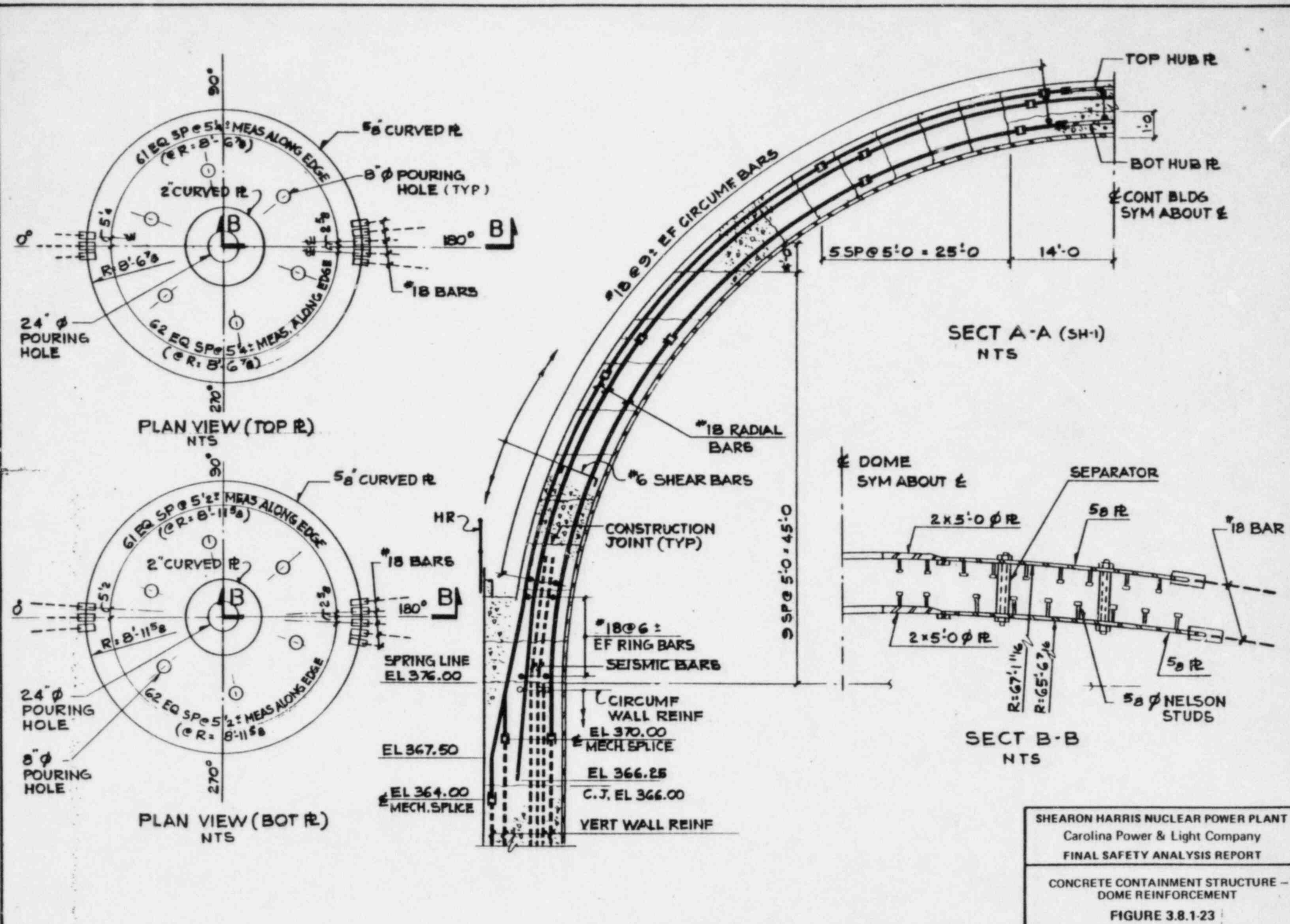
CONCRETE CONTAINMENT STRUCTURE
VALVE CHAMBER
FIGURE 3.8.1-21



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE – DOME REINFORCEMENT

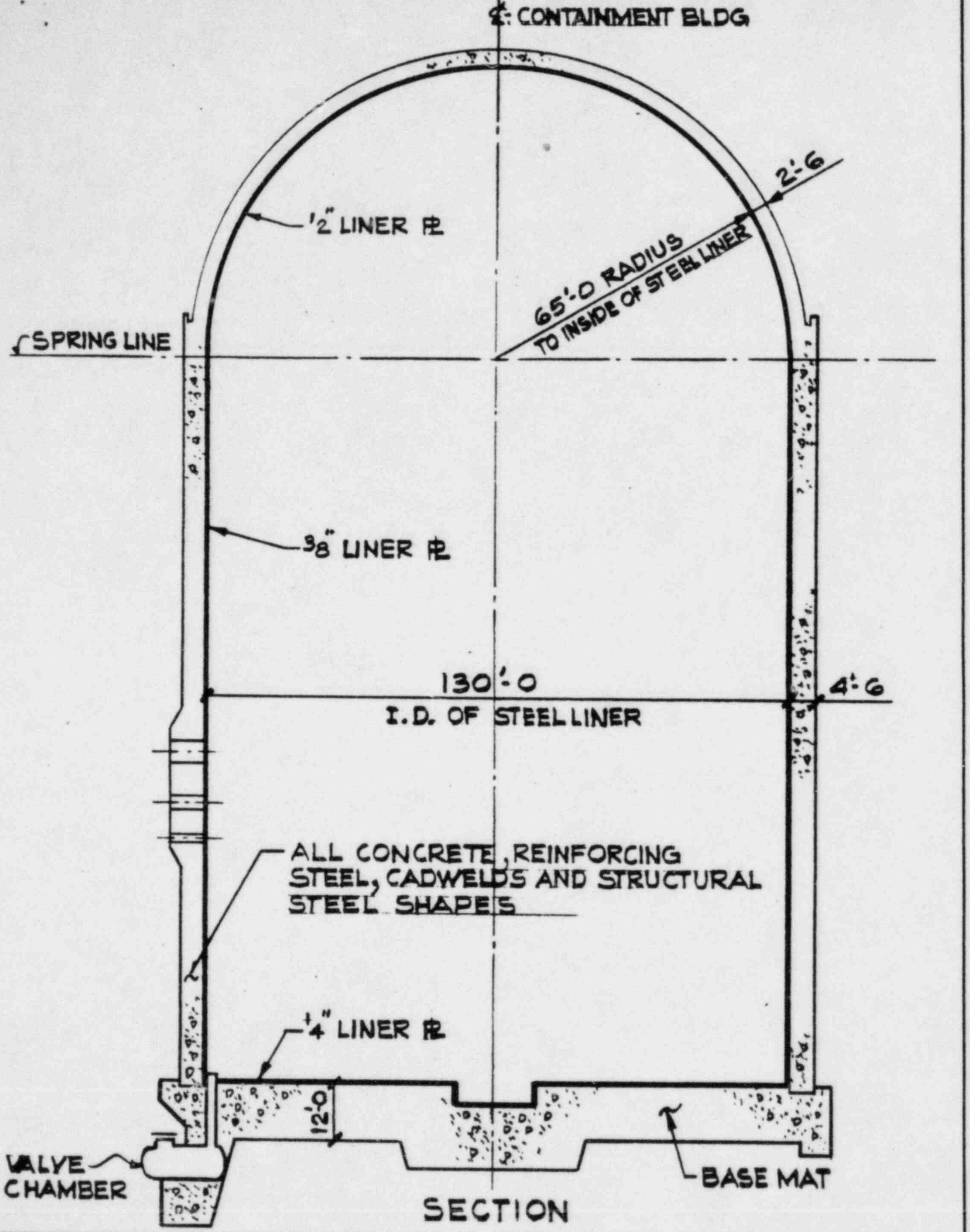
FIGURE 3.8.1-22



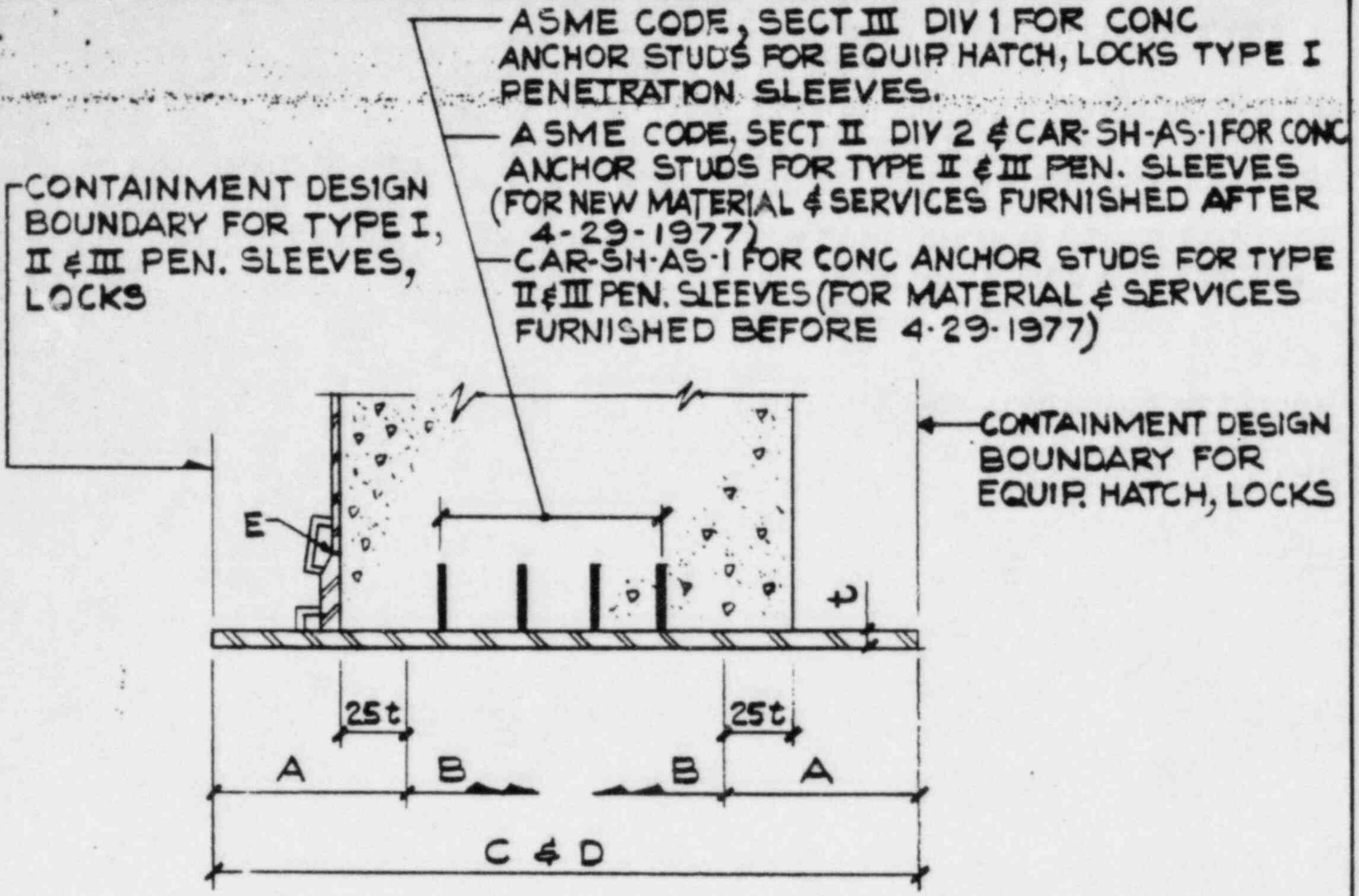
SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE – DOME REINFORCEMENT

FIGURE 3.8.1-23



SHEARON HARRIS NUCLEAR POWER PLANT Carolina Power & Light Company FINAL SAFETY ANALYSIS REPORT	CONCRETE CONTAINMENT STRUCTURE BOUNDARIES	FIGURE 3.8.1-24



A - DIV. BOUNDARY FOR ASME CODE, SECT III, DIV I WITH SUBSECTION CA (FOR NEW MATERIAL & SERVICES FURNISHED AFTER 4-29-1977) FOR TYPE II & III PENETRATION SLEEVES.

B - DIV. BOUNDARY FOR ASME CODE SECT III, DIV 2 (FOR NEW MATERIAL & SERVICES FURNISHED AFTER 4-29-1977) FOR TYPE II & III PENE. SLEEVES

C - DIV. BOUNDARY FOR ASME CODE SECT III DIV 1 WITH SUBSECTION NA FOR EQUIP. HATCH, LOCKS TYPE I PEN SLEEVES.

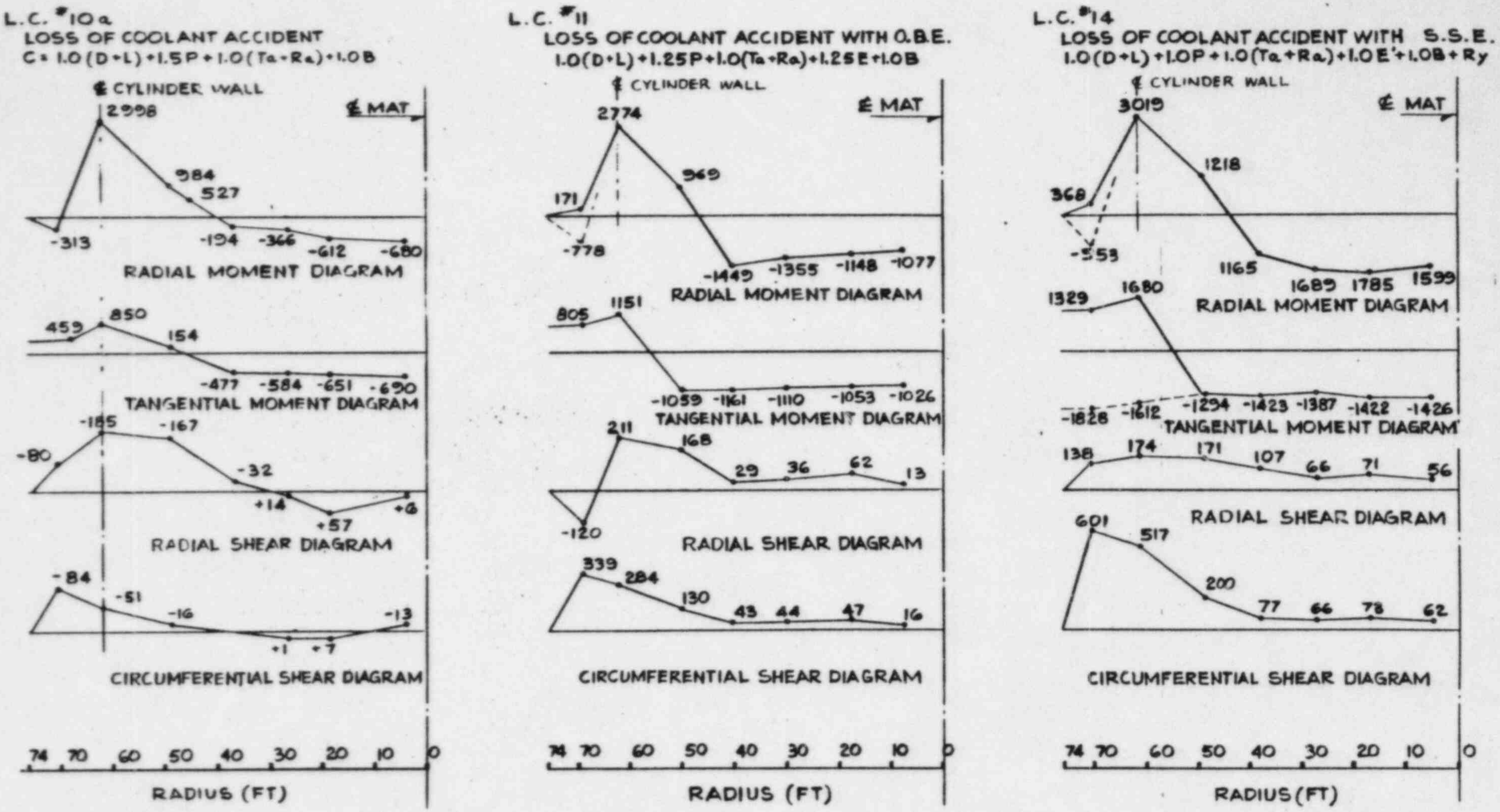
D - DIV. BOUNDARY FOR ASME CODE, SECT III, DIV 1 WITH SUBSECTION NA FOR TYPE II & III PEN SLEEVES (FOR MATERIAL & SERVICES FURNISHED BEFORE 4-29-1977)

E - ATTACHMENT WELD BETWEEN ASME CODE SECTION III DIVISION 1 ITEMS AND LINER ARE IN ACCORDANCE WITH THE ASME CODE SECTION III DIVISION 2

DIV. BOUNDARIES FOR THE TYPE II & III PENETRATION SLEEVES

LOCKS AND EQUIPMENT HATCH

SHEARON HARRIS NUCLEAR POWER PLANT Carolina Power & Light Company FINAL SAFETY ANALYSIS REPORT	CONCRETE CONTAINMENT STRUCTURE PENETRATION BOUNDARY	FIGURE 3.8.1-25

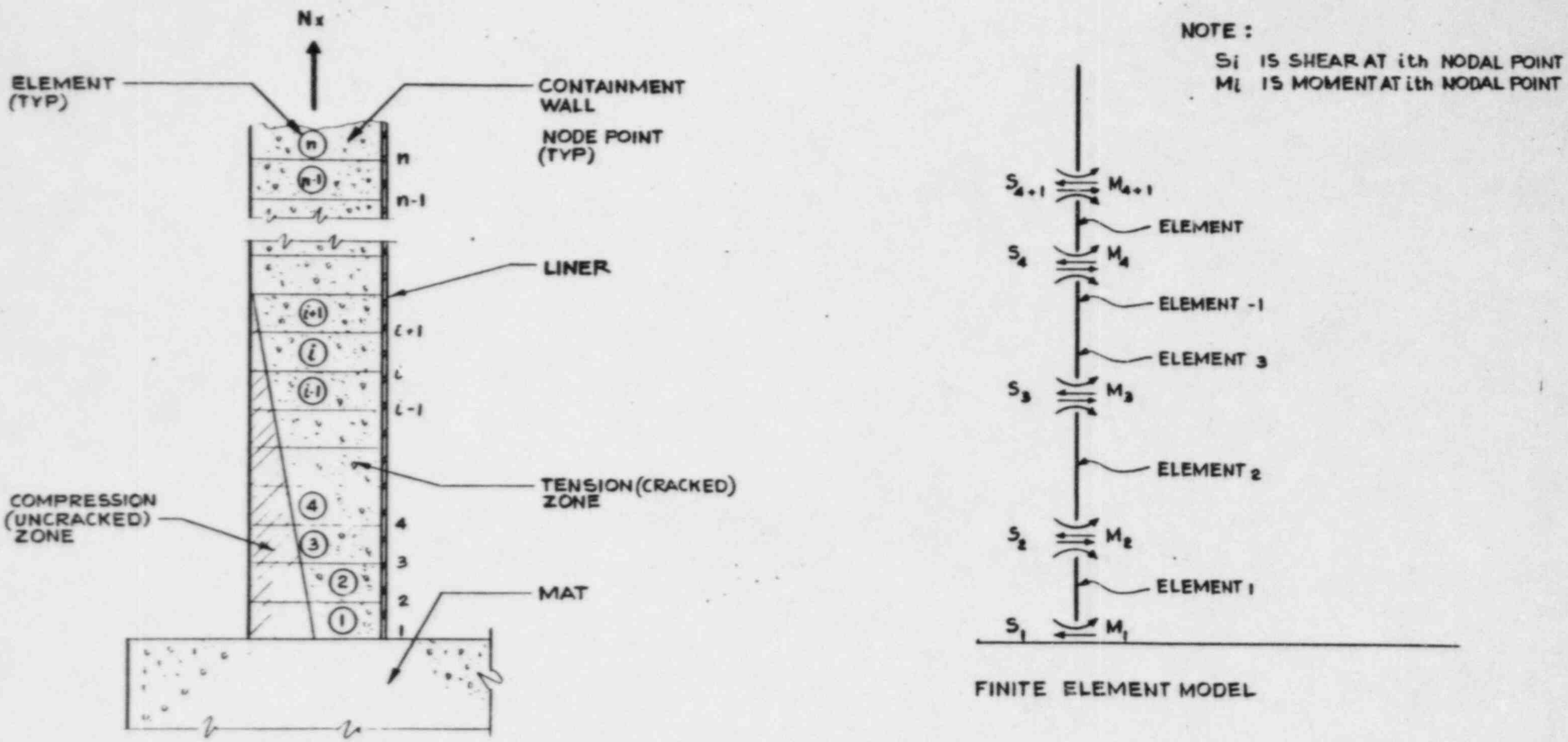


UNITS: MOMENT IN "FT-KIP"
SHEAR IN "KIP"

SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE MAT STRUCTURAL RESPONSES

FIGURE 3.8.1-26



MODELING OF CRACKS

FINITE ELEMENT MODEL

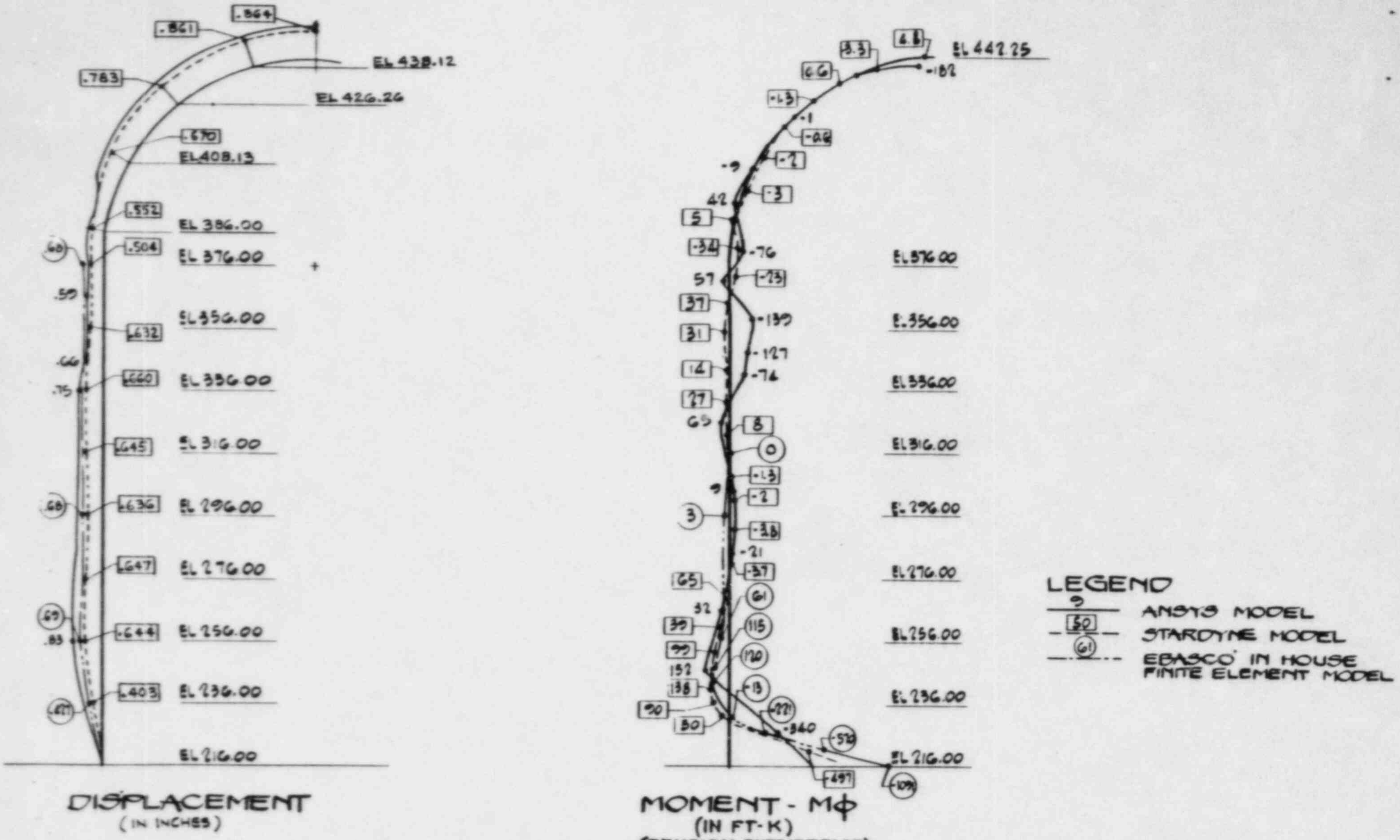
SHEARON HARRIS NUCLEAR POWER PLANT

Carolina Power & Light Company

FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE CYLINDRICAL WALL FINITE ELEMENT MODEL AND MODELING OF CRACKS

FIGURE 3.8.1-27

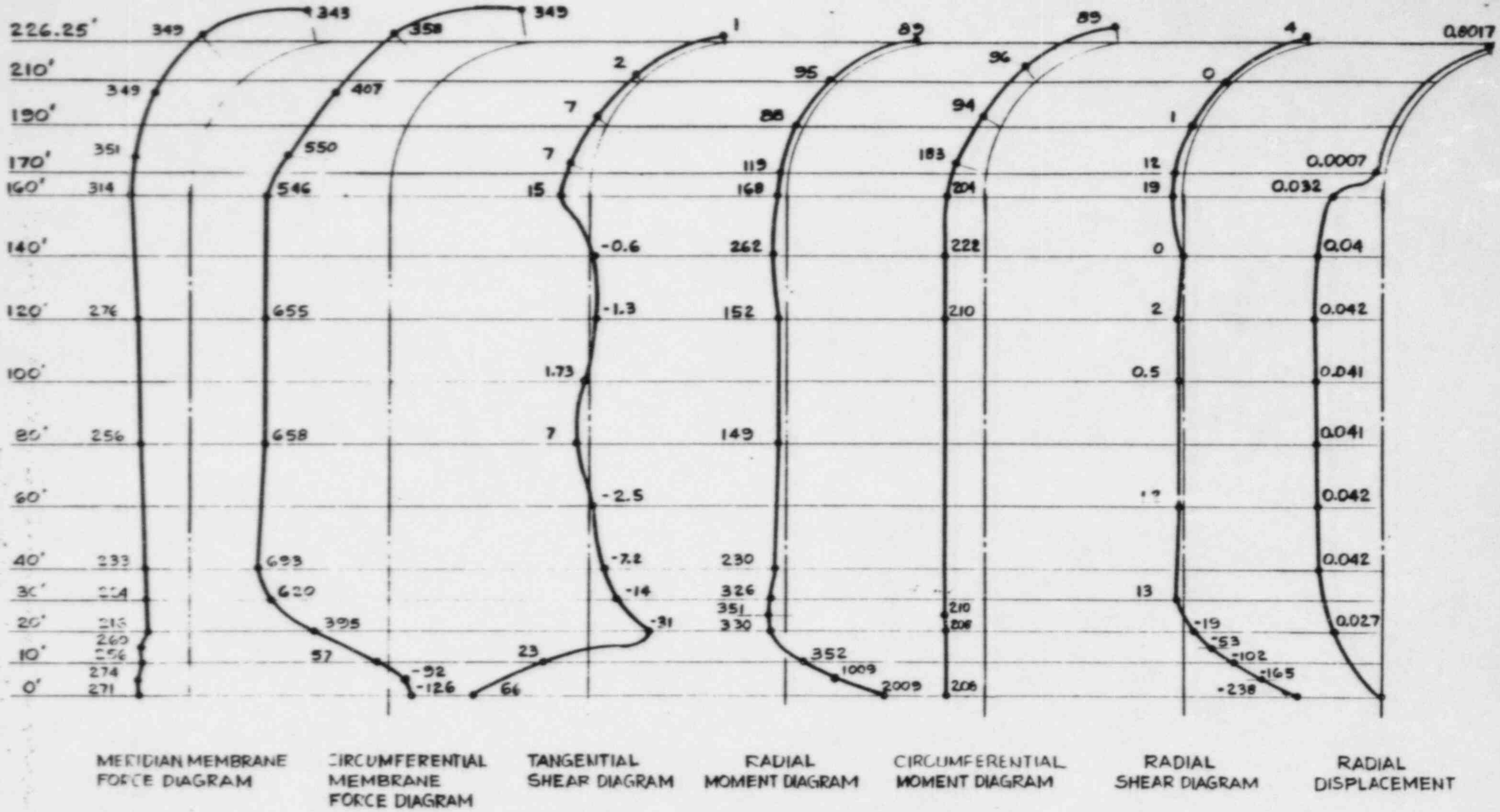


NOTE:
DIAGRAMS ARE FOR TEST PRESSURE LOAD COMBINATION

Amendment No. 5

SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE
AXISYMMETRIC LOADS
FINITE ELEMENT MODELS
COMPARATIVE STUDY RESULTS
FIGURE 3.8.1-28



UNITS: FORCE IN KIPS, MOMENT IN FT-KIP, DISPLACEMENT IN FT

MEMBRANE FORCE SHOWN: TENSION IS POSITIVE
COMPRESSION IS NEGATIVE

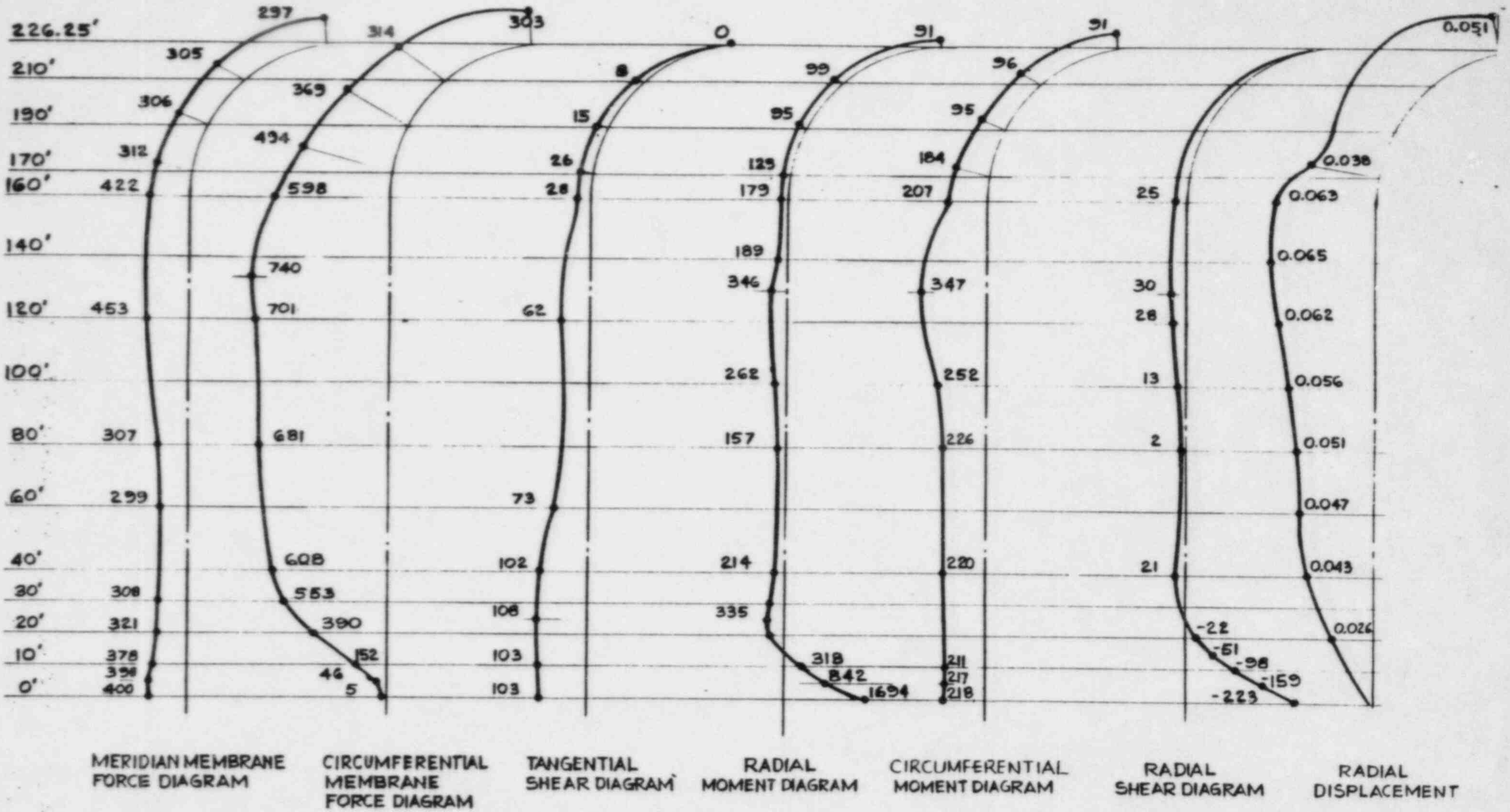
MOMENT IS SHOWN ON TENSION SIDE

LOAD COMBINATION #10 a
LOSS OF COOLANT ACCIDENT
C: 1.0(D+L)+1.5P+1.0(Ta+Ra)

SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE – CYLINDRICAL WALL AND DOME STRUCTURAL RESPONSES

FIGURE 3.8.1-29



UNITS: FORCE IN KIPS, MOMENT IN FT – KIP, DISPLACEMENT IN FT.

MEMBRANE FORCE SHOWN: TENSION IS POSITIVE
COMPRESSION IS NEGATIVE

MOMENT IS SHOWN ON TENSION SIDE

LOAD COMBINATION NO. 11

LOSS OF COOLANT ACCIDENT WITH OBE

$C = 1.0\,(D+L) + 1.25P + 1.0\,(T_a + R_a) + 1.25\,E$

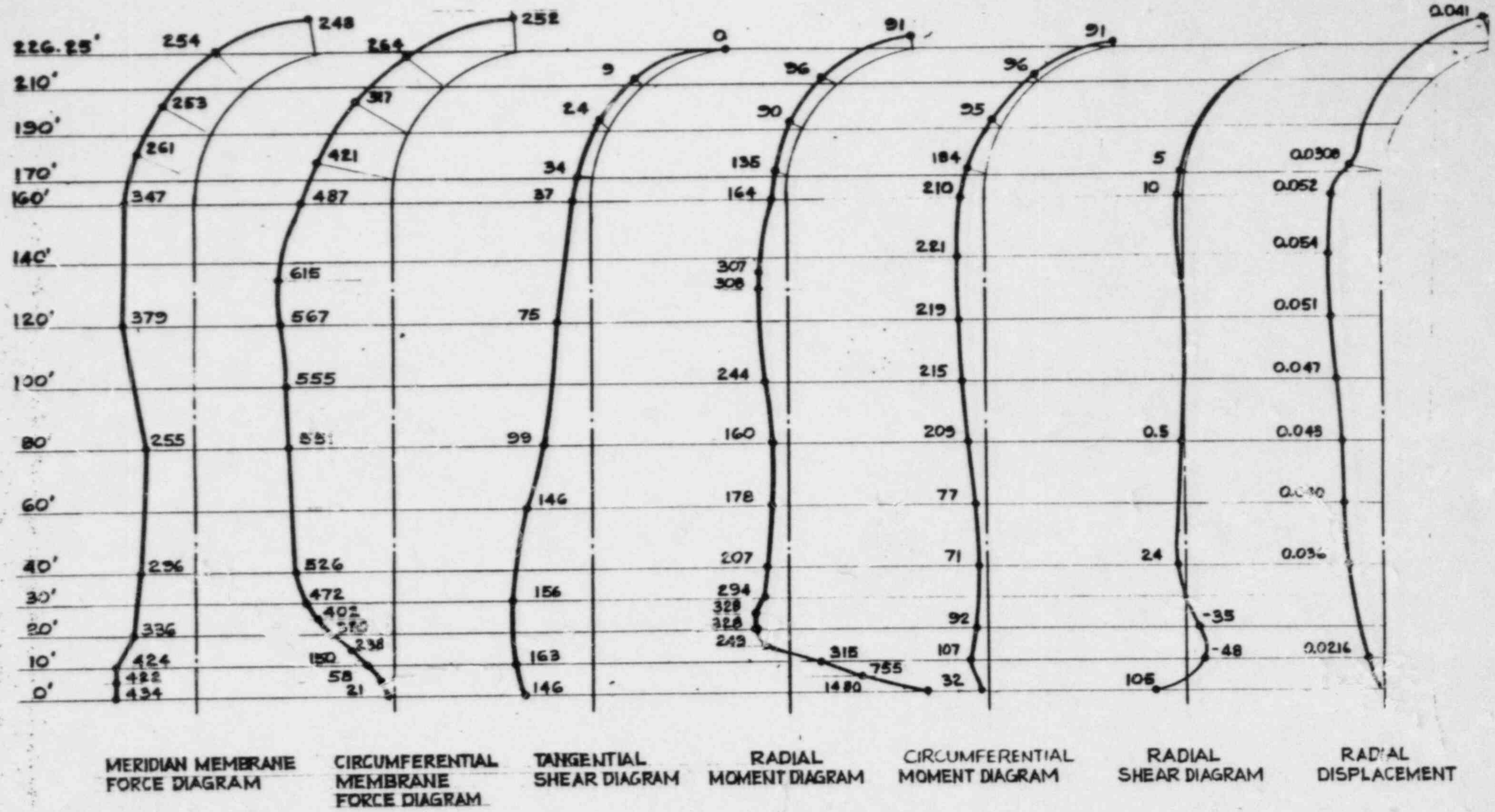
SHEARON HARRIS NUCLEAR POWER PLANT

Carolina Power & Light Company

FINAL SAFETY ANALYSIS REPORT.

CONCRETE CONTAINMENT STRUCTURE – CYLINDRICAL WALL AND DOME STRUCTURAL RESPONSES

FIGURE 3.8.1-30



UNITS: FORCE IN KIPS, MOMENT IN FT-KIP, DISPLACEMENT IN FT.

MEMBRANE FORCE SHOWN: TENSION IS POSITIVE
COMPRESSION IS NEGATIVE

MOMENT IS SHOWN ON TENSION SIDE

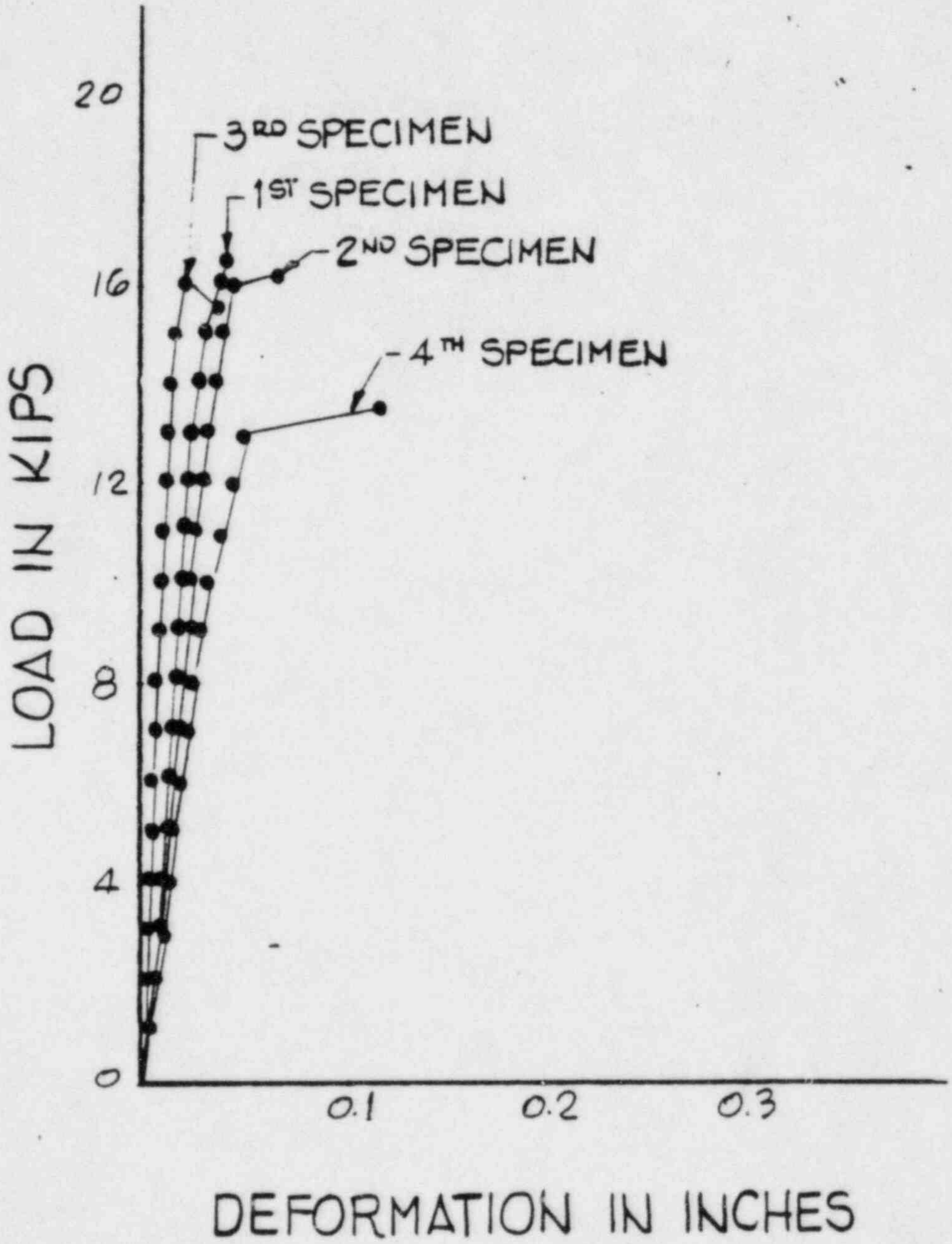
LOAD COMBINATION #14
LOSS OF COOLANT ACCIDENT WITH SSE
C = 1.0 (D + L + P + Ta + Ra + E + Rr)

SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE – CYLINDRICAL WALL AND DOME STRUCTURAL RESPONSES

FIGURE 3.8.1-31

CURVE	SPRING CONSTANT K/IN.	ULTIMATE LOAD K	ULTIMATE DEFORMATION IN.
1	400	16.50	.046
2	311	16.25	.070
3	750	15.50	.037
4	275	13.73	.125



SHEARON HARRIS NUCLEAR POWER PLANT

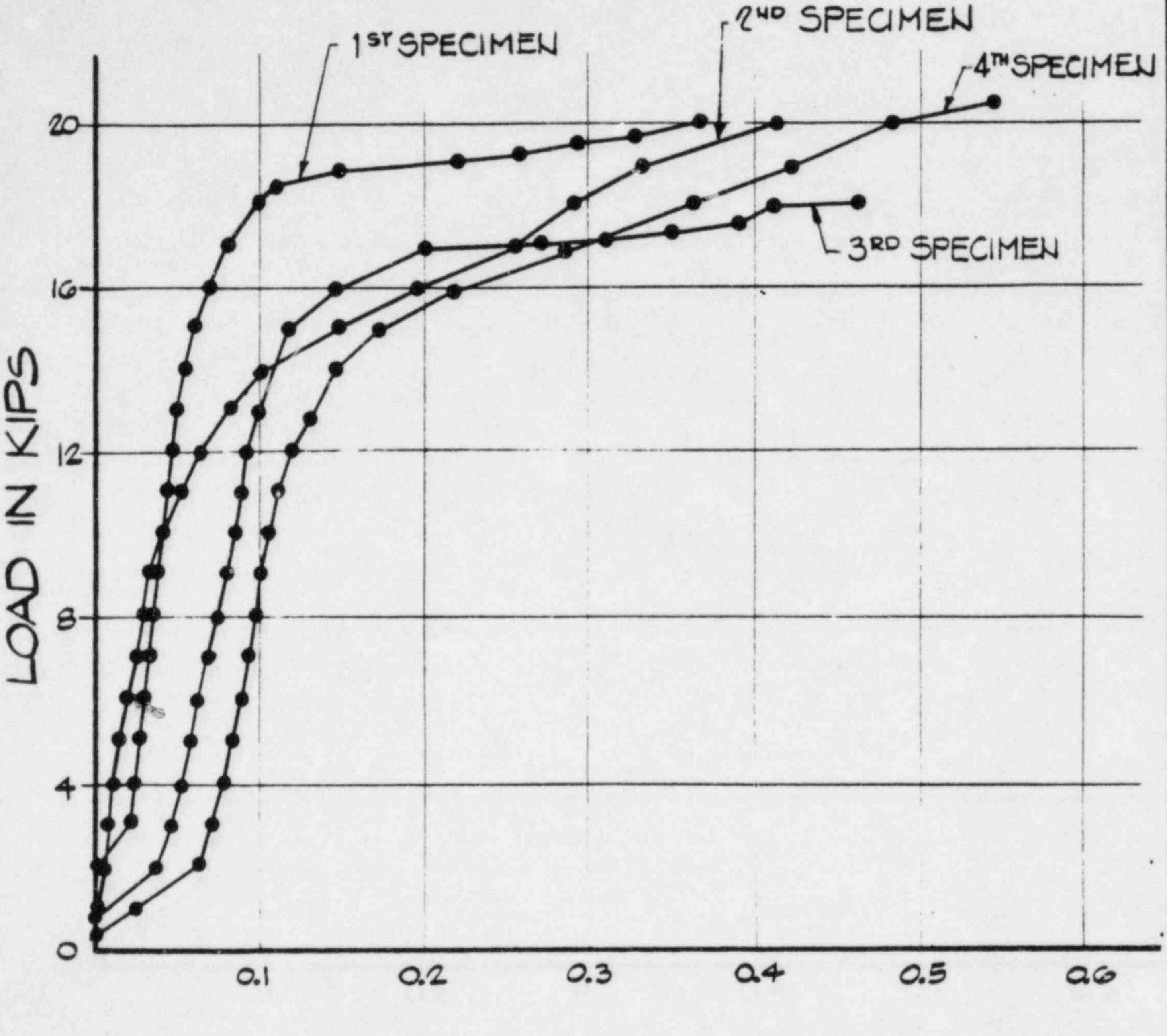
Carolina Power & Light Company

FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE
TEST OF 5/8" DIAMETER X 4" LONG HEADED STUDS IN TENSION-CONCRETE IN TENSION

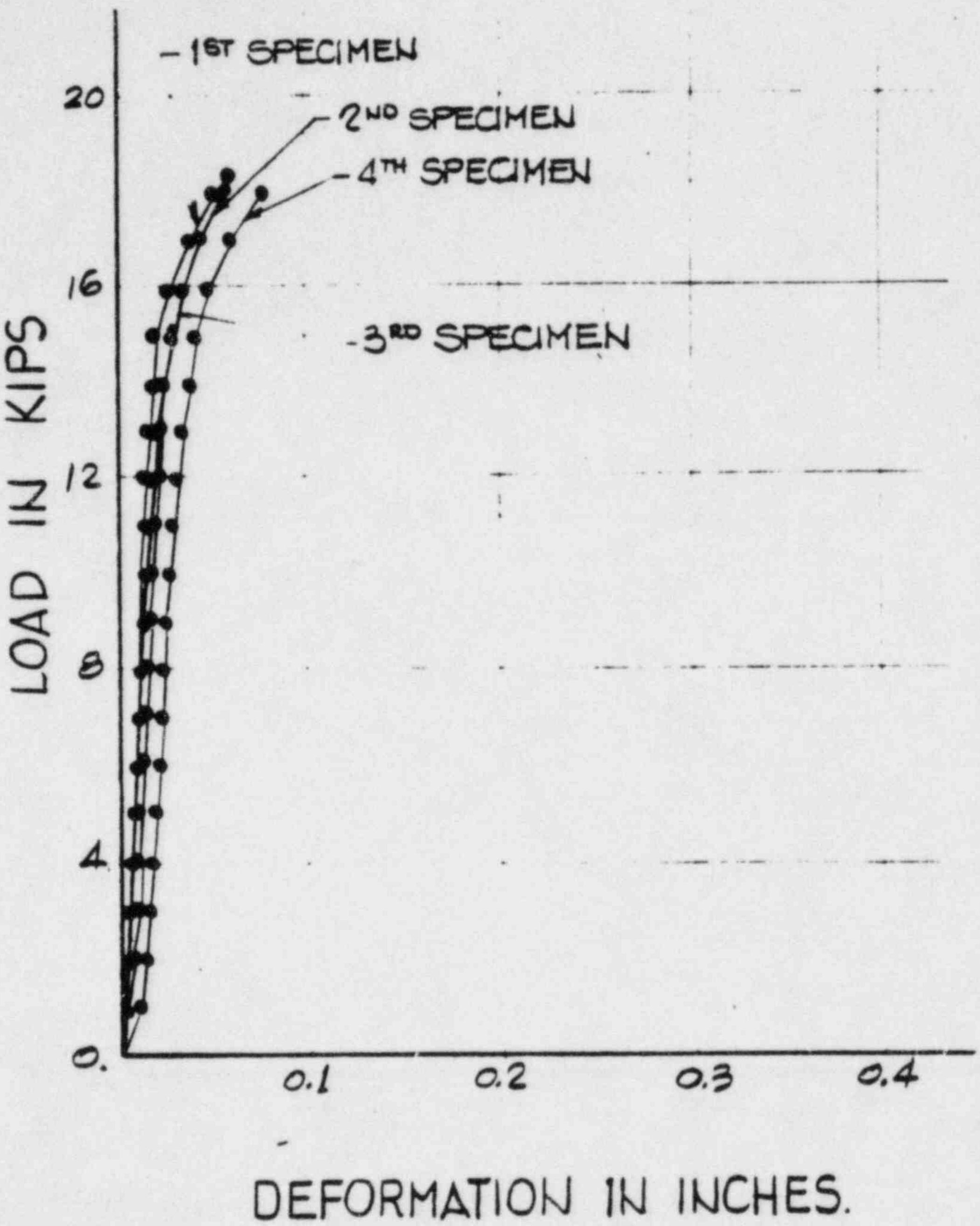
FIGURE 3.8.1-32

CURE	SPRING CONSTANT K/IN.	ULTIMATE LOAD K	ULTIMATE DEFORMATION IN
1	333	20.0	0.367
2	267	19.9	0.413
3	172	18.2	0.469
4	200	20.5	0.546



SHEARON HARRIS NUCLEAR POWER PLANT Carolina Power & Light Company FINAL SAFETY ANALYSIS REPORT	CONCRETE CONTAINMENT STRUCTURE TEST OF 5/8" DIAMETER X 4" LONG HEADED STUDS IN SHEAR-CONCRETE IN TENSION	FIGURE 3.8.1-33

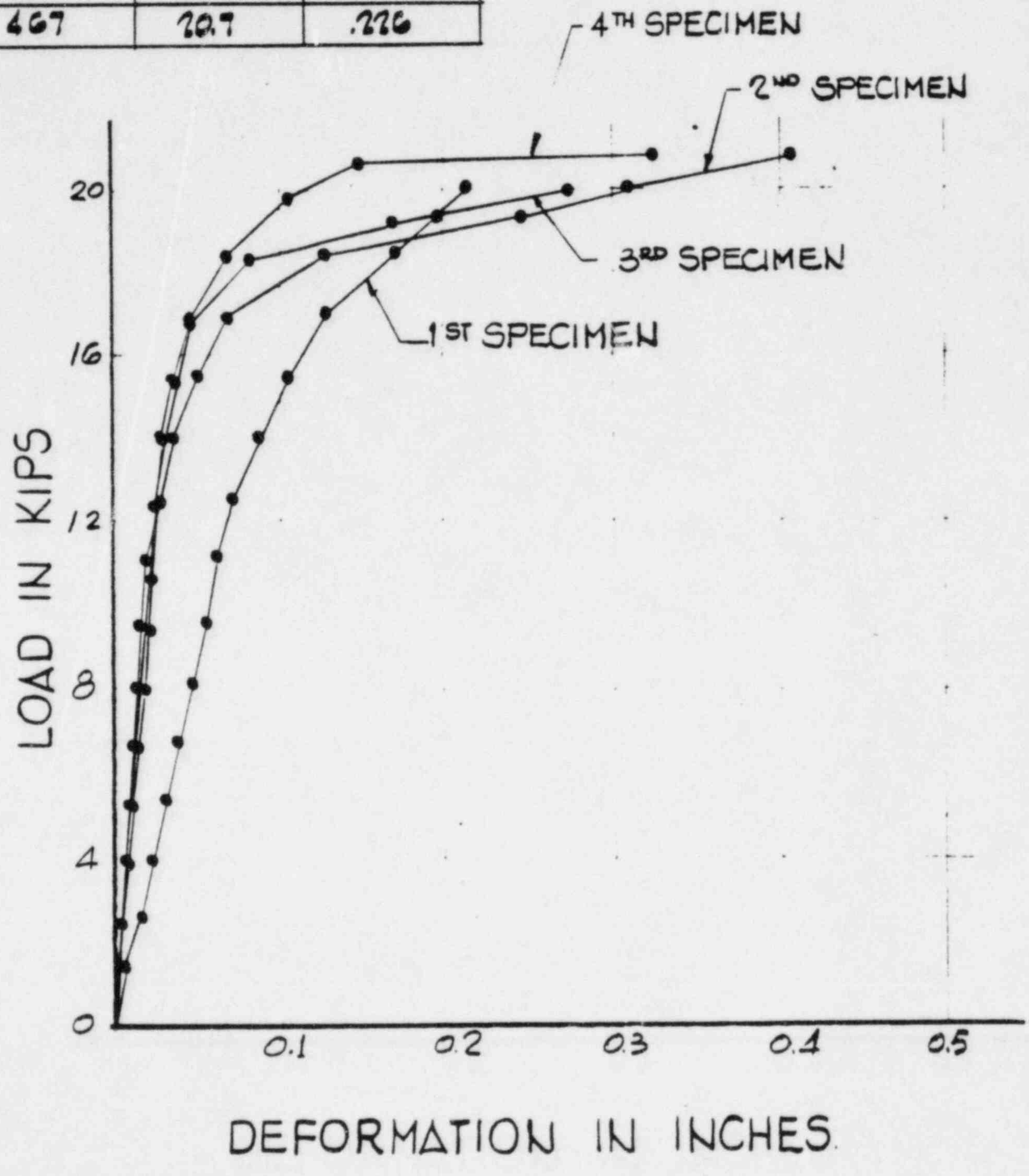
CURVE	SPRING CONSTANT K/IN.	ULTIMATE LOAD K	ULTIMATE DEFORMATION IN.
1	700	18.2	.06
2	360	18.0	.056
3	733	16.0	.035
4	480	18.6	.096



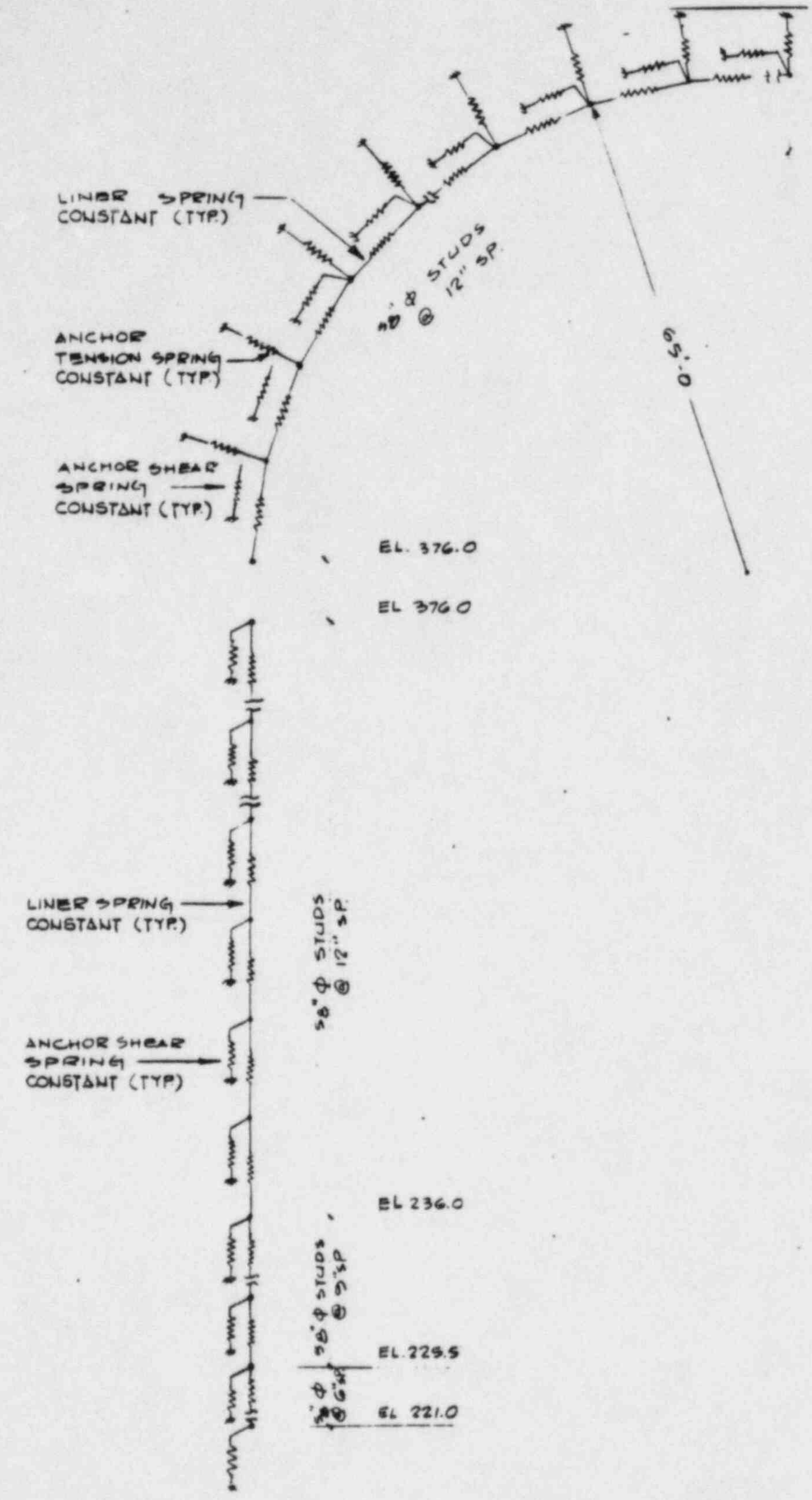
CONCRETE CONTAINMENT STRUCTURE
TEST OF 5/8" DIAMETER X 4" LONG HEADED STUDS IN TENSION-CONCRETE UNLOADED

FIGURE 3.8.1-34

CURVE	SPRING CONSTANT K/IN.	ULTIMATE LOAD K	ULTIMATE DEFORMATION IN.
1	182	20	.211
2	490	20.7	.410
3	400	20.0	.227
4	467	20.7	.226



SHEARON HARRIS NUCLEAR POWER PLANT Carolina Power & Light Company FINAL SAFETY ANALYSIS REPORT	CONCRETE CONTAINMENT STRUCTURE TEST OF 5/8" DIAMETER X 4" LONG HEADED STUDS IN SHEAR-CONCRETE UNLOADED	FIGURE 3.8.1-35



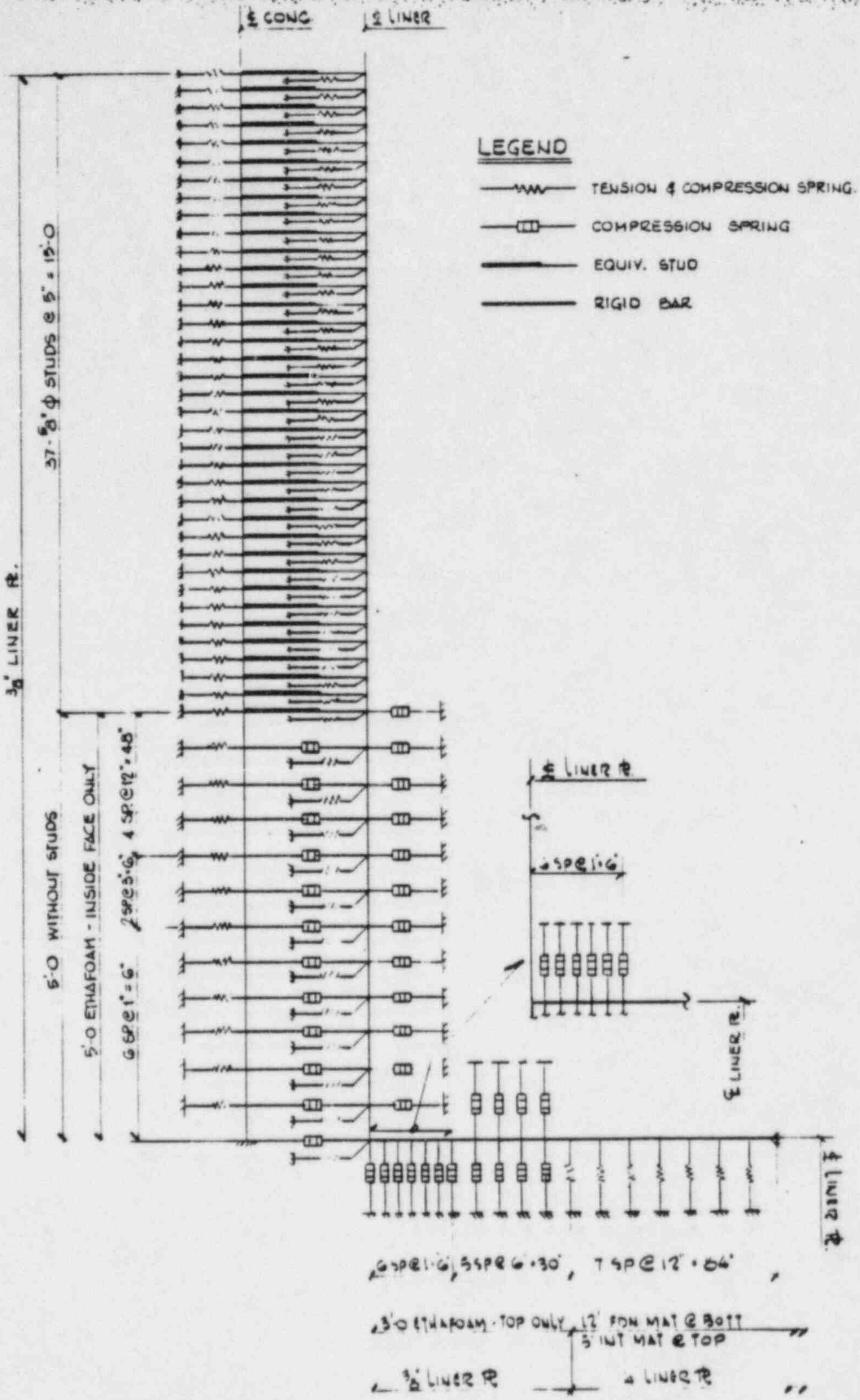
SHEARON HARRIS NUCLEAR POWER PLANT

Carolina Power & Light Company

FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE FINITE ELEMENT MODEL FOR LINER ANCHORAGE ANALYSIS

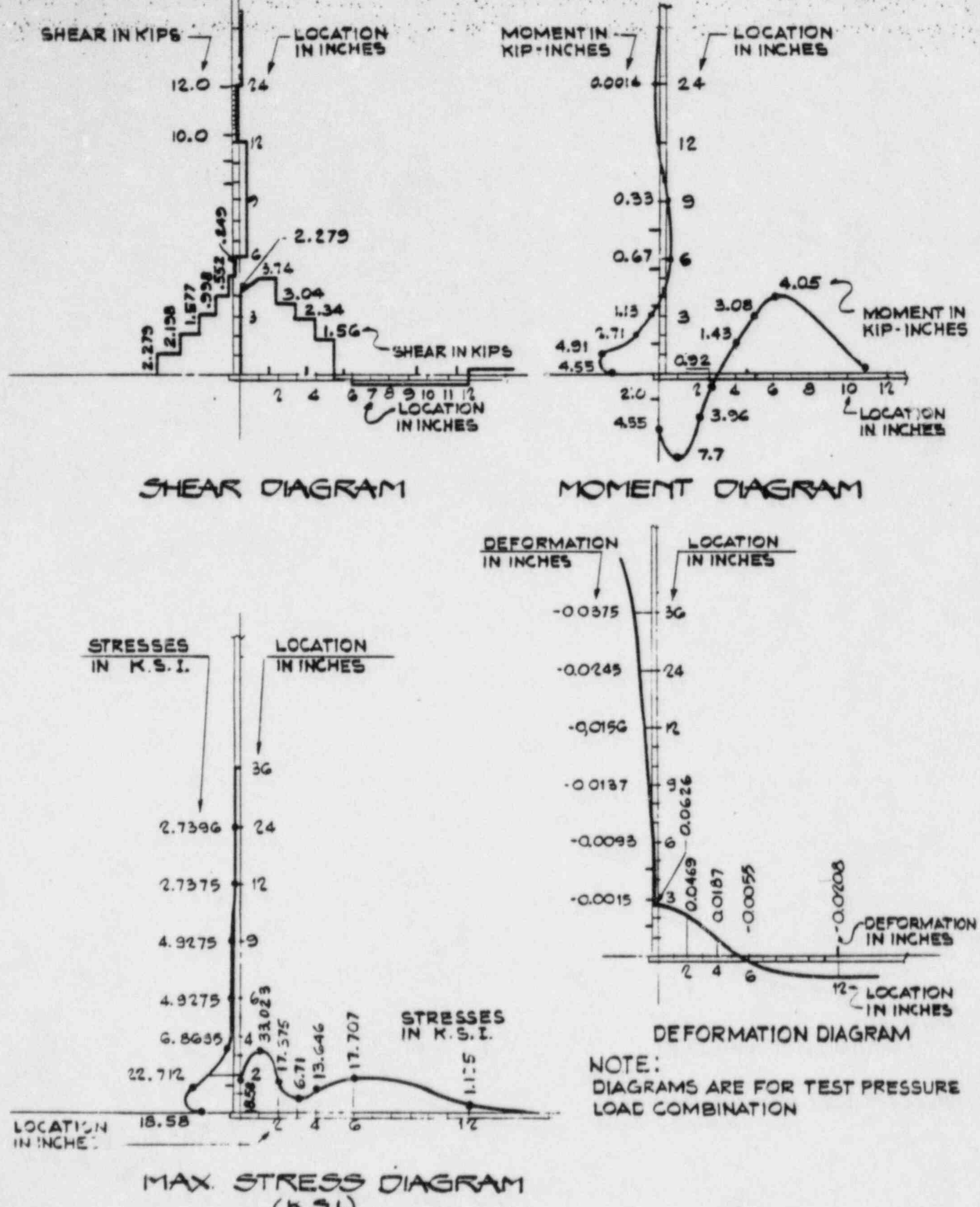
FIGURE 3.8.1-36



SHEARON HARRIS
NUCLEAR POWER PLANT
Carolina
Power & Light Company
FINAL SAFETY ANALYSIS REPORT

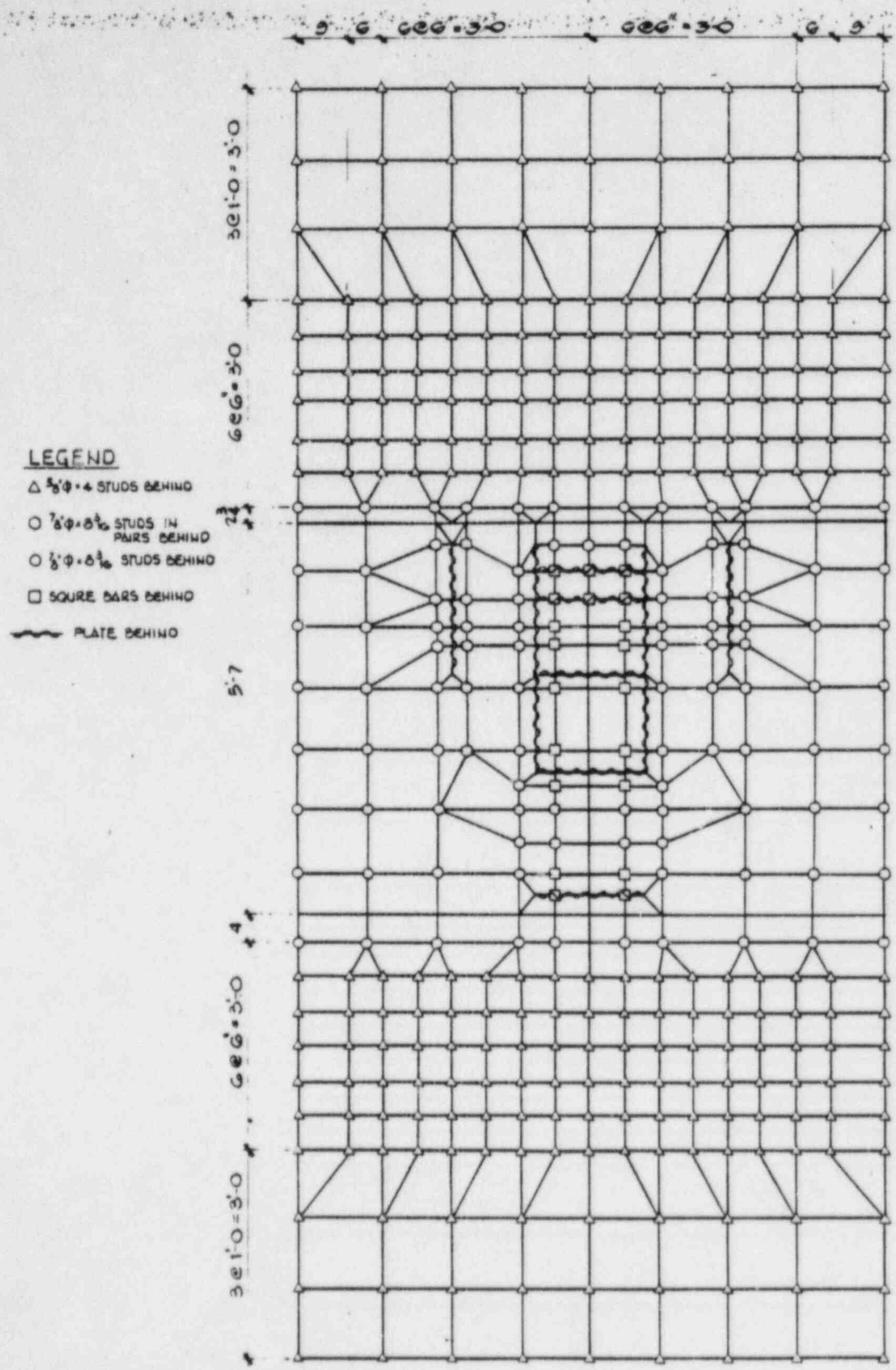
CONCRETE CONTAINMENT BUILDING
FINITE ELEMENT MODEL OF WALL
MAT. LINER CONNECTION

FIGURE
3.8.1-37



NOTE:
DIAGRAMS ARE FOR TEST PRESSURE LOAD COMBINATION

SHEARON HARRIS NUCLEAR POWER PLANT Carolina Power & Light Company FINAL SAFETY ANALYSIS REPORT	CONCRETE CONTAINMENT STRUCTURE – WALL MAT LINER CONNECTION STRUCTURAL RESPONSES	FIGURE 3.8.1-38



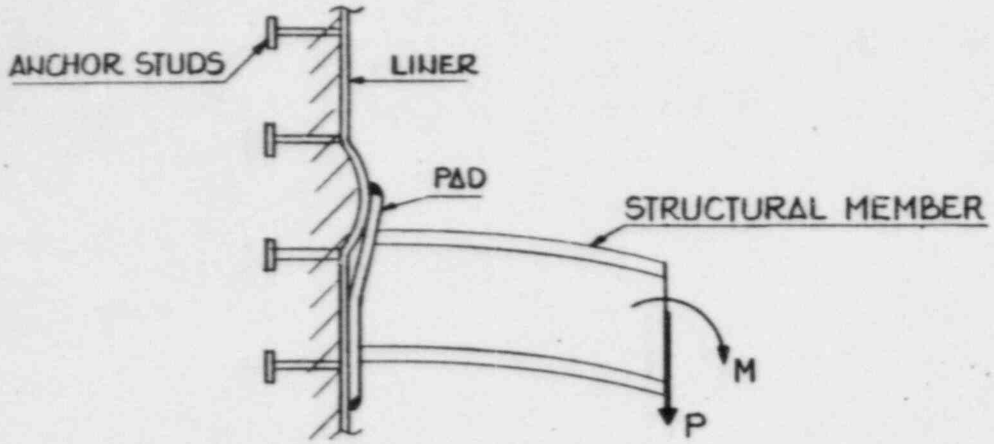
SHEARON HARRIS NUCLEAR POWER PLANT

Carolina Power & Light Company

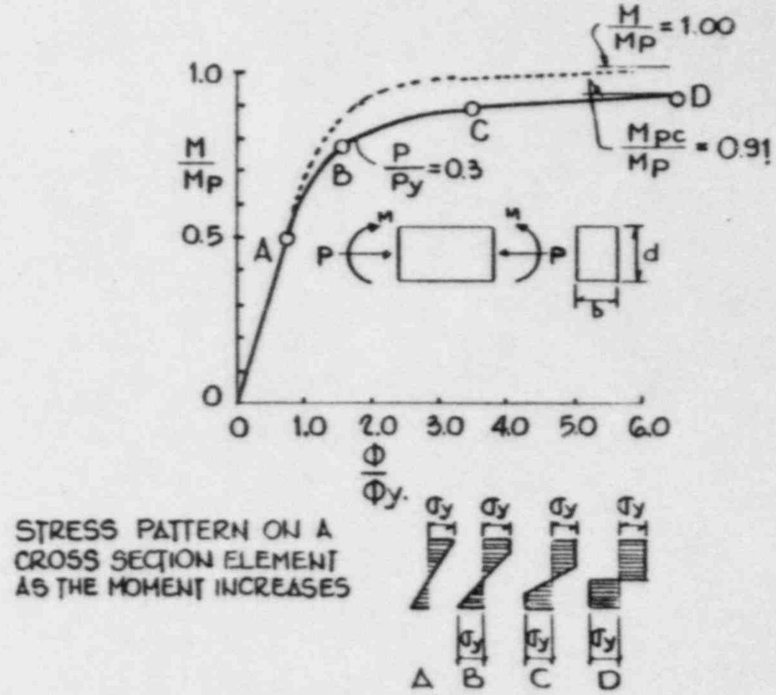
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE FINITE ELEMENT MODEL OF LINER PLATE AT CRANE GIRDER BRACKET

FIGURE 3.8.1-39



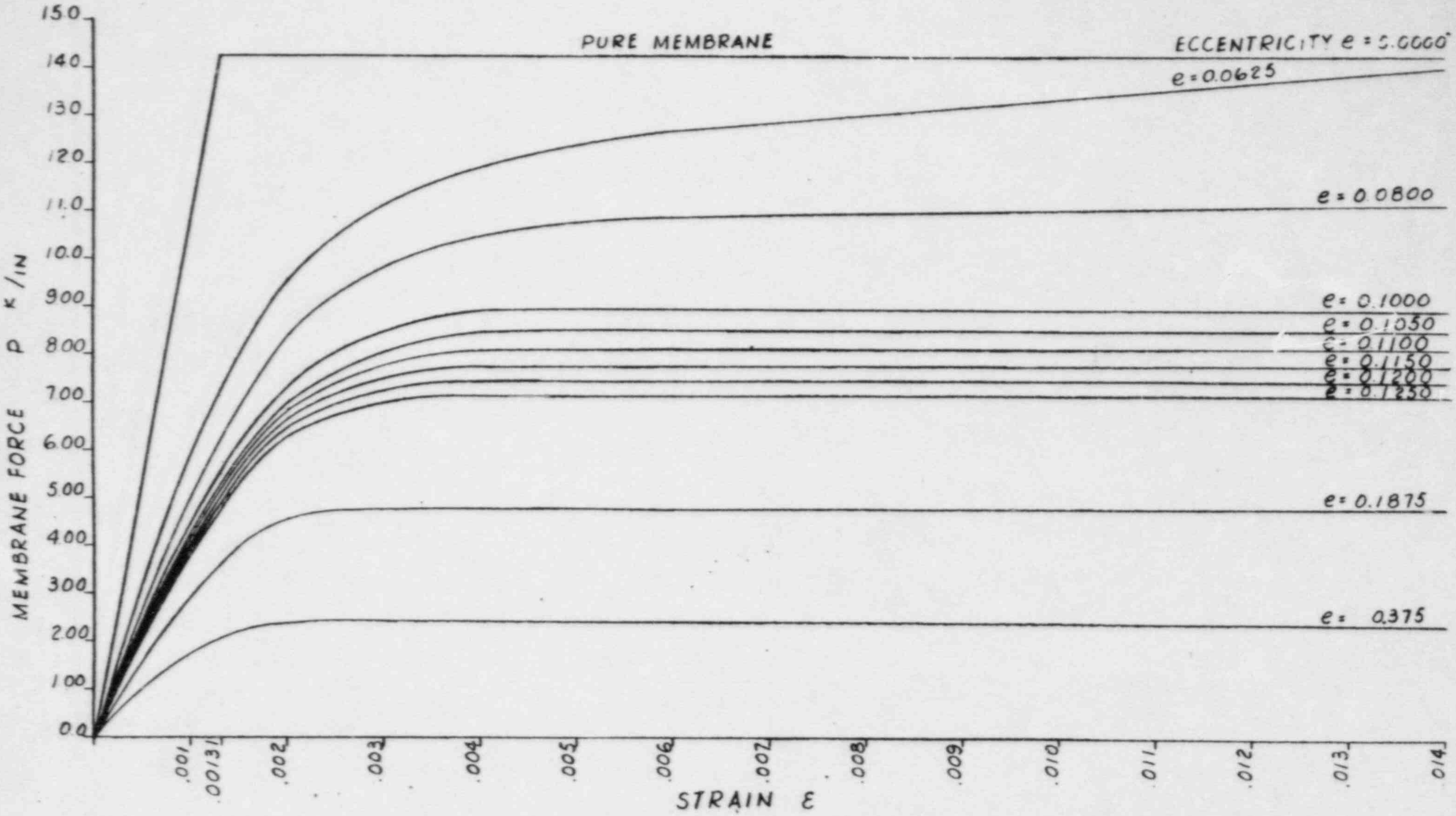
DEVELOPMENT OF ADDITIONAL BENDING STRESS
DUE TO MECHANICAL CONDITIONS



MOMENT - CURVATURE RELATIONSHIP
FOR RECTANGULAR SECTION -
PLASTIC DESIGN

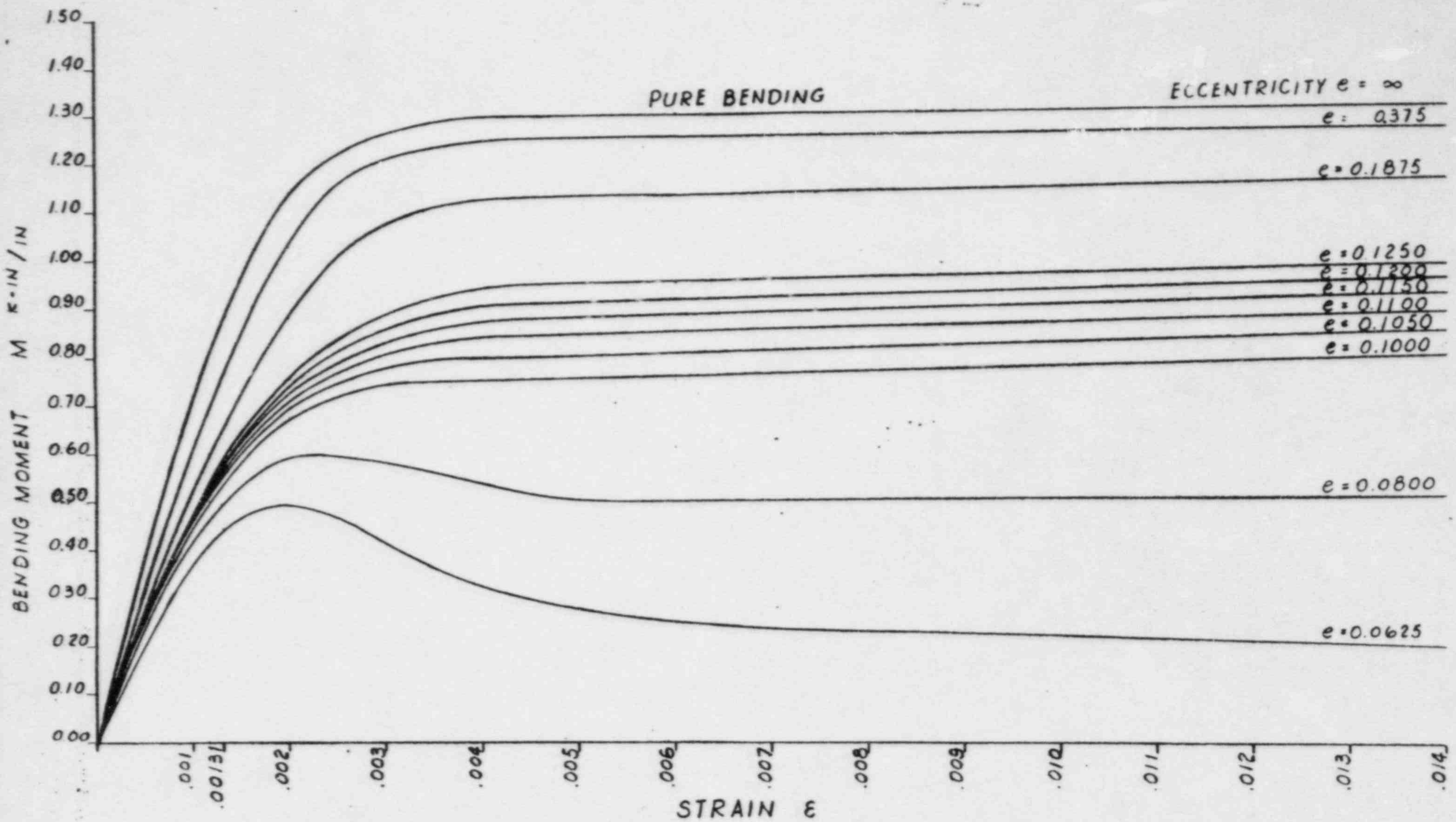
SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE
ANALYSIS OF LINER ATTACHMENTS
FIGURE 3.8.1-40



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

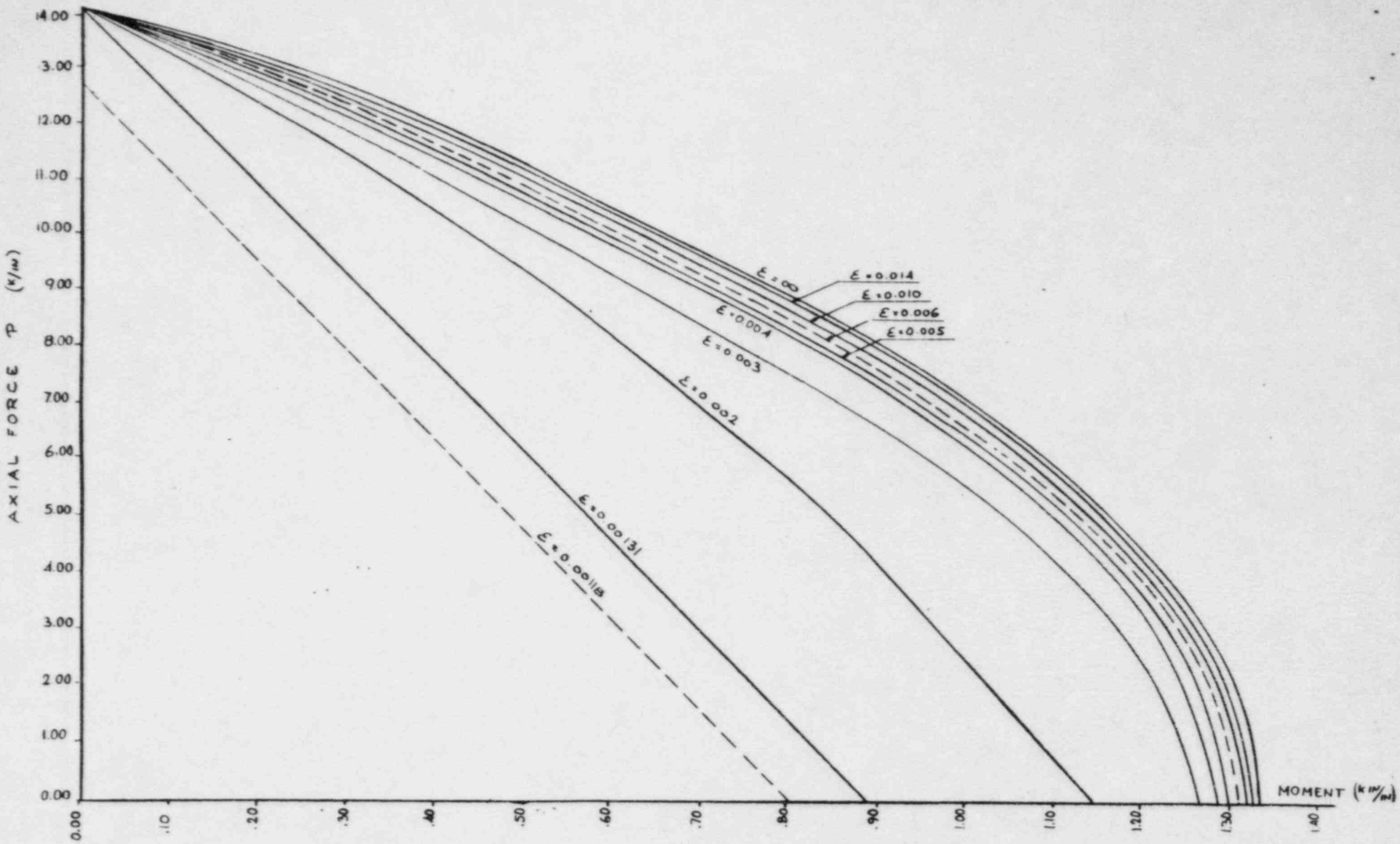
CONCRETE CONTAINMENT STRUCTURE
3/8" LINER FORCE – STRAIN DIAGRAM
FIGURE 3.8.1-41



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

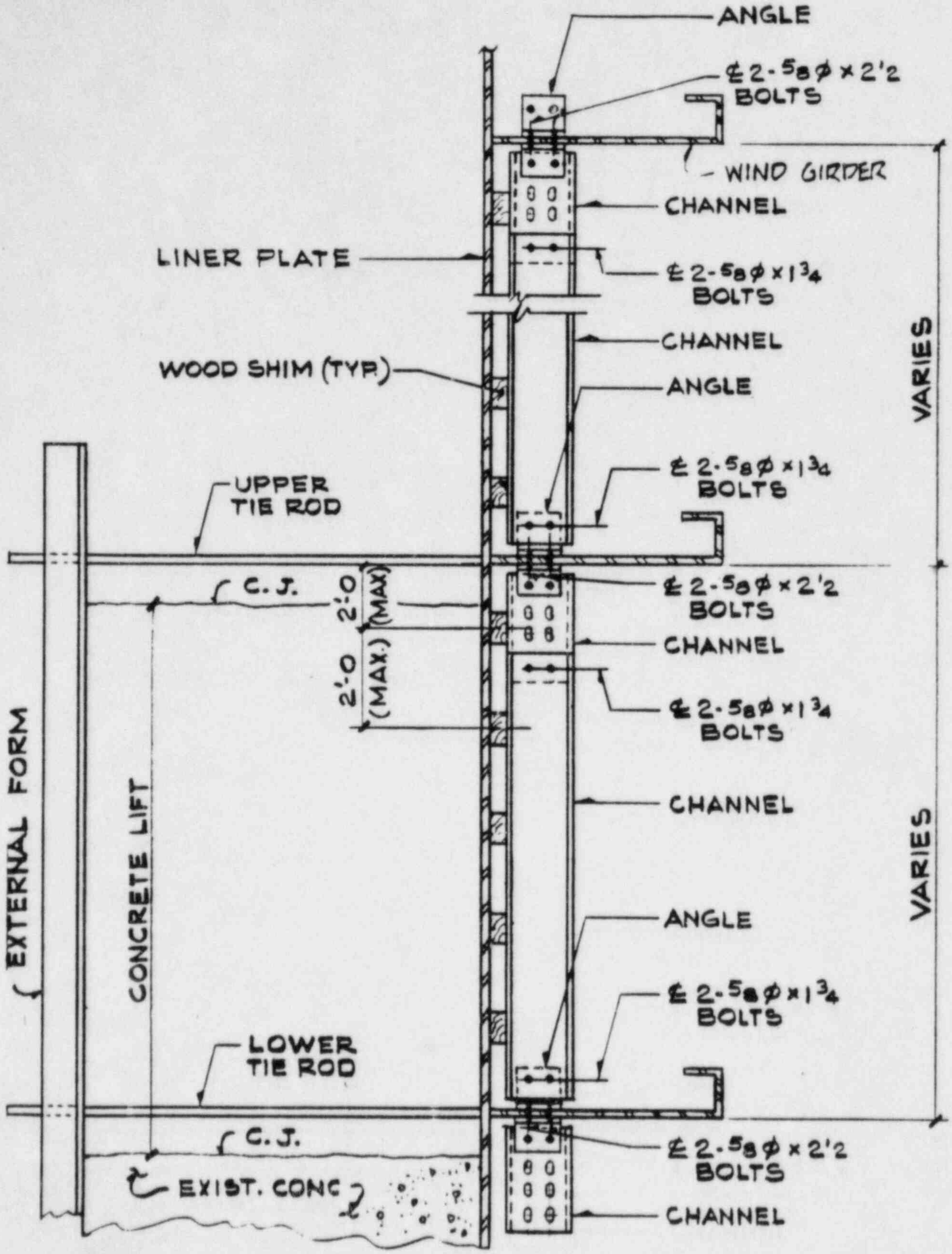
CONCRETE CONTAINMENT STRUCTURE
3/8" LINER MOMENT – STRAIN DIAGRAM

FIGURE 3.8.1-42

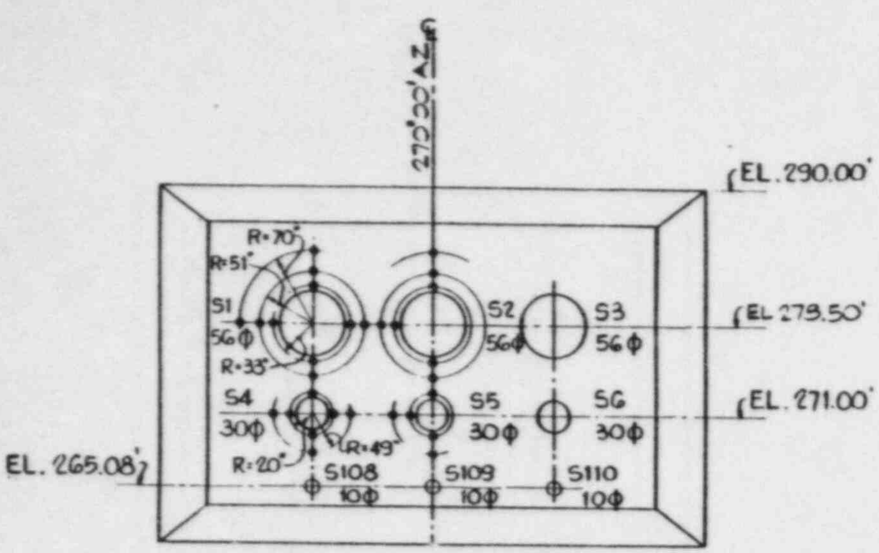


SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

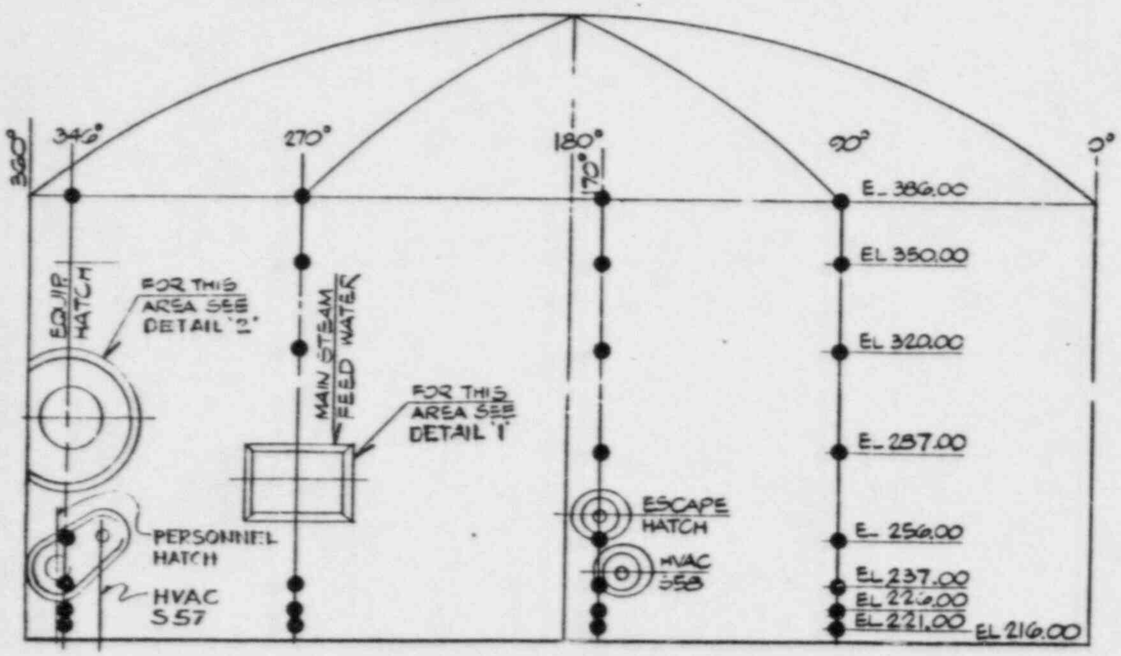
CONCRETE CONTAINMENT STRUCTURE
3/8" LINER FORCE – MOMENT
CAPACITY DIAGRAM
FIGURE 3.8.1-43



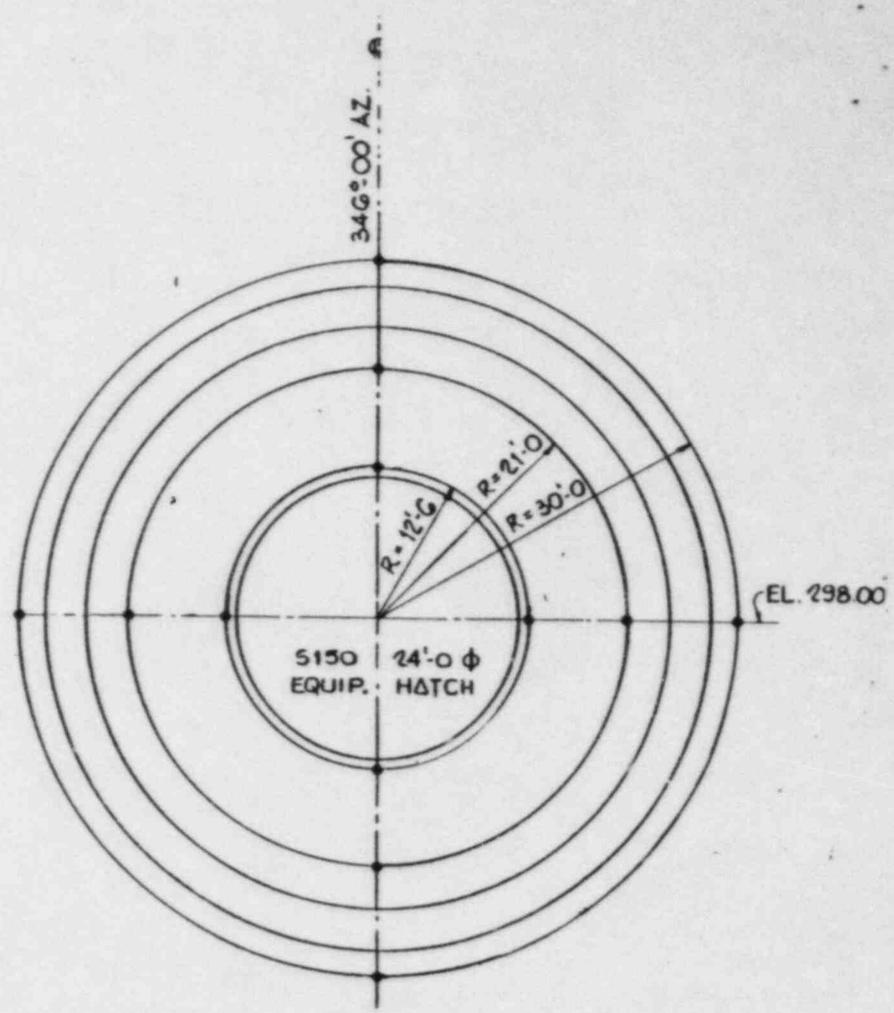
SHEARON HARRIS NUCLEAR POWER PLANT Carolina Power & Light Company FINAL SAFETY ANALYSIS REPORT	LINER REINFORCEMENT FOR CONCRETE PLACEMENT	FIGURE 3.8.1-44



DETAIL '1' MAIN STEAM-FEED WATER



DEVELOPED ELEVATION - OUTSIDE FACE OF WALL

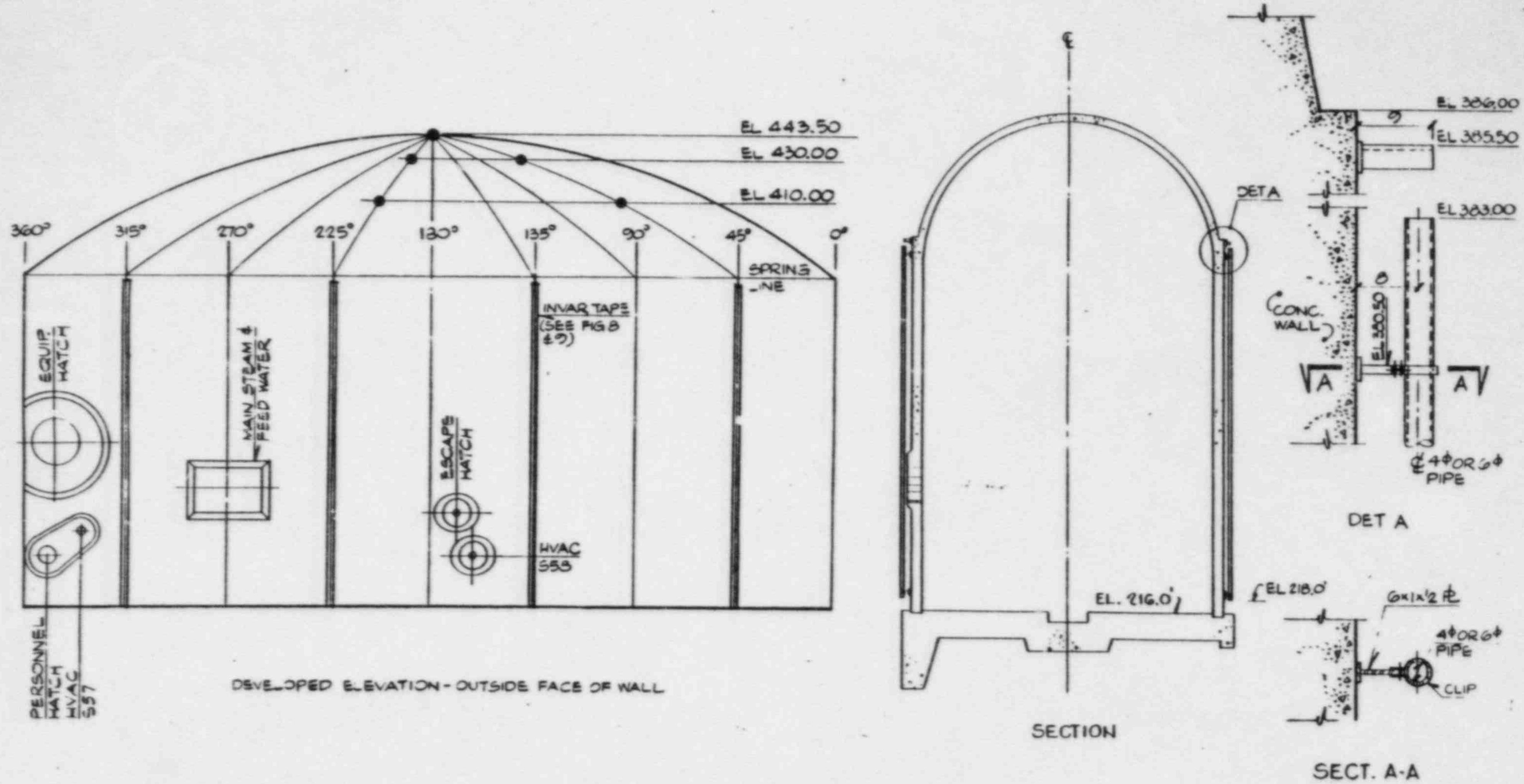


DETAIL "2"
EQUIPMENT HATCH

SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE
STRUCTURAL INTEGRITY TEST – RADIAL DISPLACEMENT MEASUREMENT LOCATIONS

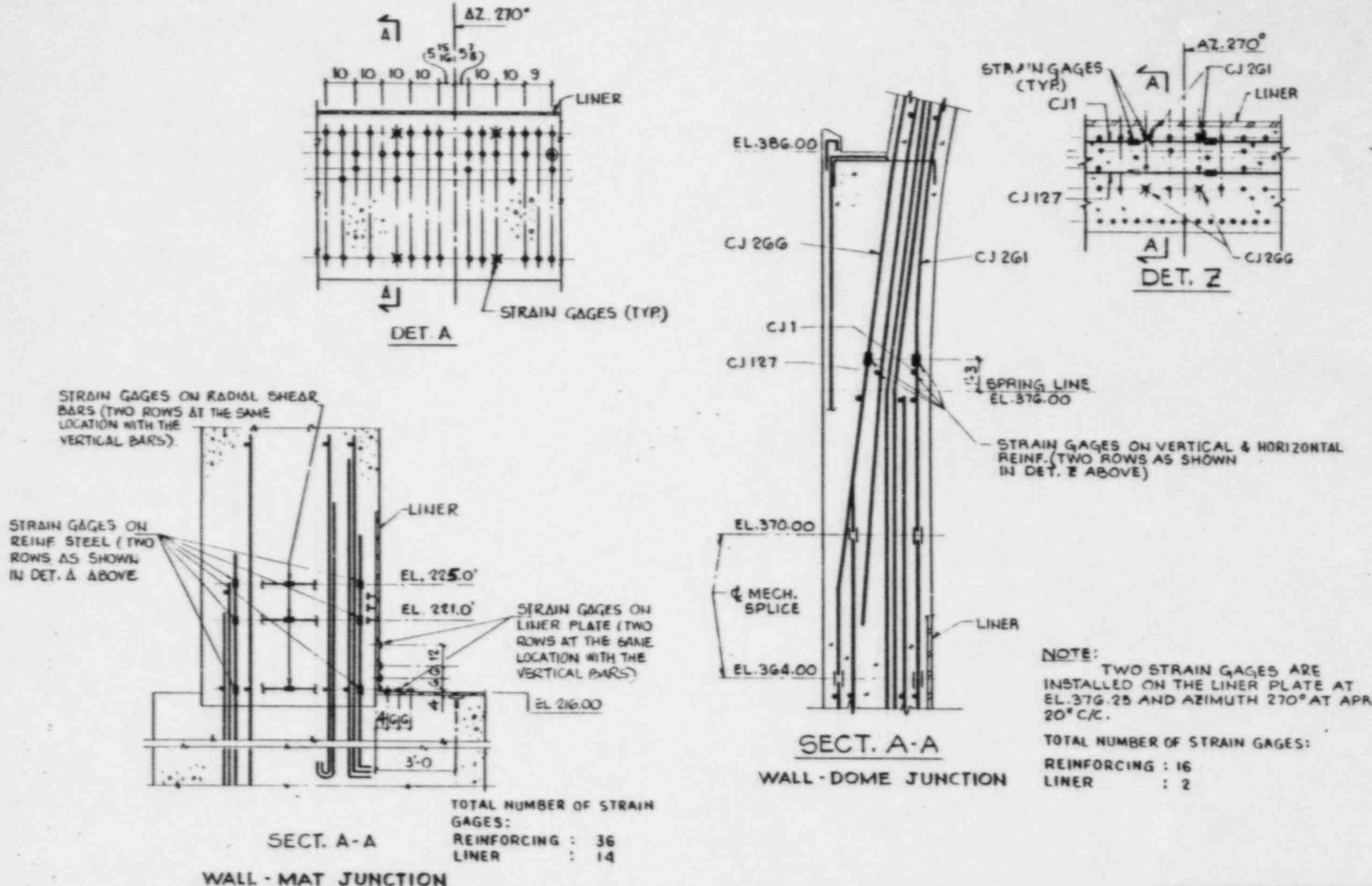
FIGURE 3.8.1-45



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE
STRUCTURAL INTEGRITY TEST – VERTICAL
DISPLACEMENT MEASUREMENT LOCATIONS

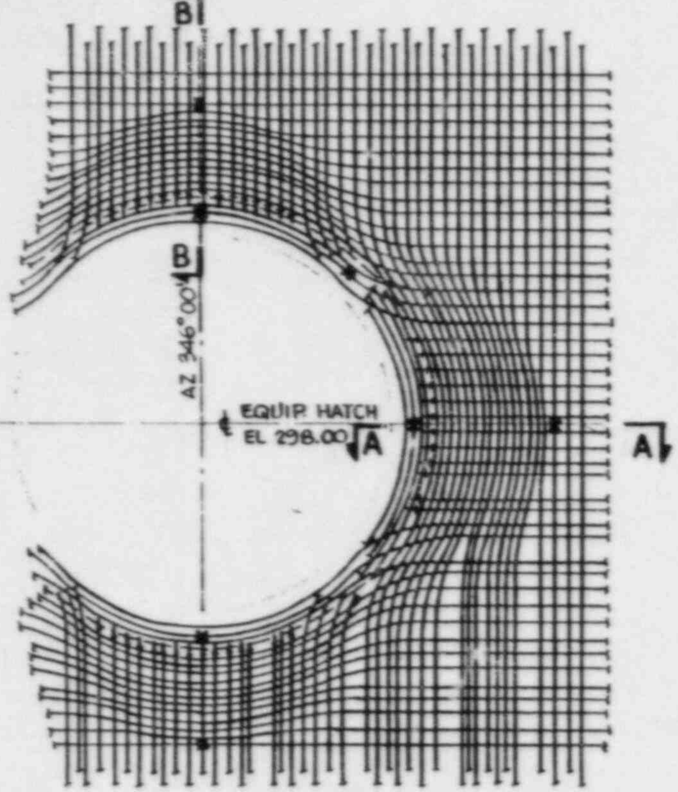
FIGURE 3.8.1-46



SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

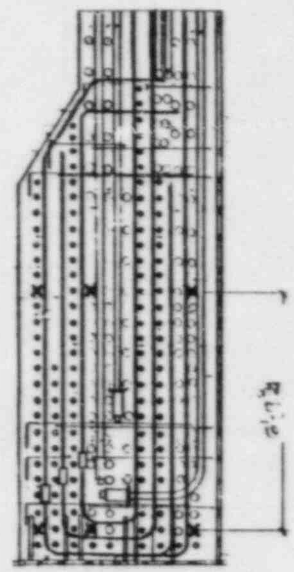
CONCRETE CONTAINMENT STRUCTURE
STRUCTURAL INTEGRITY TEST – WALL
STRAIN MEASUREMENT LOCATIONS

FIGURE 3.8.1-47



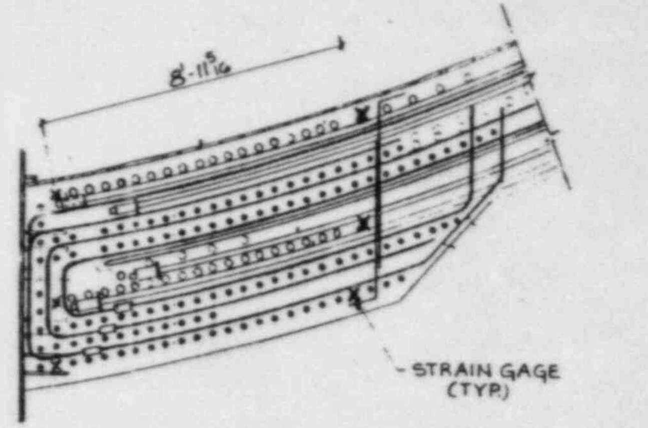
DEVELOPED ELEVATION AT EQUIPMENT HATCH

N.T.S.



SECT. B-B

N.T.S



SECT. A-A

N.T.S

NOTE:

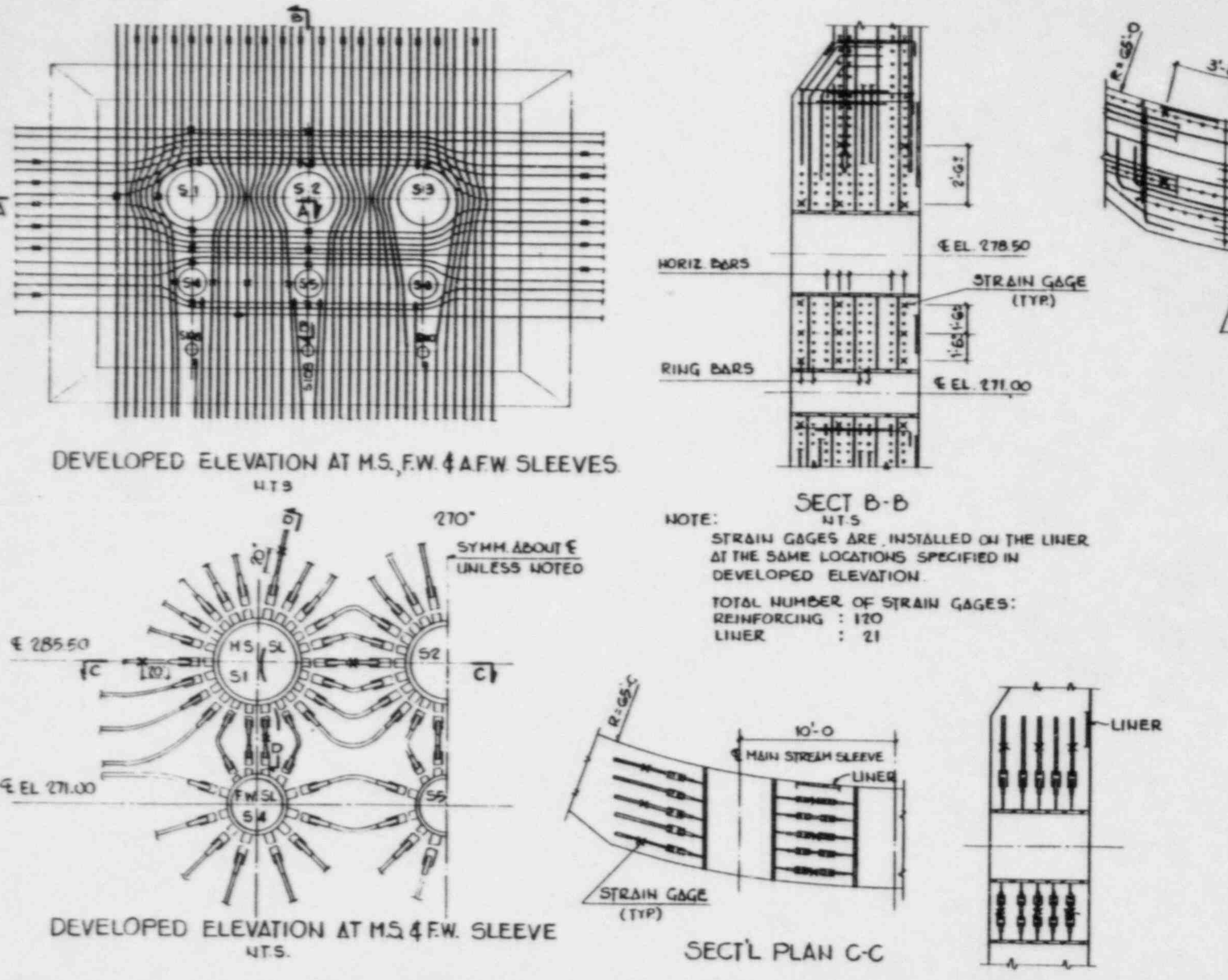
STRAIN GAGES ARE INSTALLED ON THE LINER PLATE AT THE SAME LOCATIONS SPECIFIED IN DEVELOPED ELEVATION.

TOTAL NUMBER OF STRAIN GAGES:
- REINFORCING - 42
- LINER - 7

SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT – INTEGRITY TEST – PENETRATION STRAIN MEASURE LOCATIONS

FIGURE 3.8.1-48



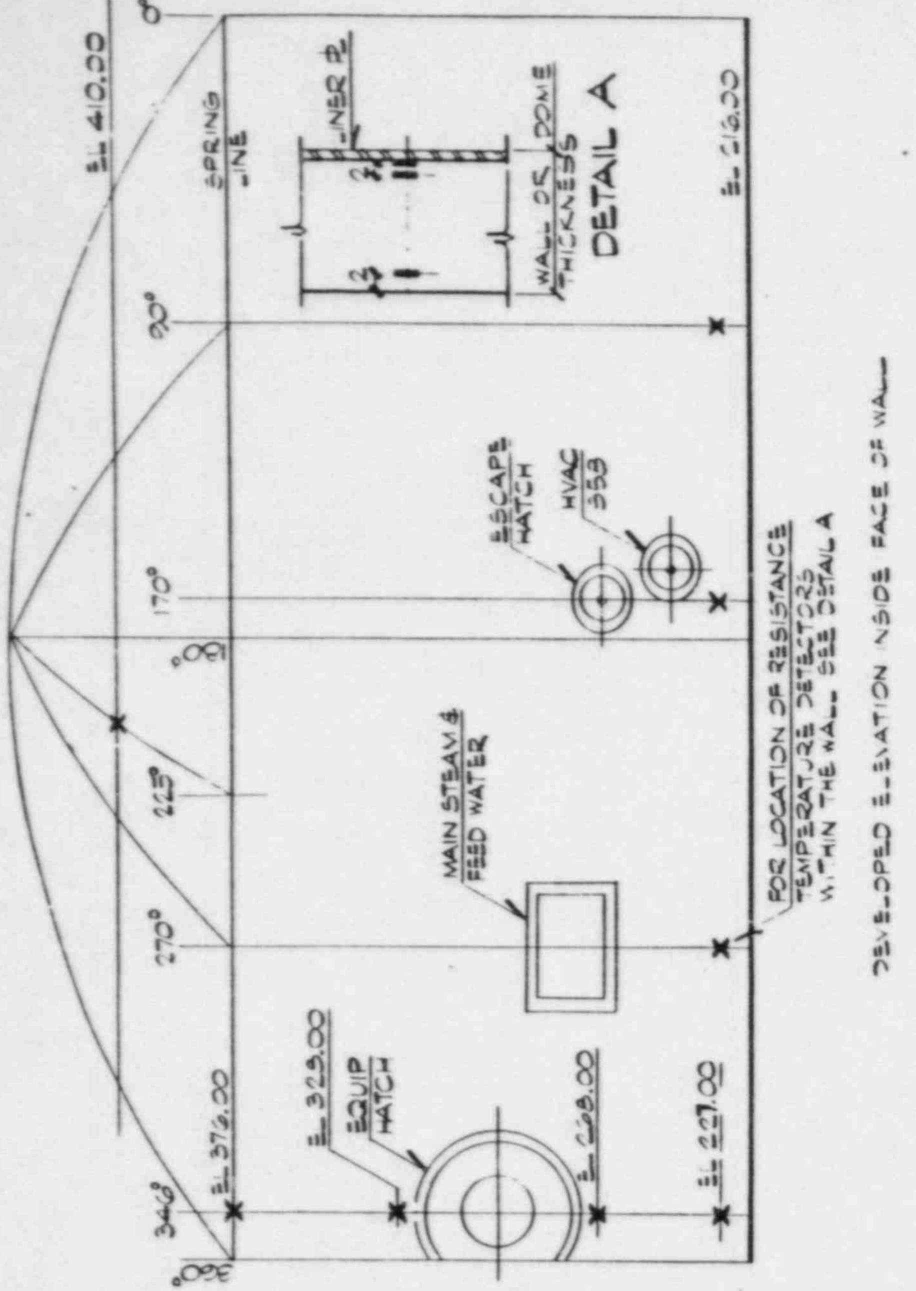
NOTE: NO. 18 REINF. BARS ARE PREPARED PRIOR TO WELDING IN PLACE; STRAIN GAGES ARE INSTALLED IN PLACE AFTER THE WELDING OF PLATES.

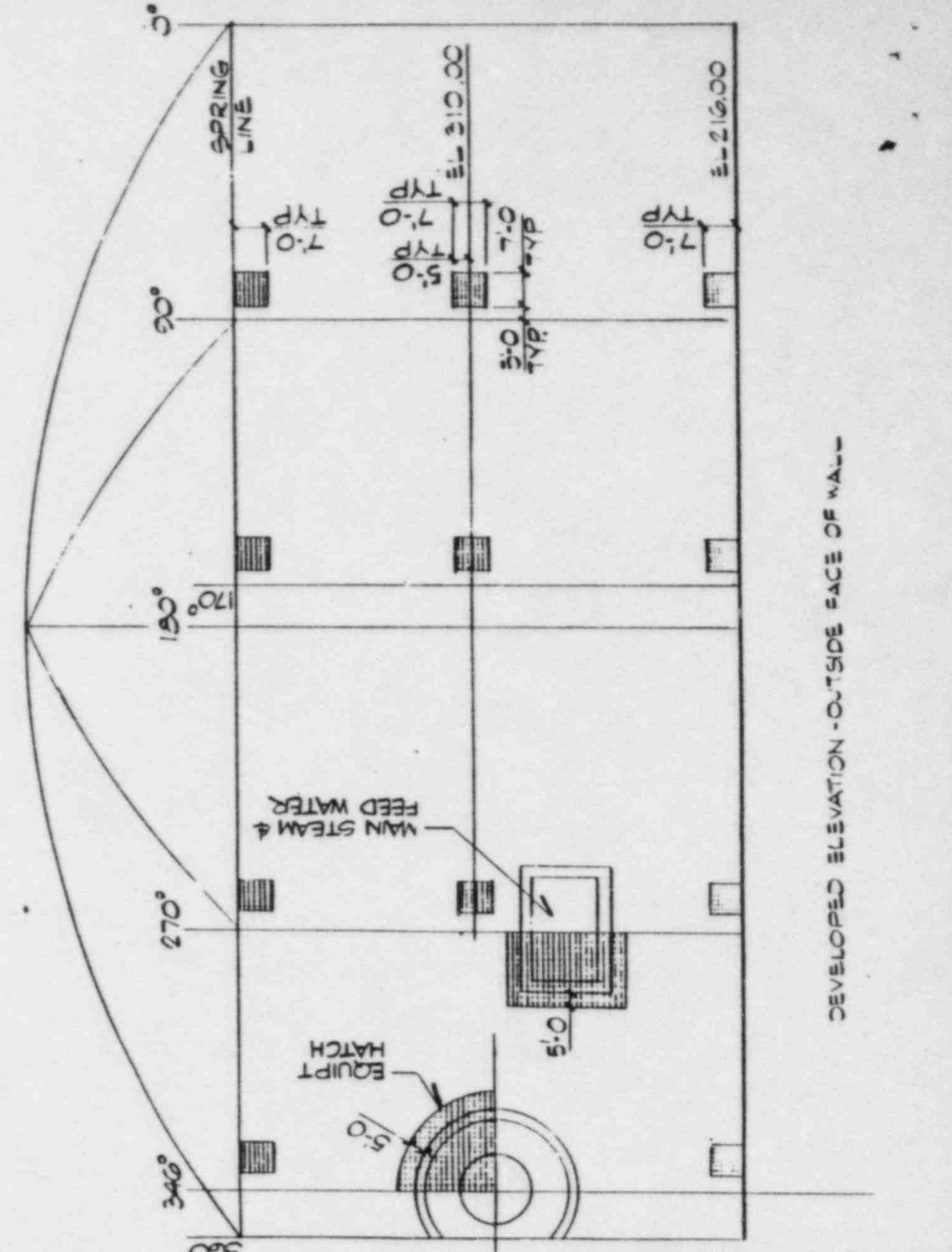
TOTAL NUMBER OF STRAIN GAGES: REINF. – 24

SHEARON HARRIS NUCLEAR POWER PLANT
Carolina Power & Light Company
FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT – INTEGRITY TEST – PENETRATION STRAIN MEASUREMENT LOCATIONS

FIGURE 3.8.1-49





SHEARON HARRIS NUCLEAR POWER PLANT

Carolina Power & Light Company

FINAL SAFETY ANALYSIS REPORT

CONCRETE CONTAINMENT STRUCTURE STRUCTURAL INTEGRITY TEST – CRACK MAPPING & TEMPERATURE MEASUREMENT LOCATIONS

FIGURE 3.8.1-50