3.8 DESIGN OF CATEGORY I STRUCTURES

3.8.1 Concrete Shield Building

The Shield Building is a Category I structure in its entirety and is designed to remain functional in the event of a Safe Shutdown Earthquake (SSE) or a tornado.

The Shield Building is designed as described in Sections 3.8.1.1 through 3.8.1.1.7. The evaluation and modification of the Shield Building reinforced concrete structure are optionally done using the ultimate strength design method in accordance with the codes, load definitions and load combinations specified in Appendix 3.8E.

3.8.1.1 Description of the Shield Building

The Shield Building, shown in Figures 3.8.1-1 through 3.8.1-7, is a reinforced concrete structure surrounding the steel containment structure and is designed to provide the following: radiation shielding from accident conditions, radiation shielding from parts of the reactor coolant system during operation, and protection of the steel containment vessel from adverse atmospheric conditions and external missiles propelled by tornado winds. The Shield Building is a reinforced concrete cylinder supported by a circular base slab and covered at the top with a spherical dome. It is located adjacent to the concrete Auxiliary and Valve Room Buildings and is physically separated from them by a 1-inch fiberglass-filled expansion joint. There is a polyvinyl chloride seal placed in formed grooves on the face of the Shield Building where it abuts the Auxiliary Building, thus providing watertightness between the two buildings up to grade level of Elevation 728.0. The seal is embedded in the groove with epoxy adhesive mortar. The Shield Building is maintained watertight to Elevation 742.0. A sectional view through the Shield Building is shown in Figure 3.8.1-1. Only the base slab resists the LOCA pressure load which is transmitted to it through a steel plate liner anchored to its top face. For further discussion of the base slab see Section 3.8.5.

The cylinder wall is approximately 150 feet in height from the top of the base slab to the spring line of the dome. It has an inside diameter of 125 feet 1 inch and a thickness of 3 feet. Conventional steel reinforcing bars were used throughout the structure and were placed in a horizontal and vertical pattern in each face of the cylinder wall. The area of reinforcement in each direction of each face is not less than 0.0015 times the gross concrete area.

The effects of penetrations through the wall were considered. Penetrations, 12 inches or less in diameter, do not significantly disturb the reinforcing pattern in the wall. Therefore, no special reinforcing considerations were made at these areas.

For penetrations larger than 12 inches, reinforcing is terminated at the opening. Supplemental reinforcing is added, both vertically and horizontally, to replace the reinforcing, terminated at rectangular penetrations larger than 12 inches and circular penetrations larger than 24 inches. The amount of supplemental reinforcing added is equal to or greater than the amount of reinforcing removed and is placed adjacent to the penetration. In addition, rectangular penetrations in the wall have diagonal reinforcing across the corners. Reinforcing bars were lap spliced in accordance with ACI 318-71 code requirements for strength design or have been cadwelded.

Reinforcing steel bars in the dome were arranged in a radial and circumferential pattern.

A ring tension beam is provided to resist the outward thrust from the dome roof. The tensile force in the ring beam is resisted by 24 No. 11 reinforcing bars. These bars are spliced with mechanical splices that are uniformly staggered at least 6 feet on center around the circumference of the ring beam. Therefore, at any cross section in a length of 6 feet, only three bars are spliced out of the total of 24 bars, and not more than two of these are in any one layer. That is, at any section, 21 bars are continuous and unspliced. These continuous, unspliced bars alone will carry the imposed load with only a 15 percent increase in stress. Stirrups enclosing the main reinforcement are spaced on 15-inch centers.

3.8.1.1.1 Equipment Hatch Doors and Sleeves

As shown in Figure 3.8.1-8, a double-leaf equipment door installed in a sleeve is provided for each Reactor Building. The steel sleeve forms an access through the Shield Building wall to the equipment hatch in the Containment Vessel. Each sleeve extends from inside the Shield Building to the shielded passageway leading to the Auxiliary Building floor Elevation 757.0. Each door is of the hinged, double-leaf, marine type with seals for providing an airtight closure between the annulus surrounding the steel containment vessel and the inside of the Auxiliary Building. A door will normally be opened only when the reactor is in the shutdown, depressurized condition such that secondary containment is not required.

The sleeves, embedded in the Shield Building walls, are of welded steel construction, rectangular in cross section, with corners fabricated to a radius. They form clear passageways 20 feet wide and 17 feet-8 inches high through the concrete walls of the Shield Buildings.

Floors in the sleeves are at Elevation 756.63 coinciding with the elevation of the operating floors in the Reactor Buildings.

The doors are hinged to the sleeves on the end toward the outside of the Shield Building wall and are of welded construction consisting of structural shapes with a steel skin plate.

The doors are opened and closed manually. Latching of the doors in the closed position is accomplished by hand-lever operated dogs acting on wedge surfaces around the perimeter and meeting edges of the door leaves.

The doors are part of the airtight closure between the annulus surrounding the Containment Vessel and the inside of the Auxiliary Building. These doors are to remain closed during unit operation and will only be opened during unit shutdown.

The door and sleeves will maintain their structural integrity and remain operational after being subjected to the environmental or accident conditions listed in Section 3.8.1.4.

3.8.1.2 Applicable Codes, Standards, and Specifications

The structural design of the reinforced concrete Shield Building is in compliance with the proposed ACI-ASME (ACI-359) Code for Concrete Reactor Vessels and Containment, Article CC-3000, as issued for trial use, April 1973, for the loading combinations defined in Table 3.8.1-1. Allowable stresses are based on this code with the exception of allowable tangential shear stresses in walls where the ACI 318-71 code is used. Detailing of reinforcing around opening of circular walls is based on the ACI Chimney Code (ACI 307-69), Sections 4.4.4 through 4.4.7. All reinforcing steel conforms to the requirements of ASTM Designation A615-72, Grade 60.

Unless otherwise indicated in the FSAR, the design and construction of the Shield Building is based upon the appropriate sections of the following codes, standards, and specifications. Modifications to these codes, standards, and specifications are made where necessary to meet the specific requirements of the structures.

Where date of edition, copyright, or addendum is specified, earlier versions of the listed documents were not used. In some instances, later revisions of the listed documents were used where design safety was not compromised.

(1) American Concrete Institute (ACI)

ACI 214-77 Recommended Practice for Evaluation of Strength Test Results of Concrete

ACI 318-71 Building Code Requirements for Reinforced Concrete

ACI 359 Code for Concrete Reactor Vessels and Containments, (Proposed ACI-ASME Code ACI-359 (Article CC-3000) As issued for trial use April, 1973)

ACI 347-68 Recommended Practice for Concrete Formwork

ACI 305-72 Recommended Practice for Hot Weather Concreting

ACI 211.1-70 Recommended Practice for Selecting Proportions for Normal Weight Concrete

ACI 307-69 Specification For the Design and Construction of Reinforced Concrete Chimneys

(2) American Institute of Steel Construction (AISC)

'Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings,' adopted February 12, 1969, except welded construction is in accordance with Item 4 below.

(3) American Society for Testing and Materials (ASTM), 1975 Annual Book of ASTM Standards. Specific standards are identified in Section 3.8.1.6.

(4) American Welding Society (AWS)

Structural Welding Code, AWS D1.1-72 with Revisions 1-73 and 2-74 except later editions may be used for prequalified joint details, base materials, and qualification of welding procedures and welders.

Visual inspection of structural welds will meet the minimum requirements of Nuclear Construction Issues Group documents NCIG-01 and NCIG-02 as specified on the design drawings or other engineering design output. See Item 12 below.

'Recommended Practice for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Connections,' AWS D12.1-61.

(5) Uniform Building Code, International Conference of Building Officials, Los Angeles, 1970 edition.

(6) Southern Standard Building Code, 1969 edition, 1971 Rev.

(7) 'Nuclear Reactors and Earthquakes,' USAEC Report TID7024, August 1963.

(8) American Society of Civil Engineers Transactions, Volume 126, Part II, Paper No. 3269, 'Wind Forces on Structures,' 1961.

(9) Code of Federal Regulations Title 29, Chapter XVII, "Occupational Safety and Health Standards," Part 1910.

(10) NRC Regulatory Guides;

RG1.10 Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures

RG1.12 Instrumentation for Earthquakes

RG 1.15 Testing of Reinforcing Bars for Category I Concrete Structures

RG 1.31 Control of Ferrite Content in Stainless Steel Weld Metal

RG 1.55 Concrete Placement in Category I Structures.

(11) Nuclear Construction Issues Group (NCIG)

NCIG-01, Revision 2 -Visual Welding Acceptance Criteria (VWAC) for Structural Welding

NCIG-02, Revision 0 - Sampling Plan for Visual Reinspection of Welds

The referenced NCIG documents may be used after June 26, 1985, for weldments that were designed and fabricated to the requirements of AISC/AWS.

NCIG-02, Revision 0, was used as the original basis for the Department of Energy (DOE) Weld Evaluation Project (WEP) EG&G Idaho, Incorporated, statistical assessment of TVA performed welding at WBNP. Any further sampling reinspections of structural welds subsequent to issuance of NCIG–02, Revision 2, are performed in accordance with NCIG-02, Revision 2 requirements.

The applicability of the NCIG documents is specified in controlled design output documents such as drawings and construction specifications. Inspectors performing visual weld examination to the criteria of NCIG-01 are trained in the subject criteria.

(12) TVA Reports

CEB 86-12 Study of Long Term Concrete Strength at Sequoyah and Watts Bar Nuclear Plants

CEB 86-19-C Concrete Quality Evaluation

3.8.1.3 Loads and Loading Combinations

The Shield Building dome and cylinder wall are subjected to the following loads. Design loading combinations utilized to examine the effects of localized areas are shown in Tables 3.8.1-1 and 3.8.1-2.

Dead Load

This includes weight of the concrete structure plus any other permanent load contributing to stress, such as equipment, piping, and cable trays suspended from the structures.

Earth Pressure

The static soil pressure was computed using Earth Pressure Standards from TVA's General Standards which incorporate Coulomb's "wedge of pressure" theory.

Standard soil properties for fine grained rolled fill are as follows:

Angle of internal friction = 32 degrees

Angle of friction between soil and building = 16 degrees

Dry weight = 120 lb/cu ft

Buoyant weight = 65 lb/cu ft

Due to adjacent structures the soil does not completely surround the Shield Building but lies in a 185 degree segment around it. The soil was backfilled to a height of 31 feet above the base slab. A surcharge of 200 psf was used.

Hydrostatic Pressure

Uplift forces and lateral static pressure were computed using the full hydrostatic head measured from the water surface. Water surface elevations from the probable maximum flood (Section 2.4) were used in determining hydrostatic heads.

Due to water seals between the Shield Building and adjacent structures, the lateral hydrostatic pressure was applied only to one-half of the circumference for the drawn down ground water table. For the probable maximum flood the adjacent structures are allowed to flood and lateral hydrostatic pressure was applied around the full circumference.

Loss of Coolant Accident (LOCA)

In addition to the reactions of the containment vessel and interior concrete due to the LOCA pressure transients, the LOCA produced uplift forces on the steam generator or reactor coolant pump anchors in the base slab. The LOCA also increased the temperature in the annulus space between the Containment Vessel and the Shield Building. This produced a nonlinear temperature gradient across the cylinder wall and dome. A typical gradient in shown in Figure 3.8.1-9.

Normal Temperature Gradient

The temperature gradient for normal plant operation was considered as uniformly varying through the section. The maximum temperature gradient occurs just above grade when the plant is in operation and a minimum ambient temperature exists. The normal temperature difference across the wall varies from a minimum of 35°F below grade to the maximum of 85°F as shown on Figure 3.8.1-9.

Operational Basis Earthquake (OBE)

The plant was designed to remain operational under the OBE. The OBE has a maximum acceleration of 0.09g horizontally and 0.06g vertically. In addition to the maximum values of the structural response in terms of displacement, acceleration, shear, moment, torque and axial force, the soil pressure and hydrostatic pressures were increased due to seismic motions. The static soil-pressure was increased 23% for a dry fill and 11% for a saturated fill. This incremental increase was a triangle of pressure with the apex at the rock surface and the maximum ordinate at the ground surface. The hydrostatic pressure of the water within the fill was increased by 11%.

This incremental increase was a triangle of pressure with the apex at the water surface and maximum ordinate at the rock surface. The magnitude of these increases were determined by shaking table experiments performed for another TVA project. The reaction from earthquake motion on the compressed expansion joint material separating the adjacent Auxiliary and Valve Room Buildings was also taken into consideration.

Safe Shutdown Earthquake (SSE)

The plant was designed to have the capability for safe shutdown for the SSE (maximum acceleration of 0.18g horizontally and 0.12g vertically). The incremental pressure increase for soil and hydrostatic pressure was twice that for the OBE.

Live Load

Live load includes non pipe hanger loads, plus any other permanent load such as crane loads, etc. Snowload of 20 psf was considered in the design live load.

Tornado

The tornado was assumed to have an "eye" whose pressure is 3 psi below ambient, a "funnel" having a rotational velocity of 300 mph, and a translational speed of 60 mph. The Shield Building was designed for wind loads corresponding to 360 mph and a maximum internal pressure of 3 psi. Maximum wind velocity and maximum internal pressure loadings do not coincide as shown by Figure 3.3-1. The ultimate capacity of the structure in flexure or shear is not exceeded under the combined pressure and wind velocity loadings of Figure 3.3-1.

The adjacent structures disturb the air flow around the Shield Building. The only method to determine the actual pressure distribution on the structure is by a model test. In lieu of model test, several cases of extreme pressure distributions were analyzed in an attempt to bracket the actual stresses. The normal maximum wind loading was based on Figure 1(b), from ASCE Paper 3269, "Wind Forces on Structures."

Tornado missiles are described in Section 3.5.

Construction Loads

The dome was poured in two lifts. The first lift is a 9-inch pour supported by temporary shoring bearing on the Containment Vessel. The first lift was designed to support the wet concrete dead load of the second lift plus a construction load of 50 psf.

3.8.1.4 Design and Analysis Procedures

Base Slab

The base slab is discussed in Section 3.8.5.

Cylinder Wall and Dome

The stiffness of the cylinder wall was small in comparison to that of the base slab and the cylinder wall was assumed fixed at the base. The height of the wall was such that the effect of discontinuity at one end was negligible when considering discontinuity at the other end.

For symmetrical loadings, the edge forces at the point of discontinuity were determined by writing the equations of the primary system and the equation of compatibility. The discontinuity stresses from the edge forces were superimposed on the membrane stresses. The above analysis was checked by two independent computer analyses ("Axisymmetric Finite Element Analysis, AMG032" and GENSHL 2). Unsymmetrical loadings, such as wind, were analyzed by using computer code, GENSHL. These loads were approximated through a Fourier series.

Creep and Shrinkage Effects

Creep was not considered in the design of the Shield Building. Sustained loads are essentially the dead weight loads of the structure itself with subsequent stress levels too low to influence creep deformations to any significant degree particularly since these deformations do not cause differential settlements in the structure.

Shrinkage effects are considered in the design of all structures by estimating the temperature change from peak hydration temperatures to final operating temperature conditions. In addition drying shrinkage effects are considered in all members which have an average drying path of less than 15 inches. The methods used to consider these effects are explained in an ACI Committee 207 Report 70-45, "Effect of Restraint, Volume Change, and Reinforcement on Cracking of Massive Concrete" published in July 1973.

The effects of base restraint on the cracking of a circular structure is essentially the same as the effects on a wall of equal thickness whose length is equal to the outside diameter of the circular structure.

The Shield Building was not only designed to restrict shrinkage cracking, thus holding the cracks to a minimum acceptable size, but was also waterproofed on the exterior surface below grade to eliminate possible seepage. The portion above grade is essentially out of the restraint zone and will therefore be relatively free from shrinkage cracking.

Tangential Shear

The tangential shears induced by earthquake and wind forces were assumed to vary from zero over a thickness of wall located at the extremes of a diameter parallel to the line of action of the shearing force to a maximum on a wall thickness located at the extremes of a diameter normal to the line of action of the shearing force. Distribution was assumed proportional to the cosine of the polar angle measured from the diameter normal to the line of action of the shear force with a maximum allowable shear stress in the concrete limited to 247 psi according to special provisions for shear in walls in the ACI 318-71 code.

Seismic

See Section 3.7 for a detailed description of the seismic analysis.

Equipment Hatch Doors and Sleeves

For the closed position, the structural members of the door leaves were designed as simple beams under uniformly distributed loading with the end reactions carried by the sleeve. Loads at the dogging wedges were carried to the sleeve as concentrated loads.

For the open position, the door leaves were treated as cantilever structures, and the hinge members and sleeve were designed for the resulting concentrated loads.

Design of the doors and sleeves was by TVA without the use of a computer program.

Under normal operating conditions, air pressure equal to 5 inches of water is exerted on the Auxiliary Building side of the doors. Under accident or tornado conditions, the doors are subjected to air pressure. Environmental and accident conditions which were considered in the design of the doors and sleeves are as follows:

(1) The OBE and the SSE with accelerations as hereinafter defined.

(2) An inadvertent release of the cooling sprays in the Containment Vessel will cause a pressure drop within the annulus surrounding it and result in an air pressure load of 2 psi on the Auxiliary Building side of the doors and sleeves. Duration of this condition will be for a few hours maximum.

(3) A tornado condition which causes a pressure drop within the Auxiliary Building will result in a pressure of 3 psi on the annulus side of the doors. Duration will be for 3 seconds.

(4) A LOCA accident in the Containment Vessel which will result in a pressure equal to 3/4 inch of water on the Auxiliary Building side of the doors. A partial vacuum is created in the annulus by vacuum pumps, and this condition may exist for a period of several months.

Earthquake accelerations used in design of the doors and sleeves were determined by dynamic analysis of the supporting structure of the Shield Building. Accelerations at the centerline of the equipment hatch for the OBE are as follows:

Lateral (north-south)	0.16g
Lateral (east-west)	0.16g
Vertical	0.12g

Accelerations at the centerline of equipment hatch for a SSE are as follows:

Lateral (north-south)	0.36g

Lateral (east-west)	0.36g
Vertical	0.23g

These accelerations were used as static loads for determining component and member sizes. After establishing the component and member sizes, a dynamic analysis, using appropriate response spectrum, was made of each sleeve and its doors to determine that allowable stresses had not been exceeded.

3.8.1.5 Structural Acceptance Criteria

Controlling Conditions Shield Building Structure

The SSE in combination with a LOCA (load combination 8) produced the largest overturning moment. For this combination, the percent of the base slab in compression was 51% and the factor of safety for overturning was 1.74.

The uplift on the equipment from the LOCA combined with the SSE controlled the design of the base slab.

Minimum steel requirements of 0.65 square inches per foot (minimum steel ratio of 0.0015 in each face and in both vertical and horizontal directions) controlled the inside face vertical steel requirements throughout the shell and the inside face horizontal steel requirements above grade.

The SSE in load combination 8 controlled the design of the outside face vertical reinforcement at the base of the cylinder wall. Due to earth and hydrostatic pressure, outside face horizontal reinforcement requirements were greatest 16 feet above the base of the cylinder wall at elevation 713.0.

The construction loading controlled the reinforcement design in the dome and the upper portion of the cylinder wall.

The SSE produced a maximum tangential shear stress at the base of the wall of 189.7 psi which was 76.8% of the allowable.

The effects of repeated reactor shutdowns and startups during the plant's life will not degrade the above margins of safety because the Shield Building is minimally affected by these operations. The only effects from normal operations are from interior temperature changes which are insignificant compared to normal exterior temperature variations.

Equipment Hatch Doors and Sleeves

Allowable stresses for all load combinations used for the various parts are given in Table 3.8.1-2. For normal load conditions, the allowable stresses provide safety factors of 1.67 (F_y/0.6 F_y) to 1 on yield for structural parts and 5 to 1 on ultimate for mechanical parts. For a limiting condition such as a Safe Shutdown Earthquake (SSE), stresses do not exceed 0.9 yield.

3.8.1.6 Materials, Quality Control and Special Construction Techniques

General

The principal materials used in the construction of the Shield Building base slab, wall, and dome were concrete and reinforcing steel. Steel is used for the structural parts of the equipment hatch doors and sleeves with rubber used for the seals.

3.8.1.6.1 Materials

Concrete

Cement conformed to ASTM Specification C150-72 Type I. The guaranteed 28 day mortar strength was 5025 psi with a guaranteed standard deviation of 395 psi and a guaranteed maximum tricalcium aluminate content of 9.5%.

Aggregates conformed to ASTM Specification C-33-71a and were manufactured of crushed limestone.

Water for mixing concrete and also for washing the aggregates and curing concrete was tested prior to use in accordance with Corps of Engineers test method CRD-C400.

The fly ash used at Watts Bar is in general accordance with the ASTM C618-73, except for the loss of ignition and fineness of pozolanic index parameters. TVA specific requirements for loss of ignition are more restrictive while the fineness pozolanic index is less restricitve than the ASTM requirements. (See Section 3.8.3.2.1.a for more details). Sampling and testing was performed in accordance with ASTM C 311.

Air-entraining admixtures conformed to ASTM Specification C-260-69.

Water-reducing agent used for concrete mixtures containing fly ash was selected based on demonstrated achievement of TVA specified concrete strength of a control mix by actual testing.

Reinforcing Steel

Reinforcing steel conformed to ASTM Designation A615-72, Grade 60.

Equipment Hatch Sleeves and Doors

The structural parts of the sleeves and doors are fabricated from ASTM A36 steel.

3.8.1.6.2 Quality Control

Concrete

Concrete was produced in a central batch and mixing plant until 1977, and central batch and transit mix after 1977. A materials engineering unit was specifically responsible for control, documentation, and daily review of test data.

Aggregate gradation and deleterious material was checked daily. All coarse aggregate was rinsed and resized. The gradation of the fine aggregate and the amount finer than the No. 200 sieve conformed to specifications.

The other concrete material was also subject to periodic tests (see Section 3.8.3.2).

The specified strength of the concrete was 4000 psi at 28 days. Some concrete did not meet specification requirements. This was evaluated and documented in the Report CEB-86-19-C "Concrete Quality Evaluation." The results have been documented in affected calculation packages and drawings.

A testing program conducted at the site compared strengths of cylinders and concrete from 3-foot-thick wall sections subjected to exterior exposures. The results of this test program are documented in TVA report CEB 86-12, "Study of Long-Term Concrete Strength at Sequoyah and Watts Bar Nuclear Plants." These tests demonstrated the long term compressive strength gain with age which have occurred. The strength gain and age was generally 2600 psi beyond 28 days and 1300 psi beyond 90 days.

Reinforcing Steel

Testing of reinforcing steel conformed to Regulatory Guide 1.15.

Cadweld splices conformed to Regulatory Guide 1.10.

Equipment Hatch Doors and Sleeves

Design by TVA and erection by TVA were in accordance with TVA's quality assurance program. Design and fabrication by the contractor were in accordance with the contractor's quality assurance program which was reviewed and approved by TVA's design engineers. The contractor's quality assurance program covers the criteria in Appendix B of 10 CFR 50. Fabrication procedures such as welding and nondestructive testing were included in appendices to the contractor's quality assurance program. ASTM standards were used for all material specifications and certified mill test reports were provided by the contractor for materials used for all load carrying members.

Material used for seals including O-rings, was certified by a rubber technologist as being capable of withstanding the radiation and temperature conditions existing during a LOCA accident. This certification is based on testing and evaluation of seal materials performed under contract for TVA by Presray Corporation.

3.8.1.6.3 Construction Techniques (Historical Information)

The walls of the Shield Building from the base slab to the bottom of the ring beam were constructed using conventional forms. The concrete pouring was performed in two stages to facilitate other construction work in the building. The first stage consisted of concrete pours to elevation 762.0 and the second stage consisted of the remaining height of wall. Concrete temperatures were monitored throughout for a minimum period of 3 days during cold weather to assure cold weather protection requirements.

The dome roof was placed in two lifts with each lift divided into three basic rings and each ring divided into radial segments. The Steel Containment Vessel (SCV) is designed to support the formwork for the first 9-inch-thick lift and the first lift is then designed to support the remaining 15-inch lift with the formwork removed. Delays are specified between adjacent lift pours in order to minimize the effects of initial volume changes. The second lift was not placed until the first lift had attained its specified strength.

The base slab, ring beam, and parapet wall were constructed using conventional methods.

3.8.1.7 Testing and Inservice Surveillance Requirements

Since the Shield Building is not a pressure containment its wall and dome will not be pressure tested.

References

None

Table 3.8.1-1
(Sheet 1 of 2)
Loading Combinations, Load Factors And Allowable Stresses For The Shield Building Concrete Exterior Cylindrical Wall, Dome And Base Slab

		COMBINATIONS[3]										
Loading		**1**	**2**	**2a**	**2b**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
D Dead Load		1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
D Earth Pressure		1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Pmf Probable Maximum Flood												1.0
To Normal Operating Temperature		1.0	1.0	1.0	1.0	1.0	1.0					
Ta Accident Temperature								1.0	1.0	1.0	1.0	
Pa Accident Pressure								1.5	1.25	1.25	1.0	
Fegs Safe Shutdown Earthquake							1.0				1.0	
Fego Operational Basis Earthquake			1.0		1.0				1.25			
W Normal Wind				1.0	1.0					1.25		1.0
Wt Tornado[2]						1.0						
L Live Load		1.0	1.0			1.0	1.0	1.0	1.0	1.0	1.0	1.0
Cc Construction Condition		1.0										
Pv Negative Internal Pressure			1.0	1.0			1.0					
Yjyr Pipe Break Jet And Reaction Load									1.0	1.0	1.0	
Allowable Stresses*	fc	.45fc'	.45fc'	.45fc'	.45fc'	.75fc'	.75fc'	.75fc'	.75fc'	.75fc'	.75fc'	.75fc'
	fs	.5 fy[1]	.5fy[1]	.5 fy[1]	.5fy[1]	.9 fy	.9 fy	.9 fy	.9 fy	.9 fy	.9 fy	.9 fy

*fc' = Specified Strength Of Concrete
fc = Allowable Flexural Concrete Stress
fy = Yield Strength Of Reinforcing Steel
fs = Allowable Reinforcing Steel Stress

Table 3.8.1-1
(Sheet 2 of 2)
Loading Combinations, Load Factors And Allowable Stresses For The Shield Building Concrete Exterior Cylindrical Wall, Dome And Base Slab

Footnotes:

(1) Reinforcing Steel Stresses May Be Increased By 33% When Temperature Effects Are Combined Provided The Required Section Is Not Reduced From That Required Without The Temperature Effects

(2) W_t Includes Tornado Wind, Tornado Positive Internal Pressure, And Tornado Generated Missiles.

(3) Loading Combinations (Compared To Table Cc-3200-1 Of ACI-359, 1973)

1. Service - Construction
2. Service - Normal
3. Factored - Extreme
4. Factored - Environmental
5. Factored - Abnormal
6. Factored - Abnormal/severe Environmental
7. Factored - Abnormal/severe Environmental
8. Factored - Abnormal/extreme Environmental
9. Factored - Extra Case

The following loads from Table CC-3200-1 of ACI-359, 1973, as issued for trial use, are not applicable to the Shield Building exterior wall and dome.

(F, Pt, Tt, Ro, Ra, Yr, Yj, Ym, Pa, Ta) = 0

The Structural Integrity Test ($D + L + P_t + T_t$) from the ACI-359, 1973 is not a controlling load case for the base slab.

Table 3.8.1-2 Shield Building Equipment Hatch Doors And Sleeve Loads, Loading Combinations, And Allowable Stresses

Structural

No.	Load Combinations	Allowable Stresses (psi) Tension	Compression****	Shear
I	Dead load plus 2-psi pressure	0.50 F_y	0.47 F_y	0.33 F_y
II	Dead load plus 3-psi pressure inside	0.90 F_y	0.90 F_y	0.60 F_y
III	Dead load plus 2-psi pressure outside plus *OBE	0.60 F_y	0.60 F_y	0.40 F_y
IV	Dead load plus 2-psi pressure outside plus *SSE	0.90 F_y	0.90 F_y	0.60 F_y
**V	Dead load plus *OBE	0.60 F_y	0.60 F_y	0.40 F_y
**VI	Dead load plus *SSE	0.90 F_y	0.90 F_y	0.60 F_y

Mechanical

No.	Load Combinations	Allowable Stresses (psi) Tension & Compression(****)	Shear
**I	Dead load	$\frac{\text{Ult}}{5}$	$\frac{2 \times \text{Ult}}{15}$
***Ia	Dead load plus* OBE	0.60 F_y	0.40 F_y
***II	Dead load plus *SSE	0.90 F_y	0.60 F_y
III	Dead load plus 2-psi pressure outside	$\frac{\text{Ult}}{5}$	$\frac{2 \times \text{Ult}}{15}$
IV	Dead load plus 3-psi pressure inside	0.90 F_y	0.60 F_y
V	Dead load plus 2-psi pressure outside plus *OBE	0.60 F_y	0.40 F_y
VI	Dead load plus 2-psi pressure outside plus *SSE	0.90 F_y	0.60 F_y

* Acts in one horizontal direction only at any given time and acts in the vertical and horizontal directions simultaneously.

** Door open.

*** For hinges only with doors open.

**** The value indicated for allowable compression stress is the maximum value permitted when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_y when buckling controls.

Table 3.8.1-2 Shield Building Equipment Hatch Doors And Sleeve Loads, Loading Combinations, And Allowable Stresses

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^{\ 2}}\right] \text{ when} \frac{Kl}{r} \le C_c$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when} \frac{Kl}{r} > C_c$$

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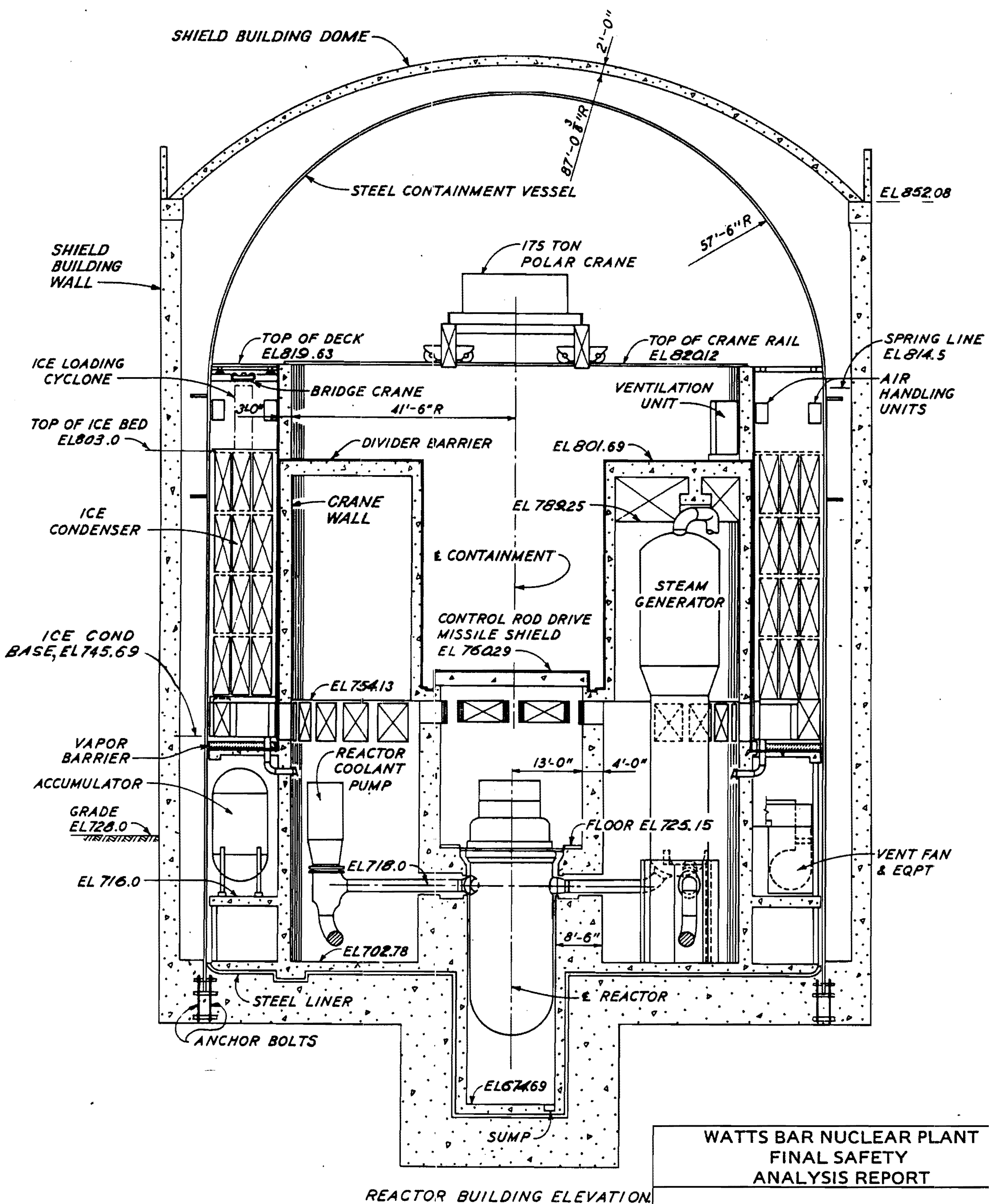


WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

REACTOR BUILDING ELEVATION

Figure 3.8.1-1

Figure 3.8.1-1 Reactor Building Elevation

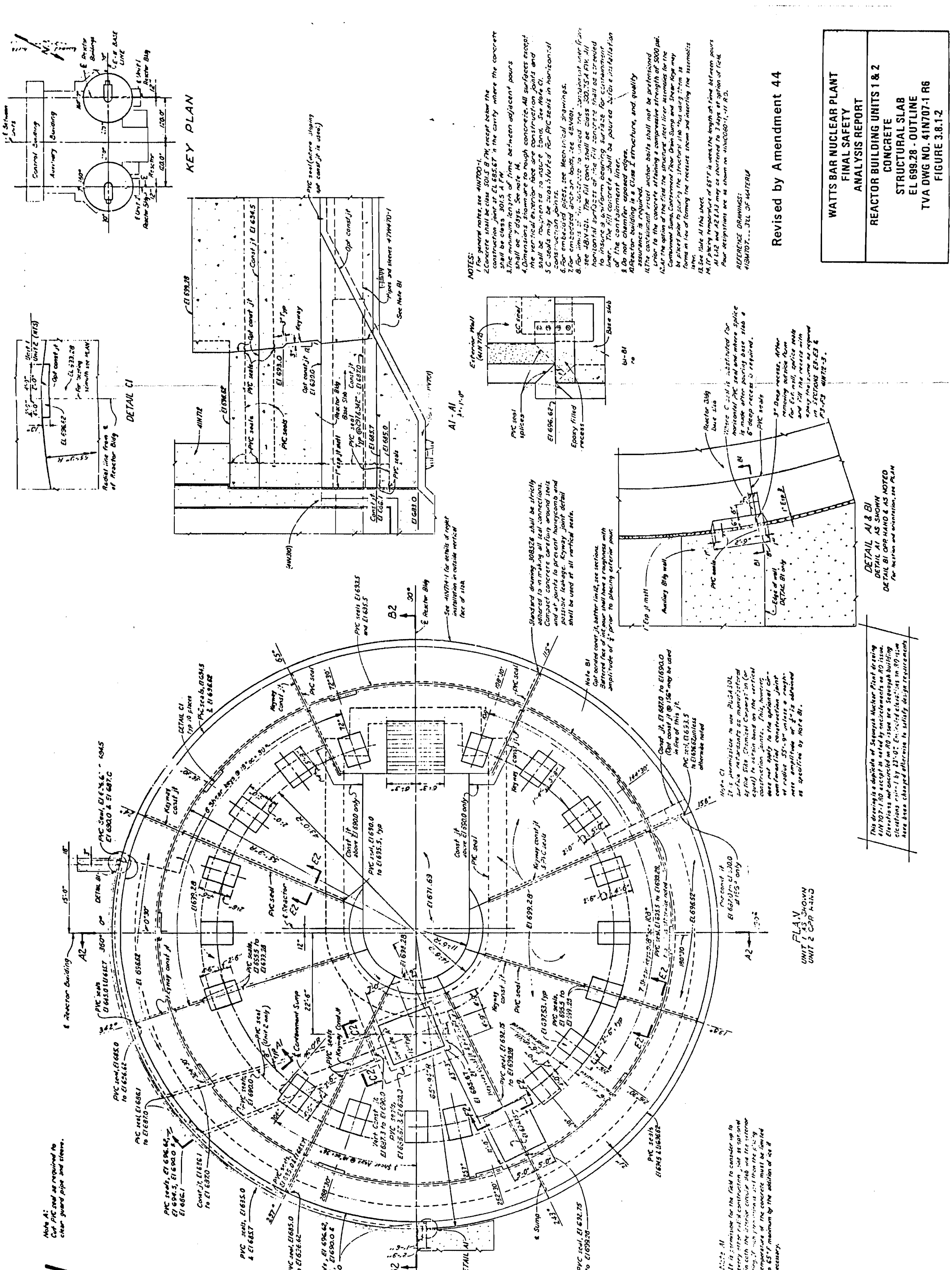


Figure 3.8.1-2 Reactor Building Units 1 and 2 Concrete Structural Slab Elevation 699.28 - Outline

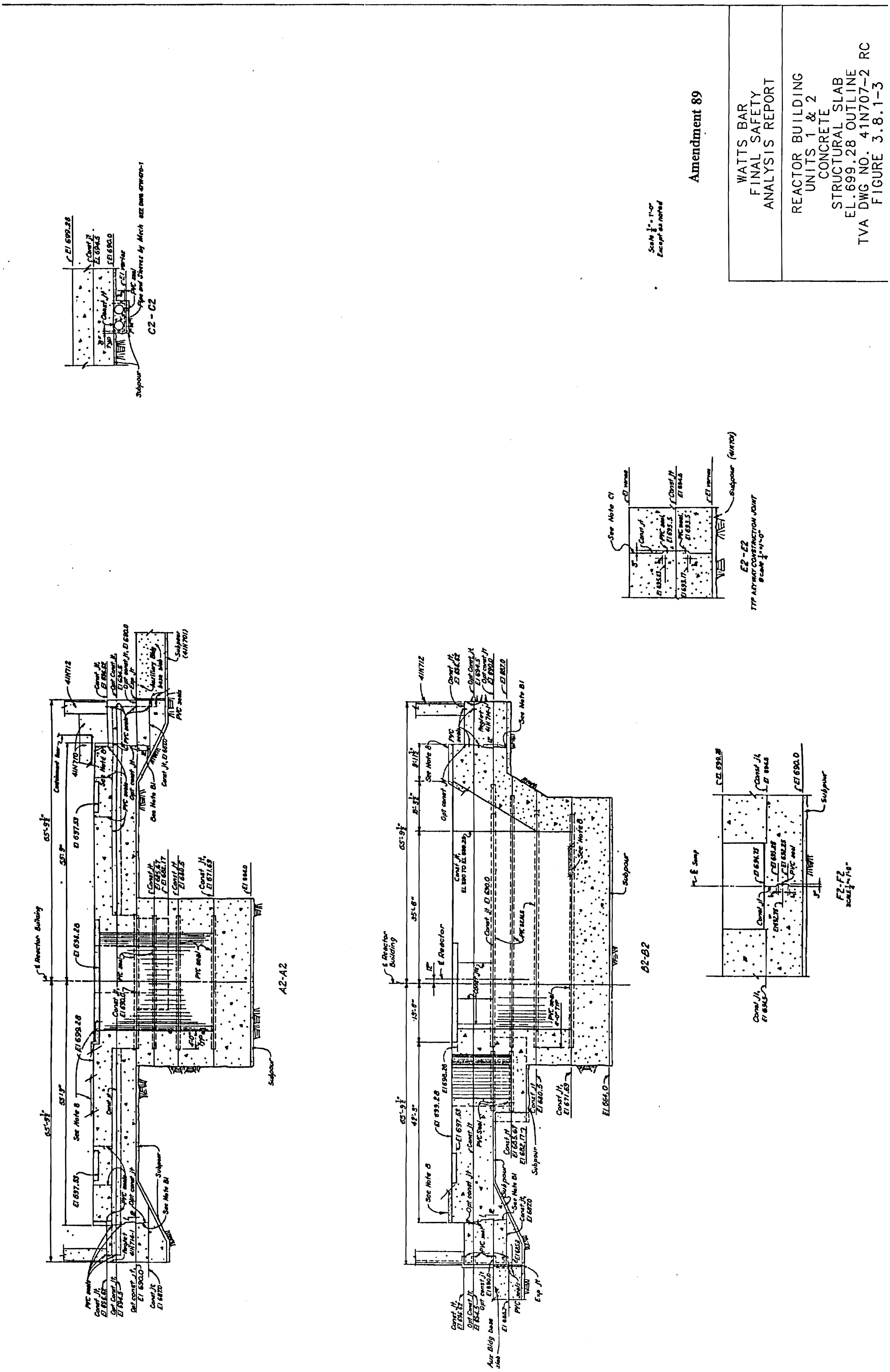


Figure 3.8.1-3 Reactor Building Units 1 and 2 Concrete Structural Slab Elevation 699.28 Outline

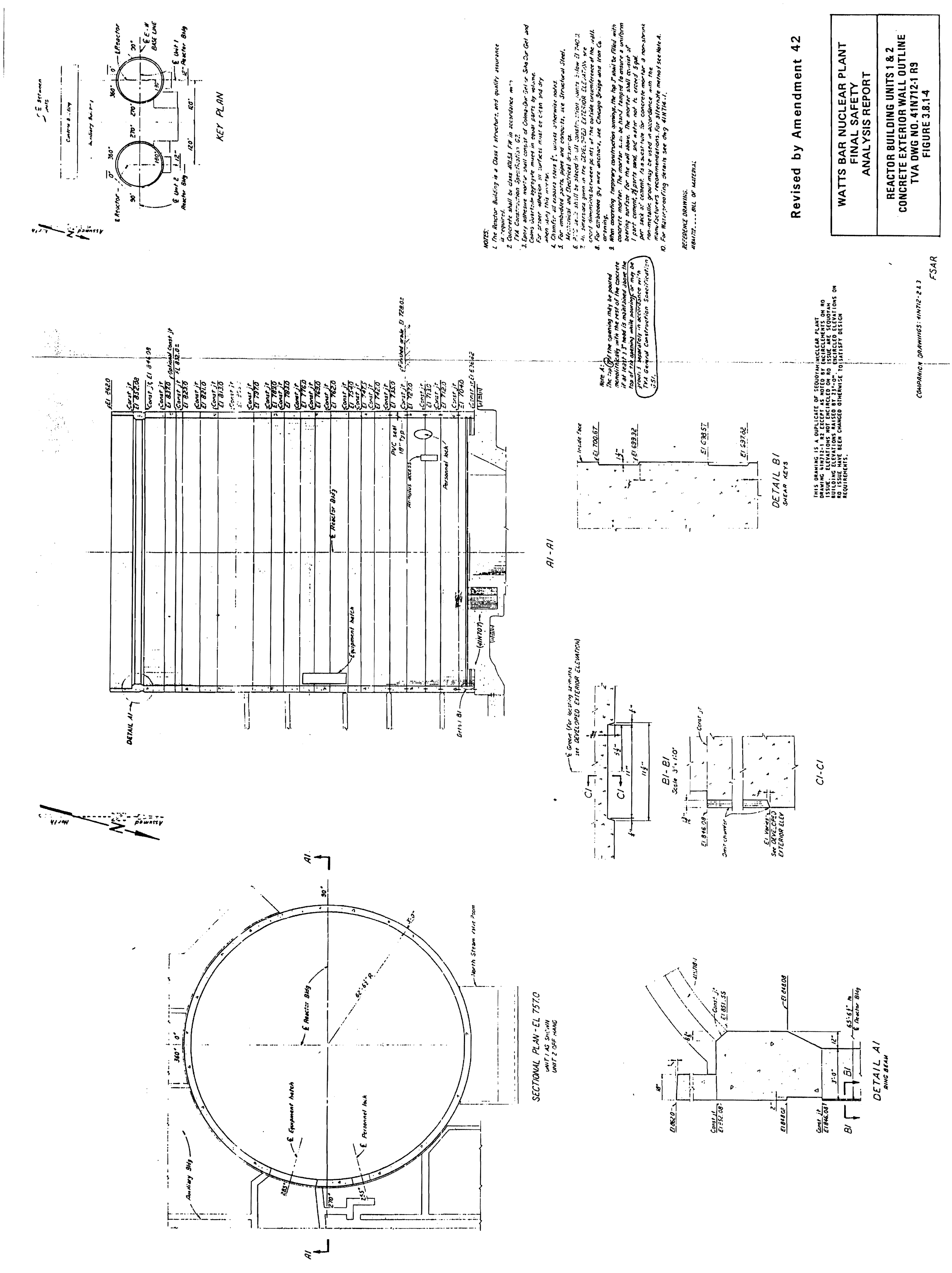


Figure 3.8.1-4 Reactor Building Units 1 and 2 Concrete Exterior Wall Outline

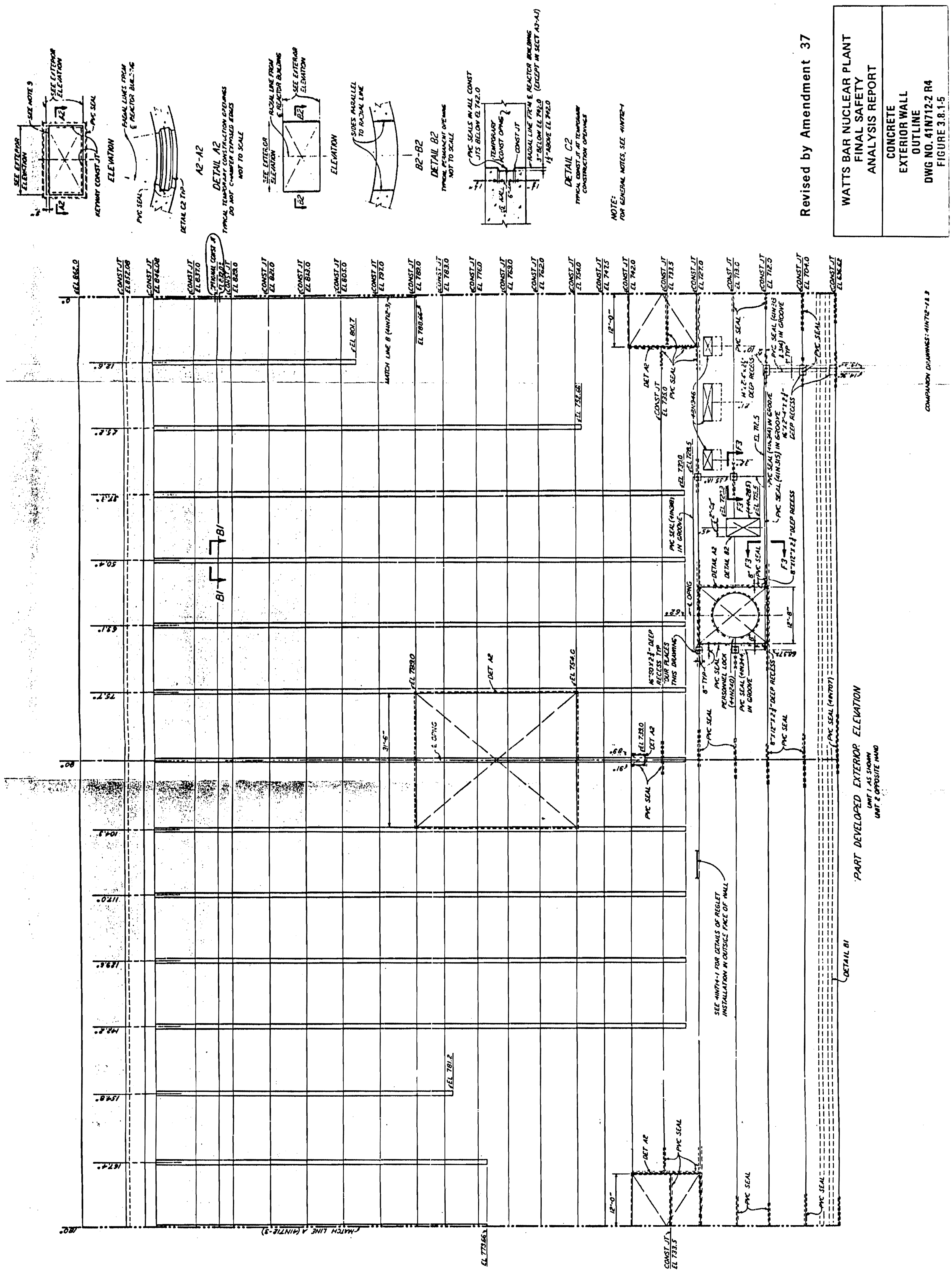


Figure 3.8.1-5 Concrete Exterior Wall Outline

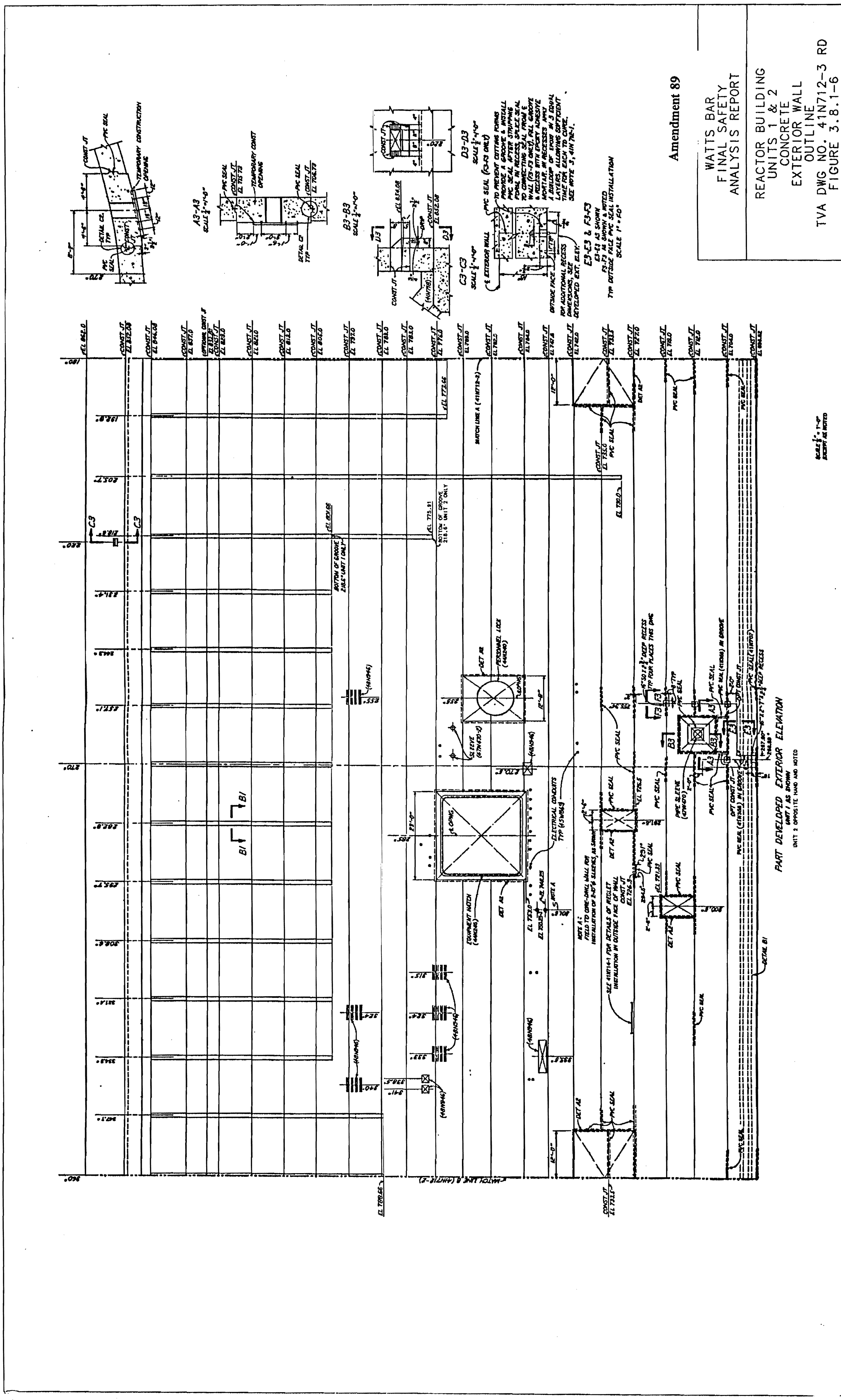


Figure 3.8.1-6 Reactor Buildings Unit 1 and 2 Concrete Exterior Wall Outline

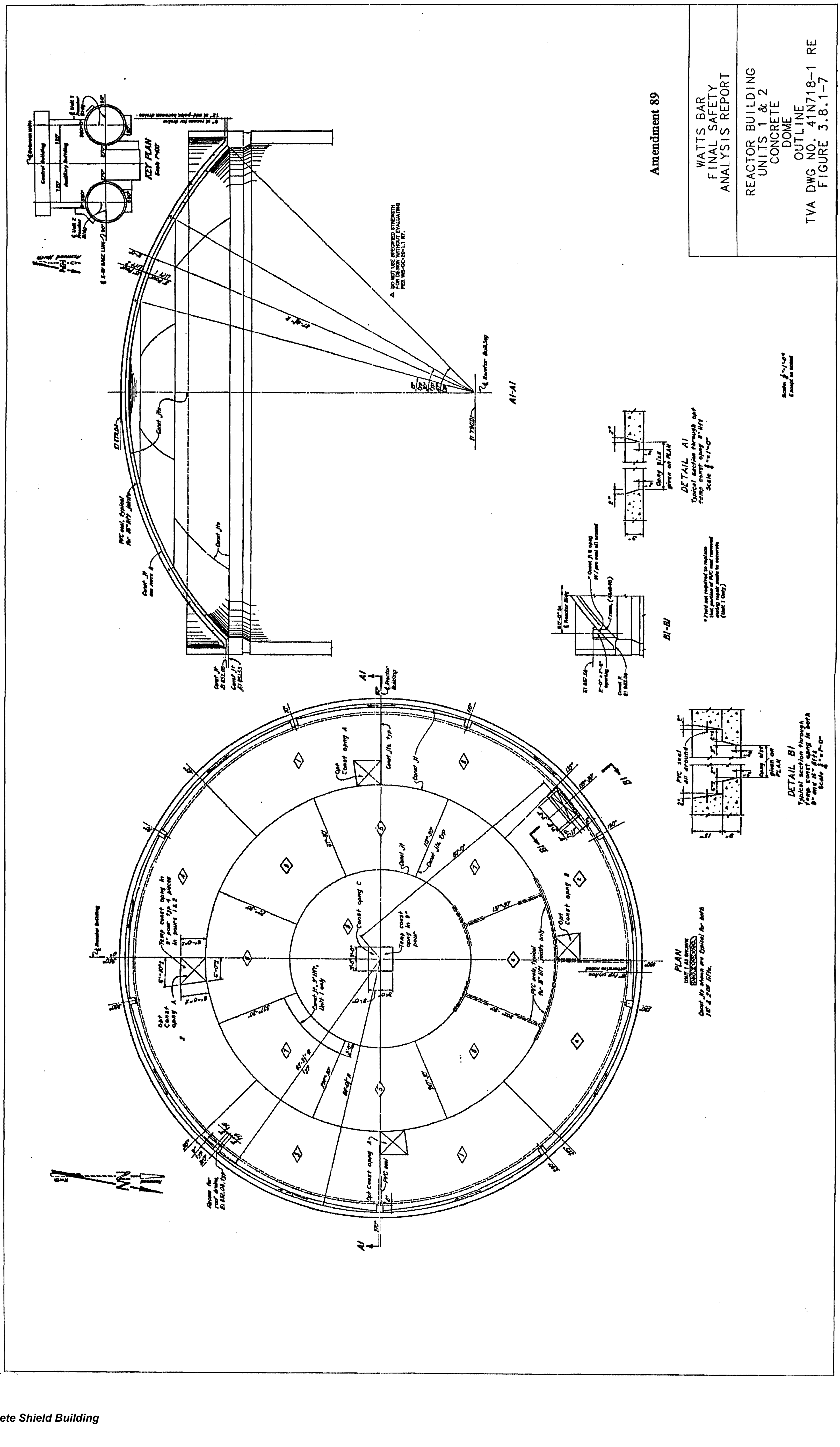


Figure 3.8.1-7 Reactor Buildings Unit 1 and 2 Concrete Dome Outline

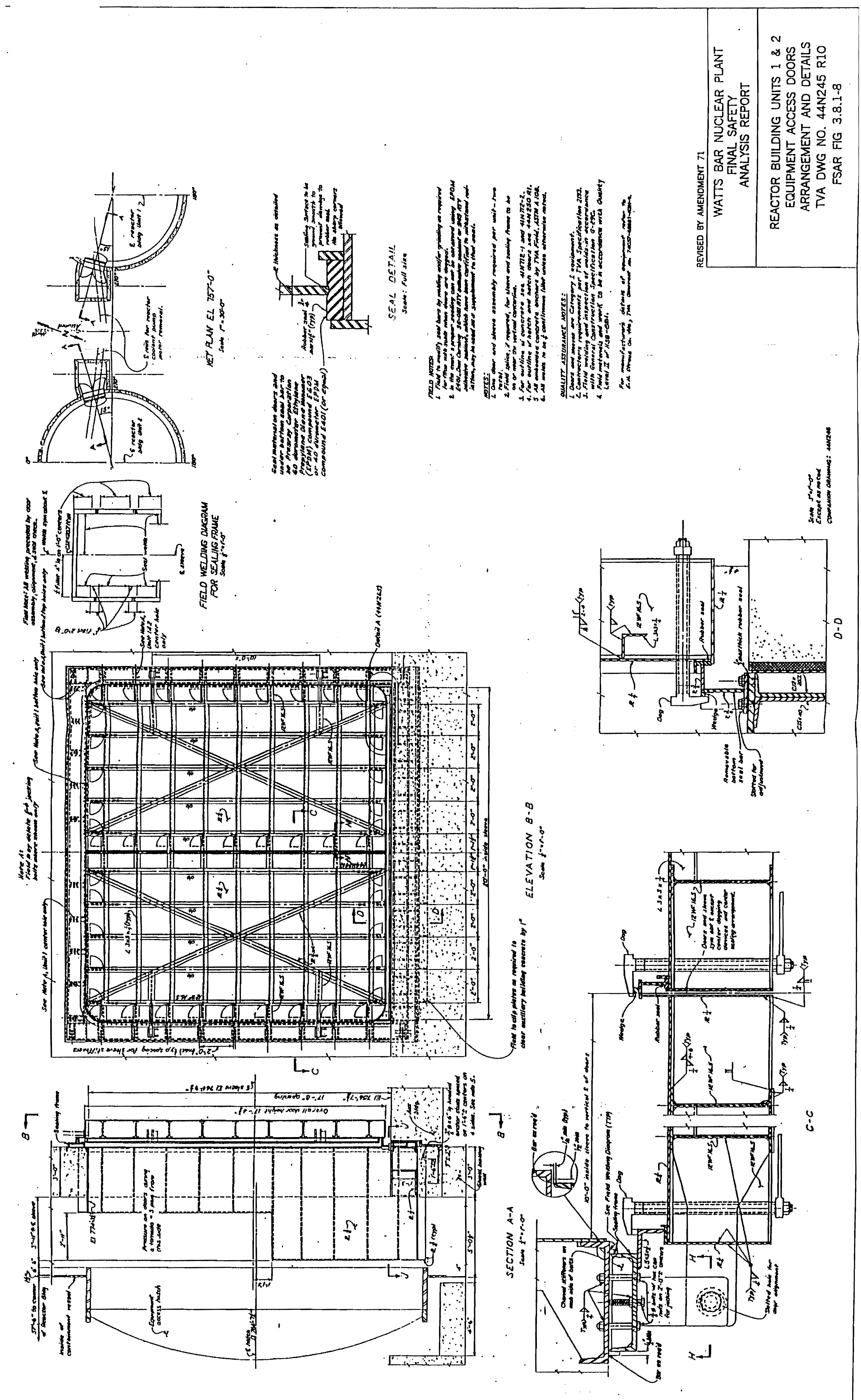
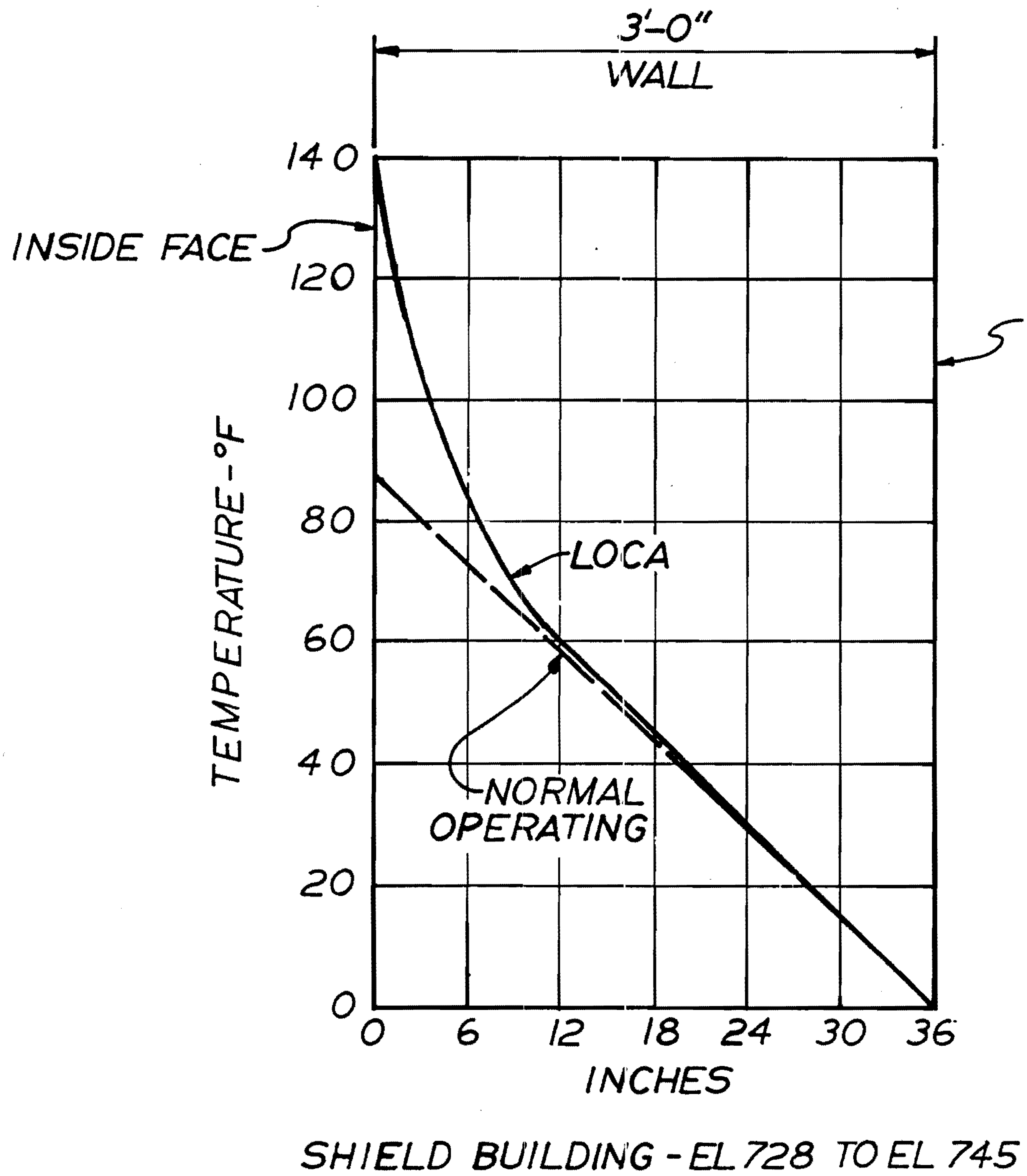


Figure 3.8.1-8 Reactor Building Units 1 & 2 Equipment Access Doors Arrangement and Details



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
SHIELD BUILDING TEMPERATURE GRADIENT elevation 728-745 Figure 3.8.1-9

Figure 3.8.1-9 Shield Building Temperature Gradient Elevation 728-745

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3.8.2 Steel Containment System

3.8.2.1 Description of the Containment and Penetrations

3.8.2.1.1 Description of the Containment

The Steel Containment Vessel (SCV) for Watts Bar is a low-leakage, freestanding steel structure consisting of a cylindrical wall, a hemispherical dome, and a bottom liner plate encased in concrete. Figure 3.8.2-1 shows the outline and configuration of the SCV.

The structure consists of cylindrical side walls measuring 114 feet 8-5/8 inches in height from the top of the liner plate to the spring line of the dome and has an inside diameter of 115 feet. The bottom liner plate is 1/4 inch thick, the cylinder varies from 1-3/8 inch thickness at the bottom to 1-1/2 inch thick at the springline, and the dome varies between 1-3/8 inch thickness and 13/16 inch thickness with 15/16 inch thickness at the apex.

The bottom liner plate serves as a leak-tight membrane only (not a pressure vessel). The liner plate is anchored to the concrete by welding it continuously to steel plates embedded in and anchored into the base mat. The anchorage system of the cylindrical walls and the juncture of the cylinder to the base mat are shown in Figure 3.8.2-2.

The SCV dome is provided with a circumferential stiffener just above the springline supports, eight penetrations, and several attachments. Two penetrations are for the residual heat removal (RHR) spray system, two penetrations are for the containment spray system, and the remaining four penetrations are spares. The major attachments to the dome consist of lighting fixture supports, header supports for the RHR spray and containment spray systems, and the collector rail supports for the polar crane. Details of these penetrations and attachments are shown in Figure 3.8.2-3.

The SCV is provided with both circumferential and vertical stiffeners on the exterior of the shell. These stiffeners are required to satisfy design requirements for expansion and contraction, seismic forces, and pressure transient loads. The circumferential stiffeners were installed on approximately 10-foot centers during erection to ensure stability and alignment of the shell. Vertical stiffeners are spaced at 5° between the two lowest circumferential stiffeners. Other locally stiffened areas are provided at the equipment hatch and two personnel locks. Exterior pipe guides and restraints for the RHR spray and containment spray systems are attached to some of the circumferential stiffeners.

3.8.2.1.2 Description of Penetrations

Most penetration sleeves were preassembled into the SCV shell plates and stress relieved prior to installation of the plates into the SCV shell. Those penetration sleeves which required field installation were provided with insert plates of the same thickness as the shell plates and stress relieved as an assembly.

Equipment Hatch

The equipment hatch is composed of a cylindrical sleeve in the containment shell and a dished head 20 feet in diameter with mating bolted flanges. The flanged joint has double gasket seals with an annular space for pressurization and testing.

The equipment hatch door, sleeve, bolts, and attachments forming the pressure boundary were designed to Section III, Class MC of the ASME Code. The hatch guide system and hatch door hoisting support structure were designed to the AISC Design Specifications.

Details of the equipment hatch are shown on Figure 3.8.2-4.

Personnel Locks

Two personnel locks are provided for each unit. Each lock has double doors with an interlocking system to prevent both doors being opened simultaneously. Remote indication is provided to indicate the position of the far door. Quick-acting type equalizing valves are used to equalize the pressure inside the lock when entering or leaving the Containment. Double seals are provided on the doors.

The personnel locks are completely prefabricated and assembled welded steel subassemblies designed, fabricated, tested and stamped in accordance with "Section III, Subsection NE" of the ASME Code.

Details of the personnel locks are shown on Figure 3.8.2-5.

Fuel Transfer Penetration

A 20-inch diameter fuel transfer penetration is provided for transfer of fuel between the fuel pool and the containment fuel transfer canal.

Expansion bellows were provided to accommodate differential movement between the connecting buildings. Figure 3.8.2-6 shows conceptual details of the fuel transfer penetration.

Spare Penetrations

Spare penetrations were provided to accommodate future piping and electrical penetrations. The spare penetrations consist of the penetration sleeve and head. Weld caps or closure plates are installed on spare penetrations to maintain containment integrity.

Purge Penetrations

The purge penetrations have one interior and one exterior quick-acting, tight-sealing isolation valve. A typical purge penetration arrangement is shown on Figure 6.2.3-2.

Electrical Penetrations

Medium voltage electrical penetrations for reactor coolant pump power use sealed bushings for conductor seals. The assemblies incorporate dual seals along the axis of each conductor.

Low voltage power, control and instrumentation cables enter the SCV through penetration assemblies which are designed to provide two leak tight barriers in series with each conductor.

All electrical penetrations are designed to maintain containment integrity for Design Basis Accident (DBA) conditions including pressure, temperature and radiation. Double barriers permit testing of each assembly as required to verify that containment integrity is maintained.

Qualification tests which may be supplemented by analysis, have been performed and documented on all electrical penetration assembly types to verify that containment integrity will not be violated by the assemblies in the event of a DBA. Existing test data and analysis on electrical penetration types may be used for this verification if the particular environmental conditions of the test were equal to or exceeded those for the Watts Bar Nuclear Plant.

Mechanical Penetrations

Typical mechanical penetrations are shown on Figures 3.8.2-7 and 3.8.2-8.

Mechanical penetration analysis is discussed in Section 3.8.2.4.6.

3.8.2.2 Applicable Codes, Standards and Specifications

3.8.2.2.1 Codes

The design of the Containment Vessel meets the requirements of the American Society of Mechanical Engineers (ASME) Code, Section III, Subsection NE, Winter 1971 Addenda and code cases 1431, 1517, 1529, 1493 and 1768.

The design of the bottom liner plates conforms to the requirements of the applicable subsections of the ASME Code, Section VIII, Division 1, and Section III, Paragraph NE-5120.

Nonpressure parts, such as supports, bracing, inspection platforms, walkways, and ladders were designed in accordance with the American Institute of Steel Construction (AISC) "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," Seventh Edition. The Eighth Edition is used for shapes not covered by the Seventh Edition.

Welding for these nonpressure parts was in accordance with the American Welding Society (AWS), "Structural Welding Code," AWS D1.1 (see Section 3.8.1.2, Item 4). Nuclear Construction Issues Group (NCIG) documents NCIG-01 and NCIG-02 (see Section 3.8.1.2, Item 11) may be used after June 26, 1985, to evaluate weldments that were designed and fabricated to the requirements of AISC/AWS.

The anchorage at the containment vessel meets the requirements of the ASME Code, Section III, with a maximum allowable stress for the anchor bolts of 2 x S_m.

All containment penetrations including the fuel transfer, purge, and mechanical within the jurisdiction of NE-1140 are designed to Section III, Class MC of the 1971 ASME Code. The penetration assemblies for those penetrations which attach to the nozzles out to and including the valve or valves required to isolate the system and provide a pressure boundary for the containment function are designed to Section III, Class 2 of the ASME Code. Spare penetrations including the nozzle caps are designed to Section III, Class MC of the ASME Code.

(Unit 1 only)
Two welds (1-074B-D045-01A and 1-074B-D045-08A) in the containment sleeves at the Unit 1 RHR sump have radiographic indications which have been interpreted as exceeding the radiographic acceptance criteria of ASME Section III. TVA has performed calculations (WBN-MTB-025 and CEB-CQS-415) which document the basis for the acceptability of these welds.

3.8.2.2.2 Design Specification Summary

Design Criteria

The containment vessel, including access openings and penetrations, is designed so that the leakage of radioactive materials from the containment structure under conditions of pressure and temperature resulting from the largest credible energy release following a loss-of-coolant accident (LOCA), including the calculated energy from metal-water or other chemical reactions that could occur as a consequence of failure of any single active component in any emergency cooling system, will not result in undue risk to the health and safety of the public, and is designed to limit below 10 CFR 100 values the leakage of radioactive fission products from the containment under such LOCA conditions.

The basic structural elements considered in the design are the vertical cylinder and dome acting as one structure, and the bottom liner plate acting as another. The bottom liner plate is encased in concrete and is designed as a leak tight membrane only. The liner plate is anchored to the concrete by welding it continuously to steel members embedded and anchored in the concrete base mat.

On the exterior at approximately 20-foot centers the containment shell is provided with circular inspection platforms which also are designed as permanent circumferential stiffeners. Additional circumferential stiffeners are provided at personnel and equipment hatches and at other large attached masses, along with vertical stiffeners for some distance above and below these attachments. Also, additional permanent circumferential stiffeners were added for stability. Temporary stiffening was not required to meet tolerance requirements specified by TVA in the erection of the vessel. The design provides for movements of the vessel and supports due to expansion and contraction, pressure transient loads, and seismic motion. No allowance is made for corrosion in determining the material thickness of the vessel shell.

The following pressure and temperatures were used in the design of the vessel:

Overpressure test (1)	16.9 psig
Maximum internal pressure (2) (3) (4)	15.0 psig at 250° F
Design internal pressure (3)	13.5 psig at 250° F
Leakage rate test pressure	15.0 psig
Design external pressure	2.0 psig
Lowest service metal temperature	30° F
Operating ambient temperature	120° F
Operating internal temperature	120° F
Design temperature	250° F

In addition, the evaluations of the vessel design have considered a harsh environment temperature of 327° F.

(1) 1.25 times design internal pressure as required by ASME Code, NE-6322.

(2) See Paragraph NE-3312(b) of Section III of the ASME Code which states that the "design internal pressure" of the vessel may differ from the "maximum containment pressure" but in no case shall the design internal pressure be less than 90% of the maximum containment internal pressure.

(3) Typical pressure transient curves are presented in Section 6.2.1. These curves show the transient pressure buildup in the compartments after a LOCA or DBA before a steady-state pressure of 15.0 psig is reached.

(4) Shell temperature transient curves are presented in Appendix 3.8A. These curves show the shell temperature at the lower compartment wall, upper compartment wall, and ice condenser wall. The maximum containment wall temperature is 220°F.

(5) A postulated main steam line break (MSLB) results in high environmental temperatures (325°F maximum) inside the lower compartment of the SCV. However, the coincident internal pressure is lower[10].

In order to ensure the integrity of the containment, an analysis of the missile and jet forces due to pipe rupture was considered. This problem was eliminated by providing barriers to protect the Containment Vessel. Typical barriers are the main operating floor (Elevation 756.63) and the crane support wall. An example of a special barrier is the guard pipe enclosing the main steam and feedwater pipes between the Shield Building and the crane wall.

Allowable Stress Criteria

Allowable stress criteria for the Containment Vessel are shown in Table 3.8.2-1. The response of the Containment Vessel to seismic and pressure transient loadings results in a condition in which buckling of the steel shell may occur. Since the ASME Code does not define the allowable buckling stresses for this type of loading condition, an acceptable buckling criteria with appropriate factors of safety is given in Appendix 3.8B.

3.8.2.2.3 NRC Regulatory Guides

Applicable NRC Regulatory Guides are shown below. These guides were used as the basis for design of a number of safety oriented features.

Regulatory Guide 1.4: Assumptions Used for Evaluating the Potential Radiological Consequences of a Loss of Coolant Accident for Pressurized Water Reactors

A dynamic analysis of the Containment Vessel was made for the pressure transient loadings. The Containment Vessel and penetrations were designed to withstand the maximum internal pressure that could occur due to a LOCA and the jet forces associated with the flow from the postulated pipe rupture.

Regulatory Guide 1.7: Control of Combustible Gas Concentrations in Containment Following a Loss of Coolant Accident

The containment vessel has a hydrogen mitigation system designed to mitigate the effects of hydrogen releases after a LOCA (Section 6.2.5).

Regulatory Guide 1.28: Quality Assurance Program Requirements (Design and Construction).

A Quality Assurance Plan for the Watts Bar Nuclear Plant was developed as a comprehensive plan for the design and construction of the Watts Bar Nuclear Plant. The Quality Assurance Plan of the Westinghouse Electric Corporation, the supplier of the Nuclear Steam Supply System, is also contained therein.

The plans were prepared to assure that the control of quality was achieved and documented for each phase of design, material selection, fabrication installation, and/or erection in accordance with the approved specifications and drawings. The plans relate principally to the reactor coolant and safety system, the containment and other components necessary for the safety of the nuclear portion of the plant.

The plan assures that:

(1) Final design requirements and final detailed designs are in accordance with applicable regulatory requirements and design bases.

(2) Components and systems to which this plan applies are identified and that the final design takes into account the varying degrees of importance of components and systems as evidenced by the possible safety consequences of malfunction or failure.

(3) Purchased material and components fabricated in vendor shops conform to the final design requirements.

(4) Components and systems are assembled, constructed, erected, and tested in accordance with the final design requirements and to requirements specified in the FSAR.

(5) The as-constructed plant can be operated and maintained in accordance with requirements specified in the FSAR.

3.8.2.3 Loads and Loading Combinations

3.8.2.3.1 Design Loads

The following loads are used in the design of the Containment Vessel and appurtenances. The loadings for the Containment Vessel were combined as in Section 3.8.2.3.2. The allowable stress criteria are shown in Table 3.8.2-1.

Dead Loads

These loads consist of the weight of the SCV, penetration sleeves, equipment and personnel access hatches, and attachments supported by the vessel.

Live Loads

Penetration loads as applicable.

Floor load of 100 psf or 1,000 pounds concentrated moving loads applied to the passage area of the personnel air locks.

Construction and snow loads at 50 psf, snow load at 20 psf during construction is considered but not simultaneously with other construction loads.

Floor load of 50 psf plus 225 pounds per linear foot for walkways.

Thermal Stresses During Design Basis Accident (DBA)

The Containment Vessel is designed to contain all the effluent which would be released by a hypothetical LOCA. This accident assumes a sudden rupture of the reactor coolant system which would result in a release of steam and a steam-air mixture in the vessel. It is calculated that this mixture would cause a lower compartment temperature of 250° F and an upper compartment temperature of 190° F, both occurring essentially instantaneously. After the accident, an internal spray system will commence spraying in the upper compartment only. The spray will discharge water on the interior of the upper compartment. For shell temperature transients refer to Appendix 3.8A.

A MSLB produces temperatures in the lower compartment of 325° F with coincident internal pressure and seismic loadings defined in load combinations 3A and 4A.

Hydrostatic Loads

The Containment Vessel is designed for three separate flood conditions. Hydrostatic load, Case IB, accounts for the flooded condition due to ice melt from the ice condenser after the DBA.

After all the ice has melted the containment will be flooded to Elevation 719 feet - 3 inches. Also considered is the loading condition during meltdown (hydrostatic load, Case IA). Water will rise to a depth of 2 feet on the floor of the ice condenser. At this time, the depth of water on the containment cylindrical shell will be 9 feet - 3 inches.

Hydrostatic load, Case II, accounts for the post-accident fuel recovery condition. In order to remove fuel from the containment after the DBA, the Containment Vessel is designed for an internal hydrostatic head of 47 feet- 3 inches.

For hydrostatic load cases refer to Figure 3.8.2-1.

Ice Condenser Duct Panel Loads

The outer duct panels of the ice condenser are attached to the containment with threaded studs. These panels impart small horizontal and vertical forces on the containment shell under seismic conditions. The distribution of these loads to the shell is shown in Figure 3.8.2-1.

Equipment Loads

Equipment loads are those specified on drawings supplied by manufacturers of the equipment.

Overpressure Test

To test the structural integrity of the vessel an overpressure test of 125% of design pressure is applied under controlled conditions.

External Pressure Load

The Containment Vessel is stiffened and designed to withstand an external pressure of 2.0 psig.

Seismic Loads

Seismic loads are generated using the methodology discussed in Sections 3.7.1 and 3.7.2.

Wind Loads

The Containment Vessel and its penetrations are completely enclosed by the Shield Building, and are therefore not subject to the effects of wind and tornadoes.

However, during construction, the vessel dome was exposed to the elements for a short duration. For this construction condition, a wind load of 30 psf on the projected area of the vessel dome was considered.

Non-Axisymmetric Transient Pressure Loads

The division of the containment into compartments is described in Section 6.2.1 and in Section 3.8.2.4.4.

Pressure transient loads are considered for occurrence of the DBA (double- ended rupture of the reactor coolant system) in all 6 lower compartment volumes. The curves presented in Section 6.2.1 represent the containment pressure transients for the controlling break locations 1 through 6 for each of the 49 containment elements.

The pressures and differential pressures shown on these figures have no margin. The initial containment pressure was assumed to be 0.3 psig. This allows for an initial containment pressure before containment venting is required. The most severe containment pressure differences occur during the first 0.9 second of the blowdown.

For structural design purposes the pressures represented by the curves are increased by 45%. This allows for changes in such factors as equipment configuration and openings between compartments, which can influence the flow characteristics of the containment space, the effects of moisture entrainment, and tolerances in the analytical constraints used in the code. (The effects of moisture entrainment, investigated by TVA and Chicago Bridge and Iron Company (CB&I), do not control the design of the Containment Vessel for any loading condition).

Local loadings from commodities attached to the SCV are calculated using dynamic response spectra generated for each area of the vessel. These spectra reflect the response of the vessel to localized dynamic pressure loadings resulting from postulated high energy pipe breaks. See Sections 3.6A and 3.6B for discussions of how these high energy break locations are determined.

3.8.2.3.2 Loading Conditions

The following loading conditions are used in the design of the Containment Vessel:

(1) Normal Design Condition

– Dead load of Containment Vessel and appurtenances

– Lateral and vertical load due to OBE

– Personnel access lock floor live load

– Penetration loads

– Design Internal Pressure or Design External Pressure

– Design temperature

(2) Normal Operation Condition Operating Basis Earthquake (OBE)

– Dead load of Containment Vessel and appurtenances

– Lateral and vertical load due to OBE

– Penetration loads

- Spray header and lighting fixture live loads
- Walkway live loads
- Personnel access lock floor live load
- Internal temperature range 60°F to 120°F

(3A) Upset Condition - DBA and OBE

- Dead load of Containment Vessel and appurtenances
- Design internal pressure of 13.5 psig
- Lateral and vertical load due to OBE
- Penetration loads
- Thermal stress loads including shell temperature transients
- Hydrostatic Load Case IA or IB
- Internal temperature range 80°F to 250°F

(3B) Upset Condition - DBA and OBE

- Dead load of containment vessel and appurtenances
- Pressure transient loads
- Lateral and vertical load due to OBE
- Penetration Loads
- Thermal stress loads including shell temperature transients
- Hydrostatic Load Case IA or IB
- Internal temperature range of 60°F to 120°F

(3C) Upset Condition MSLB

- Dead load of Containment Vessel and apurtenances
- Internal pressure coincident with MSLB[10]
- Lateral and vertical load due to OBE.
- Spray header loads
- Ice condenser duct load

– Thermal load due to temperature range 80°F to 325°F

– Penetration loads

(4A) Emergency Condition - DBA and SSE

– Dead load of Containment Vessel and appurtenances

– Design internal pressure of 13.5 psig

– Lateral and vertical load due to SSE

– Penetration loads

– Thermal stress loads including shell temperature transients

– Hydrostatic Load Case IA or IB

– Internal temperature range 80°F to 250°F

(4B) Upset Condition - DBA and SSE

– Dead load of containment vessel and appurtenances

– Pressure transient loads

– Lateral and vertical load due to SSE

– Penetration Loads

– Thermal stress loads including shell temperature transients

– Hydrostatic Load Case IA or IB

– Internal temperature range of 60°F to 120°F

(4C) Emergency Condition MSLB

– Loads are same as in Condition 3C except lateral and vertical load due to SSE

(5) Construction Condition at Ambient Temperature

– Dead load of Containment Vessel and appurtenances

– Snow load at 20 psf

– Lateral load due to wind

– Temporary construction live loads on catwalks, platforms, and hemispherical head including support of the first pour of the concrete Shield Building dome.

(6) Test Condition at Ambient Temperature

- Dead load of Containment Vessel and appurtenances
- Internal test pressure
- Weight of contained air

(7) Post-Accident Fuel Recovery Condition with Flooded Vessel

- Dead load of Containment Vessel and appurtenances
- Hydrostatic Load Case II

3.8.2.4 Design and Analysis Procedures

3.8.2.4.1 Introduction

The design, fabrication, and erection of the SCV were contracted to Chicago Bridge and Iron Company (CB&I), Oakbrook, Illinois. The design of the vessel was reported by CB&I in a 12-volume stress report from which the following design and analysis procedures were taken. TVA reviewed the stress report as required by ASME Code Section NA-3260. Furthermore, TVA performed a complete design review of CB&I work to insure the adequacy of the design. As part of the design review, independent analyses were performed for seismic, thermal and pressure transient loading conditions.

Compressive stresses in the Containment Vessel are produced by dead, live, seismic, and pressure transient loads. But pressure transient loads are by far the most significant loads to the stability of the vessel. Therefore, buckling is addressed only in Section 3.8.2.4.4.

3.8.2.4.2 Static Stress Analysis

A detailed stress analysis of all major structural components was prepared in sufficient detail to show that each of the stress limitations of the ASME Boiler and Pressure Vessel Code, Section III, Section NE-3000 was satisfied when the vessel is subjected to the loading combinations enumerated in this section.

Details of the juncture of the cylinder to the base mat are shown in Figure 3.8.2-2. In the analysis, the juncture was considered to be a point of infinite rigidity. The cylinder at this point cannot expand or rotate under the internal pressure and temperature load conditions; hence, shear and moment are introduced into the cylinder wall.

At the point the knuckle is welded to the vessel, a backup stiffener is used. This stiffener gives added rigidity at the point of the weld. Additional protection of the knuckle is accomplished by encasing the knuckle in 'Fiberglass' before floor concrete placement.

The embedded knuckle was designed to take interior pressure plus internal or external hydrostatic loads. It was assumed that cracks can occur in the concrete allowing pressure loads on the embedded knuckle. Anchor bolts were post-tensioned to prevent any cracking of the concrete. Thermal and pressure discontinuity stresses in the containment occur one foot above the last weld of the knuckle.

The stresses due to dead loads internal, and snow loads were determined at a sufficient number of locations to define the state of stress in the vessel under these loadings. Wind, snow and external support loads on the dome occurred during construction. Stresses due to dead loads, internal and external pressure were determined by hand calculations using classical strength of materials theory. Detail stresses in the embedment region at the base of the vessel were determined from a shell model of the vessel using CB&I computer program 781 described in Appendix 3.8C. The circumferential stiffeners on the embedment region were modeled as horizontal elements and the effect of vertical stiffeners was considered by modeling the shell plate as an orthrotropic material. Forces and bending moments due to the various loads were given by CB&I computer program 781, whereas the resulting detailed stress distribution was calculated using actual geometry of the vessel and stiffening in this region.

Design of spherical and cylindrical vessels for internal and external pressure is explicitly treated in Section NE of the ASME Boiler and Pressure Vessel Code. The vessels as designed are in full compliance with the Code requirement for internal and external pressure and provisions applicable to other load conditions.

3.8.2.4.3 Dynamic Seismic Analysis

The SCV dynamic analysis is discussed in Section 3.7.2.1.

3.8.2.4.4 Non-Axisymmetric Pressure Loading Analysis

The non-axisymmetric pressure loading (NASPL) results from an assumed sudden rupture in the reactor coolant system. The associated pressure loads are dynamic in nature and vary with time in both the circumferential and meridional directions in the vessel. The loads are non-axisymmetric for a short period culminating in uniform internal pressure throughout the containment. For analysis purposes, the containment was subdivided into forty-nine volumes and pressure-time histories determined for each volume for the postulated rupture, i.e., each break in the reactor coolant system. The pressure histories for each of the volumes were computed by the Westinghouse Electric Corporation using the TMD code network documented in Section 6.2.1.3. Figures 3.8.2-10 and 3.8.2-11 show the volumes used to characterize the pressure in the containment.

Dynamic analyses were made by CB&I for twelve breaks in the reactor coolant piping, six hot leg and six cold leg breaks. Two separate and distinct analysis methods were used in the design process. The overall vessel response was determined by a dynamic analysis treating the vessel as a lumped mass cantilever beam and by a dynamic shell analysis which considered the effects of local vibration modes.

(1) Beam Analysis

In the CB&I lumped mass beam analysis, each mass represented the mass of the vessel stiffeners and attached masses. The cantilever beam model was loaded with the forces from the NASPL. The forces were resolved into X and Y components and applied as mass point loads in the north-south and east-west directions.

The response of the model to non-axisymmetric pressure transients was calculated by CB&I Program 1642 described in Appendix 3.8C. It employs the method of numerical integration and solves for natural frequencies, accelerations, overturning moments, and shears.

(2) Shell Analyses

Independent dynamic shell analyses of the containment were performed by both CB&I and TVA. The shell model used by CB&I is shown in Figure 3.8.2-12. The method of analysis involves a numerical integration technique operating on the governing differential equations. Linear behavior and axisymmetric geometry were assumed. The total transient response was calculated by the sum of the harmonic responses with the input loads being represented by Fourier Series. A full explanation of the method is given in Reference [1]. A number of CB&I proprietary programs, all described in Appendix 3.8C, were employed to arrive at the final shell responses. Figure 3.8.2-13 is a flow diagram of the analysis process with a brief description of the function accomplished by each computer program. CB&I Program 1624 (also in Appendix 3.8C) calculated acceleration response spectra at various elevations and azimuths from the acceleration histories.

TVA performed an independent shell analysis of the transient pressure response. A finite element model was used and the solution calculated by numerical integration. The agreement with the CB&I analysis was good. Since the TVA shell analysis was merely a check on the CB&I analysis, full documentation of the process and the programs used are not included herein.

The pressures were factored by 1.45 for computing responses to be used to ensure compliance with the buckling criteria in Appendix 3.8B. A factor of 1.80 was used in the design of the anchorage (see Section 3.8.2.4.8).

3.8.2.4.5 Thermal Analysis

A thermal analyses was performed on the containment for a loss-of-coolant accident. The shell temperature transients due to a double end rupture of a reactor coolant pipe are described in Appendix 3.8A. The tolerable temperature rise for the steel containment is well above the temperatures shown, since the steel shell was designed for the basic stress limits of Section NB-3221 and Section NB-3222.2 of the ASME Boiler and Pressure Vessel Code, Section III, for ASME SA-516, grade 70 steel at 300° F.

Also, as seen by these curves, the containment shell will experience an unbalanced temperature loading for the three compartments. The temperature difference between any two adjacent points on the vessel is held within the limits of Section NB-3222.4 of the code.

TVA performed a study to determine the effect of MSLB temperature on the SCV. The impact of the thermal movements on attached penetrations and appurtenances was also accounted for in this study. This study indicated that the SCV and attachments are still within acceptable ASME stress limits under MSLB.

3.8.2.4.6 Penetrations Analysis

The vessel manufacturer is responsible for the design of the steel containment including the reinforcement required at the penetrations. The specifications required the manufacturer to submit all preliminary design calculations for TVA's review before any material was detailed or fabricated. Penetrations requiring requalification after CB&I completed their contract were analyzed by TVA. TVA used essentially the same methodology and design criteria as CB&I. However, TVA used its own in-house developed computer program TPIPE to sum load combinations and hand calculations to calculate nozzle stresses and a public domain program (WERCO) to calculate shell stresses. The WERCO program employs the methodology of the Welding Research Council Bulletin No. 107.

Also, TVA performed an independent analysis of the steel containment, including the reinforcement required at penetrations.

Secondary and local stresses at penetrations subjected to applied loads were analyzed by CB&I programs 1027 and 1036, which are described in Appendix 3.8C. These programs employ the methods of the Welding Research Council Bulletin No. 107 in the analysis of the containment shell.

Penetrations not subjected to applied loads were designed in accordance with Section NE-3332 of Section III, ASME Code. Most penetrations were preassembled into the Containment Vessel shell plates and stress relieved prior to installation of the plate into the Containment Vessel shell. All other penetrations were installed in insert plates of the same thickness at the perimeter as the shell plates and stress relieved as assemblies. As a result, no reinforcement is provided in excess of that available in the shell and neck. Large penetrations, such as the large equipment hatch and personnel access locks, require stiffeners for reinforcement.

The penetrations subjected to external loads are supplied with pipe of sufficient wall thickness to resist these loads. Where one or more externally loaded penetrations are in close proximity to another externally loaded penetration or pad plate, the shell was analyzed for the interactive effects of these loaded penetrations.

The external loads were assumed to be reversible and the maximum stress combination was determined. Since pressure affects the design of the penetrations, a pressure equal to the internal design pressure is considered to act in conjunction with the externally applied loads.

Figure 3.8.2-14 shows the stresses assumed to be present in the analysis of the shell in the vicinity of the penetrations. These assumed stresses, which are due to internal containment pressure, are added to the stresses resulting from the externally applied loads before determining the stress intensities. The assumed stresses are employed as shown in Figure 3.8.2-14 for most of the penetrations. However, it is permissible to reduce these initial stresses when the penetration is provided with greater reinforcement than is required by Section III. At the point of intersection of the shell and penetration, a factor equal to the ratio of the area required for reinforcement within the two-thirds limit to the area available for reinforcement may be used to reduce the assumed initial stresses. At points in the shell away from this intersection, the factor becomes the ratio of required shell thickness to actual shell thickness. This reduction method was used on penetrations which were over-stressed when the assumed initial stresses used were as shown in Figure 3.8.2-14. While the factor for all penetrations using this method was less than 0.5, the minimum factor used in the analysis was 0.5.

The neck of the penetration was analyzed using CB&I Program 1392, described in Appendix 3.8C. This program computes the stresses in the neck at two points. The first point is located at a distance from the shell that is outside the normal limits for area replacement. The stresses at this point are due to the external loads and to the containment design pressure acting within the pipe. The second point is located within the area considered for area replacement. In addition to the stresses due to external loads and containment pressure, an assumed stress is also included. This assumed stress is as outlined above at the point of intersection of the shell and penetration and may be modified as discussed above. Permanent caps for spare penetrations are designed in accordance with ASME rules. Flanged penetrations are provided with double gasket details which permit the testing of the gaskets by pressurizing the air space between the gaskets.

The Heating, Ventilation and Air Conditioning (HVAC) penetrations were also analyzed by TVA. The entire piping assembly from the flexible connection in the Reactor Building to the flexible connection in the annulus was modeled including pipe, isolation valves, and pipe supports using discrete finite element representation. Shell flexibility was taken into account at the nozzle/shell intersection and at the hanger/shell attachments by inputting equivalent translational and rotational stiffness rates.

A response spectrum modal analysis was performed for the seismic and design basis accident condition using the floor response spectra nearest to the penetration locations. The total stress in the nozzle was calculated using the absolute summation of dead load, seismic, and DBA. The stress in the shell was analyzed by inputting the loads above into WERCO.

Other nonprocess and electrical penetrations were also analyzed by TVA. These penetrations were analyzed using the static acceleration technique in which the weight is multiplied by the peak accelerations from the seismic and DBA spectra times a 1.5 amplification factor and applied at the mass center of the assembly. The resulting stresses in the nozzle and shell were calculated using the technique used for qualifying mechanical penetrations.

3.8.2.4.7 Interaction of Containment and Attached Equipment

Some items rigidly attached to the containment respond in a nonrigid manner due to the local flexibility of the containment. This effect was analyzed for a number of penetrations and other attachments, but was found to be significant only for the equipment hatch, two personnel locks, and the HVAC penetrations.

The following procedure was followed in the equipment hatch and personnel lock analyses:

(1) Linear and rotational mass moments of inertia were calculated in the radial, circumferential, and longitudinal directions. (The rotational degrees-of-freedom were considered because the centers of mass did not lie in the plane of the containment shell).

(2) The local stiffnesses of the hatch and locks were calculated for the above degrees-of-freedom. A method developed by Bijlaard[2] was used.

(3) The periods of vibration were calculated for motions in the radial (push-pull) direction, and in the circumferential and longitudinal (swinging) directions by the equation:

$$T = 2\pi\sqrt{\frac{I_o}{K}}$$

where I_o and K are the mass moments of inertia and stiffnesses, respectively.

(4) The response accelerations for seismic excitation and the pressure transients were taken from the spectra described in Sections 3.8.2.4.3 and 3.8.2.4.4, respectively.

(5) The total structural response was found by the sum of the effects of the seismic, pressure transient, and dead weight loads.

All of the above calculations were performed by hand. The periods of vibration of the equipment hatch and the personnel locks in the three principal directions were all greater than 0.03 seconds, which is used as the demarcation between rigid and non-rigid vibration.

3.8.2.4.8 Anchorage

The Containment Vessel anchorage system consists of anchor bolts, an embedded anchor plate, and an anchor bolt bearing ring which attaches to the first shell ring. Details of the anchorage are shown in Figure 3.8.2-2.

Two rows of 3-1/2 inch anchor bolts are provided with one row on the outside of the shell and one row on the inside of the shell. The bolts in each row are spaced at two degrees and located in pairs on radial lines. The rows are located at equal distances from the center line of the shell.

The anchor bolts are embedded in the concrete to the maximum depth available. The majority of the bolts are embedded to a depth such that the lowest point on the bolts is slightly above Elevation 687.0. The remainder of the anchor bolts, located in the area of the pipe sleeves which extend from the penetration for the containment sump, are embedded with their lowest points at Elevation 689.3 being slightly above the sleeves. An embedded anchor plate at the lower end of the bolts is provided to transfer the bolt load to the concrete. The design of the bolt is based on using an allowable stress of 2 x S_m. Allowable stresses in the concrete are based on a specified strength of 5000 psi.

Loads considered in the design consist of dead loads, seismic loads, and NASPL loads. The NASPL loads have been increased by 80% for the design of the anchorage.

The anchor bolts were pretensioned during construction to assure fixity of the base during an operating accident. Since the concrete is subject to creep over a period of time, the effects of creep were calculated and bolt preload was increased accordingly. The initial bolt strain was calculated based on this preload.

The embedded anchor plate is a ring designed to transfer the bolt loads to the concrete. The design assumed that the ring is discontinuous at points midway between bolts. This approach permits the butts in the ring to be unwelded.

The tensile loads in the shell are greater than the compressive loads. Since the bolts are preloaded, the effect is that the anchorage is placed in compression. As a result, the anchorage system was designed for the bolt pre-load plus the compressive shell load.

3.8.2.5 Structural Acceptance Criteria

3.8.2.5.1 Margin of Safety

A certified stress report was prepared by CB&I for the vessel in accordance with the requirements of the ASME Code. This report contains several hundred pages and therefore is not included in this report.

Design values for transient pressure loads were determined by multiplying the calculated values by 1.45 as described in Section 3.8.2.3.1. In addition, the buckling criteria, in Section 5 of Appendix 3.8B, require a load factor of 1.25.

Nonpressure parts such as walkways, handrail, ladders, etc., were designed in accordance with AISC "Manual of Steel Construction," seventh edition, so that the stress in the members and welds does not exceed the allowable stress criteria as set forth in the February 1969, AISC "Specifications for Design, Fabrication, and Erection of Structural Steel for Buildings." The factor of safety of these allowable stresses with respect to specified minimum yield points of the material used are as defined in

Section 1.5 of "Commentary on the Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings."

Local areas, such as the personnel and equipment hatch areas, were checked for deformations to avoid a resonant condition. The vessel as a whole was not designed to deformation limits.

Shutdowns and startups do not occur with a frequency to require a design for fatigue failure. The number of load cycles will not affect the containment vessel service life.

The stability of the containment vessel was evaluated by the criteria of Appendix 3.8B. This criteria is applicable to stiffened circular and spherical shells and independent panels. A factor of safety was used in the design related to buckling. Loading conditions which included SSE used a factor of safety of 1.1. The factor of safety for external pressure was provided by the ASME Code. The factor of safety for all other loading conditions was 1.25.

3.8.2.6 Materials, Quality Control, and Special Construction Techniques

3.8.2.6.1 Materials - General

Materials for the containment vessels, including equipment access hatches, personnel access locks, penetrations, attachments, and appurtenances meet the requirements of the following specifications of the issue in effect on the date of invitation for bids. Impact test requirements were as specified in the ASME Boiler and Pressure Vessel Code, Section III for maximum test metal temperature of 0°F. Charpy V-notch specimens, SA-370, type A, were used for impact testing materials of all product forms in accordance with the requirements of the ASME Boiler and Pressure Vessel Code, Section III. In order to provide for loss of impact properties during fabrication, all materials were either furnished with an adequate test temperature margin below the minimum NDT temperature, or the specified minimum values were effectively restored by heat treatment in accordance with ASME Code requirements.

Material Designations

Plate for Vessels

Carbon steel	SA-516, Grade 70 carbon steel plates for pressure vessels for moderate and lower temperature service.
Austenitic stainless steel	SA-240, Type 304

Forgings

Carbon steel	SA-350, Grade LF1 for welding

Austenitic stainless steel	SA-182, Grade F304 or F316
Carbon steel (for fittings or couplings)	SA-105, SA-181, Grade II, or SA-234, Grade WPB.
Austenitic stainless steel (for fittings or couplings)	SA-403, WP316 or SA-234, Grade WPB

Pipe

Carbon steel	SA-333, Grade 1 or 6, seamless, or SA-155, Grade KCF70, electric fusion-welded
Austenitic stainless steel	SA-312, Grade TP316, seamless, SA-358, Class 1, Grade 316, electric fusion-welded.
Carbon steel (for leak chase piping and platform handrail piping)	SA-53 or SA-106

Castings

Carbon steel	SA-216, Grade WCB, or SA-352, Grade-LCB
Carbon steel (for lock and hatch mechanisms)	ASTM A27, Grades 70-36
Austenitic stainless steel (for personnel lock equalizing valve bonnet, ball, and body)	SA-351, Grade CF8M
Cold finished steel (for lock and hatch mechanisms)	ASTM A108, Grades, 1018 to 1050 inclusive
Bar and machine steel (for lock and hatch mechanisms)	ASTM A576, special quality, carbon content not less than 0.30 percent.

Fasteners

Carbon steel	SA-320, Grade L7 or L43; SA-193,Grade B7; or SA-194,Grade 2H or 7
Austenitic stainless steel	SA-193, Grade B8, or SA-194, Grade 8
Carbon steel (for platform bolts and nuts)	A307, Grade B

Welding Electrodes

Carbon steel	SFA-5.1, E 70 Classification Submerged Arc SFA-5.17, EL or EM; Gas metal Arc SFA-5.18, E70-S-1 through E70-S-6; Gas Tungsten Arc SFA-5.18, E70-S-1 through E70-S-6.
Austenitic stainless steel	SFA-5.4 E308 or E309 Classification; SFA-5.9, ER308 or ER309 Classification

Structural Steel

Plates, bars, and shapes (other than vessel plates)	ASTM A36, A283, Grade C, A514 Type F A537, Class 1
Plates (leak chase and built-up sections)	SA-516, Grade 70.
Plates (platform walkways and personnel lock floor plate)	Regular quality carbon steel nonskip S400
Fittings	A105, A181, Grade II

Gasket materials, including O-ring seals and flexible membrane seals, shall be of Ethylene Propylene Diene Monomer (EPDM) material, Presray elastomer compound E603 or other suitable elastomers in continuous rings and with a Shore A durameter of hardness of 50-70 prior to exposure at operational conditions.

Installed seals, packages spares and replacement are to be examined after delivery. Prior to initial startup and then at 18-month intervals thereafter, the installed seals are to be examined. Visual examination is required to determine if there is any evidence of cracking which would result in establishing a leak path for air. If any cracking of the seal is observed, the seal is to be replaced.

Minimum values of seal material properties are to be as following:

	ASTM Spec	Before Exposure	After Exposure
Durometer	D2240	50-70	45-75
Min. tensile	D412	1800 psi	900 psi
Min. elongation	D412	400%	150%
Max. compression set	D095	20%	30%

Seals and gasket materials are required to withstand radiation of 10^8 Rads.

3.8.2.6.2 Corrosion Protection

Potential corrosion of the steel containment has been considered at both the embedded bottom liner in conjunction with the concrete, at the inner face in the region of the ice condenser, and at the outer face exposed to the annulus atmosphere.

The conditions which determine corrosion are basically the electro-potential of the materials involved, the presence of oxygen and an electrolyte, temperature and may induced electro-potential, from extraneous sources. These have been evaluated in the determination of corrosion.

The containment material is to specification SA-516, Grade 70, being a 1% manganese, 0.3% silicon low carbon steel, and has interfaces with concrete. Thus no unfavorable electro-potentials exist in the materials.

The climatic conditions for Chattanooga, Tennessee, show an ambient annual temperature of 0°F to 100° F [3]. The corresponding temperature for the steel containment in the region of the ice condenser are approximately 32° F to 120° F.

The corrosion of the steel containment face in contact with the containment concrete is not a design consideration since portland cement concrete provides good protection to embedded steel. The protective value of the concrete is ascribed to its alkalinity and relatively high electrical resistivity in atmospheric exposure.

Reference [4] identifies three basic conditions as being conducive to the corrosion of steel in concrete.

(1) The presence of cracks extending from the exposed surface of the concrete to the steel.

(2) Corrosion cells arising from electro-potential differences in the concrete itself.

(3) Electrolysis by induced currents in the concrete or steel.

With respect to condition (1) the base consists of a 3-foot thick concrete embedment surrounding all the steel containment. The cracking under the worst of cases is considered minimal. This quantity far surpasses minimum cover recommended by ACI 201-1 in the most corrosive marine environment.

The potential for developing corrosion cells was kept to a minimum by limiting the soluble salts and chlorides in the concrete. Further, the continuing corrosion of iron under these conditions requires that the hydrogen deposited at the cathode is freed or combined with oxygen. Since both these mechanisms are prevented by the concrete, the corrosion cells are polarized, and the reaction is brought to a standstill.

To preclude the development of induced electric currents and in keeping with good construction practice, all electrical equipment and structures are grounded as determined by the resistivity of the foundation materials for the site. Foundation material resistivity surveys were made and the result considered in the design and determination of the extent of the grounding mat.

The seasonal variation of steel containment temperature in the region of the ice condenser gives rise to a range of relative humidity from 4% at 120° F to 45% at 32° F. This is based on saturated air leaking from the cooling ducts at a temperature of 10° F and rising to the steel containment temperature at the containment surface.

The annular region exterior to the steel containment is essentially airtight. Only during periods of shutdown during which access doors are open will this seal be broken. In the event of a pipe rupture in the annular region, water would be removed by a drainage system at the base of the annulus.

Any ingress of moisture to the interior steel containment face is prevented by sealing the outer periphery of the ice condenser adjacent to the steel containment, and by the vapor barrier on the inside face of the duct panels at the boundary of the ice bed. In the event of any abnormal ingress of moisture through the seal, the leakage air from the cooling ducts has the capacity to absorb moisture up to the limits of the relative humidities quoted above. In addition, any moisture remaining will have a tendency to migrate to the colder end of the temperature gradient; i.e., for all steel containment temperatures above 10° F, moisture will migrate towards the cooling air ducts, where it will be evaporated as the cooling air increases in temperature in the course of its passage through the ducts.

For steel containment temperatures below 32° F any moisture at the steel containment face will be frozen, this condition pertaining to relative humidities greater than 45% and steel containment temperatures below 10° F when the migration of moisture could take place from the air cooling ducts to the steel containment.

In the event of actuation of the containment spray, water would be applied to the interior surface of the steel containment. Most of the water would be removed by the drainage system and the small amount of moisture remaining would be removed from the steel containment surface by evaporation.

Several references have been established which give corrosion data for the limits of the conditions described above.

For low alloy steels in any industrial atmosphere long-term tests indicate a maximum total corrosion of 0.016 inch in 40 years (based on 14g/sq dm in 18 years [5]).

For dry inland conditions which more closely simulate the steel containment conditions the total corrosion for the plant lifetime is approximately 0.010 inch[7]. This is accounted for by the fact that below relative humidities of 65%, iron oxide itself forms an adherent film affording good protection to further corrosion[6, 8]. Furthermore, at temperatures below freezing, ion transport in the electrolyte is almost entirely inhibited, obviating the mechanisms of corrosion[9].

It is concluded that the maximum total corrosion for any exposed internal surface of the steel containment in the region of the ice condenser is 0.010 to 0.015 inch over the lifetime of the plant. In general, the corrosion in the region of the ice condenser is expected to be less than in other areas of the containment, which can be readily inspected.

3.8.2.6.3 Protective Coatings

Protective coatings were applied to all exposed steel surfaces of the Containment Vessel. Surfaces embedded in concrete will not be coated. For coating systems used on the inside of the containment, see Section 6.1.2.

All exterior vessel shell surfaces and metal surfaces of platforms, floor plate, ladders, walkways, attachments, and accessories located in the annular space surrounding the Containment Vessel were cleaned in accordance with the requirements of Steel Structures Painting Council Surface Preparation Specification No. 6, Commercial Blast Cleaning, latest edition. After cleaning and having passed inspection, one complete prime shop coat of Carboline Carbozinc 11 paint (dry film thickness was not less than 2-1/2 mils) was applied in accordance with the manufacturer's instructions.

All interior surfaces of the containment vessel shell and metal surfaces of attachments thereto, except those parts embedded in the base slab and identified as the liner and areas within 2 inches of field-welded joints, were given one prime coat of Carboline Carbozinc 11 within 8 hours after blast cleaning in accordance with Steel Structures Painting Council Surface Preparation Specification No. 10, Near-White Blast Cleaning, latest edition. The primer was top-coated by TVA field forces with an epoxy coating as recommended. The surfaces of the vessel in the annular space were coated with materials selected for the ability to provide protection against atmospheric corrosion.

3.8.2.6.4 Tolerances

The Containment Vessel as constructed does not exceed the applicable tolerance requirements of the ASME Code for fabrication or erection.

The out-of-roundness tolerance does not exceed 0.5% of the nominal inside diameter.

The deviation from a vertical line of the vertical cylindrical portion adjacent to the ice condensers is limited to ±2 inches for the height of the ice condensers.

Threaded studs for attachment of ice condenser outer duct panels do not vary from their theoretical location by more than ±1/4 inch.

Penetrations do not vary from their theoretical location by more than ±1/2 inch.

3.8.2.6.5 Vessel Material Inspection and Test

ASTM standard test procedures were employed for the liner and shell plates to ascertain compliance with ASTM specifications. Certified copies of mill test reports of the chemical and physical properties of the steel were submitted to TVA for approval. Tests for qualifying welding procedures and welders were also submitted for approval. All vessel pressure boundary material was tested (one test for each heat of steel) to determine its Nil Ductility Transition Temperature (NDTT). These tests were conducted to meet the requirements of ASME Boiler and Pressure Vessel Code, Section III, Paragraph NB-2300. The tests were conducted at a maximum temperature of 0° F.

Ultrasonic inspection was required for all pressure boundary plates subjected to tensile forces normal to the plate surface. This inspection was performed in accordance with ASME Boiler and Pressure Vessel Code, Section III, NB-2530.

3.8.2.6.6 Impact Testing

Charpy V-notch impact tests were made of material, weld deposit and the base metal weld heat affected zone employing a test temperature of not more than 30° F below minimum operating temperature. The requirements of the ASME Code, Paragraph NB-2300, were met for all materials under jurisdiction of the code. All weld procedure qualifications for procedures used on the Containment Vessel shell also meet code requirements for ductility.

3.8.2.6.7 Post-Weld Heat Treatment

Field welded joints did not exceed 1-1/2 inch and therefore the containment vessel as a completed structure did not require field stress relieving. Insert plates at penetration openings did not exceed 1-1/2 inches in thickness and stress relieving was not required by ASME Code before or after they were welded to adjacent plates. Post-weld heat treatment, where required, was performed as required by and in accordance with the ASME Code.

3.8.2.6.8 Welding

All welding procedures were qualified under provisions of Part A of Section IX of the ASME Code. Welding procedures were submitted to TVA for approval before welding was started. All welding was performed by welders qualified in accordance with Part A of Section IX of the ASME Code.

3.8.2.7 Testing and Inservice Inspection Requirements

3.8.2.7.1 Bottom Liner Plates Test - Historical Information

Before concrete was placed over the bottom liner, the leak tightness of this liner was verified. All liner plate welds were vacuum box tested for leak tightness. Upon completion of a successful leak test, the welds were covered with channels, and the channels were leak tested by pressurization to 15 psig.

3.8.2.7.2 Vertical Wall and Dome Tests - Historical Information

Welds in the cylinder wall and dome in ASME Code Section III, Categories A and B, were 100% radiographed. Welds in Categories C and D were examined by magnetic particle, liquid penetrant, or by ultrasonic methods.

3.8.2.7.3 Soap Bubble Tests - Historical Information

Upon completion of the construction of the Containment Vessel, a soap bubble test was conducted with the vessel pressurized to 5 psig. Soap solution was applied to all weld seams and gaskets, including both doors of the personnel airlocks.

A second soap bubble inspection test was made at 13.5 psig upon completion of the overpressure test in accordance with the requirements of the ASME Code.

Any leaks detected by soap bubble test which could affect the integrity of the vessel or which could result in excessive leakage during the leakage rate tests were repaired prior to proceeding with the tests.

3.8.2.7.4 Overpressure Tests - Historical Information

After successful completion of the initial soap bubble test, a pneumatic pressure test was made on the Containment Vessel and each of the personnel airlocks at a pressure of 16.9 psig. Both the inner and the outer doors of the personnel airlocks were tested at this pressure. The test pressure in the Containment Vessel was maintained for not less than 1 hour.

3.8.2.7.5 Leakage Rate Test - Historical Information

Following the successful completion of the soap bubble and overpressure tests a leakage rate test at 15 psig pressure was performed on the Containment Vessel with the personnel airlock inner doors closed.

CB&I performed the leak rate testing by the "Absolute Method," which consists of measuring the temperature, pressure, and humidity of the contained air, and making suitable corrections for changes in temperature and humidity.

Equipment and instruments were calibrated and certified before any pressure tests were initiated.

Continuous hourly readings were taken until it was satisfactorily shown that the total leakage during a consecutive 24 hour period did not exceed 0.1% of the total contained weight of air at test pressure at ambient temperature in accordance with the requirements of 10 CFR 50, Appendix J.

CB&I reviewed the leakage rate data during the test to determine adequacy of the test, authorize termination, or require continuation of the test.

3.8.2.7.6 Operational Testing - Historical Information

After completion of the airlocks, including all latching mechanisms, interlocks, etc., each airlock was given an operational test consisting of repeated operation of each door and mechanism to determine whether all parts are operating smoothly without binding or other defects. All defects encountered were corrected and retested. The process of testing, correcting defects, and retesting was continued until no defects were detectable.

3.8.2.7.7 Leak Testing Airlocks - Historical Information

The airlocks were pressurized with air to 16.9 psig. All welds and seals were observed for visual signs of distress or noticeable leakage. The airlock pressure was then reduced to 13.5 psig, and a thick soap solution was applied to all welds and seals and observed for bubbles or dry flaking as indications of leaks. Leaks and questionable areas were clearly marked for identification and subsequent repair. During the overpressure testing, the inner door was locked with hold-down devices to prevent upsetting of the seals.

The internal pressure of the airlock was reduced to atmospheric pressure and all leaks repaired after which the airlock was again pressurized to 13.5 psig with air and all areas suspected or known to have leaked during the previous test were retested by above soap bubble technique. This procedure was repeated until no leaks were discernible by this means of testing.

3.8.2.7.8 Penetration Tests - Historical Information

Type B tests were performed on all penetrations with test bellows and/or pressure taps in accordance with the requirements of 10 CFR 50, Appendix J. See Section 6.2.6 for imposed leak rates and tests performed on penetrations.

3.8.2.7.9 Inservice Inspection Requirements

3.8.2.7.9.1 Components Subject to Examination and/or Test

All ASME Code Class MC and metallic liners of Code Class CC components shall be examined and tested in accordance with Section XI of the ASME Boiler and Pressure Vessel Code and as required by 10 CFR 50.55a(b)(2)(x), 50.55a(g)(4), and 50.55a(g)(6)(ii)(B), except where specific written relief has been requested. The inservice inspection requirements are contained in Section 3.8.5.1.1 for ASME Code Class CC concrete comoponents. The inservice inspection requirements are contained in Section 5.2.8 for ASME Code Class 1 components and Section 6.6 for ASME Code Class 2 and 3 components. Inservice leakage rate tests and inservice surveillance of the containment vessel are discussed in Section 6.2.6.

3.8.2.7.9.2 Accessibility

Watts Bar design was established prior to the publication of Subsection IWE of ASME Section XI; however, accessible Class MC and metallic liners of Class CC components will be inservice examined in accordance with the guidelines of Subsection IWE of ASME Section XI.

3.8.2.7.9.3 Examination Techniques and Procedures

The examination procedures used by TVA are performed in accordance with the guidelines of Subarticle IWA-2200 of ASME Section XI.

3.8.2.7.9.4 Inspection Intervals

An inspection schedule for Class MC components will be developed in accordance with Subarticle IWE-2400 of ASME Section XI and the requirements of 10 CFR 55.55a(g)(6)(ii)(B).

3.8.2.7.9.5 Examination Categories and Requirements

The examination categories and requirements for Class MC components will be in accordance with Subsection IWE of ASME Section XI and 10 CFR 50.55a(b)(2)(x) to the extent practicable.

3.8.2.7.9.6 Evaluation of Examination Results

Evaluation of examination results shall be in accordance with Article IWE-3000 of ASME Section XI. Components with unacceptable indications will be repaired or replaced in accordance with Article IWA-4000 of ASME Section XI.

3.8.2.7.9.7 System Pressure Tests

The program for Class MC and metallic liners of Class CC components system pressure tests shall be in accordance with Article IWE-5000 of ASME Section XI.

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(6) "Corrosion," Metals Handbook, Volume 13, ninth edition, Metals Park, Ohio, 1987, pp 82-83.

(7) Lauobe, C. P., "Corrosion of Steel in Marine Atmospheres," Trans. Electro Chemical Society, Volume 87, 1945, pp. 161-182.

(8) Rozenfield, I. L., "Atmospheric Corrosion of Metals," Houston, TX: NACE, 1973, pp 104-106.

(9) Evans, Ulrick R., The Corrosion and Oxidation of Metals: Scientific Principles and Practical Applications. London, Edward Arnold (Publishers) Ltd., 1960, pp. 27-37.

(10) TVA drawings 47E235-44 through 48, "Containment Harsh Environment."

Table 3.8.2-1 Allowable Stress Criteria - Containment Vessel (Sheet 1 of 1)

Material: SA-516, Grade 70	
Loading Conditions	Applicable ASME Code Reference for Stress Intensity[1]
1. Normal Design Condition	NB-3221
2. Normal Operation Condition	NB-3222
3. Upset Operation Condition	NB-3223
4. Emergency Operation Condition	NB-3224
5. Construction Condition	NB-3221
6. Test Condition	NB-3226
7. Post-Accident Fuel Recovery Condition	NB-3224

[1]All references are to the ASME Boiler and Pressure Vessel Code, 1971 Edition, Section III.

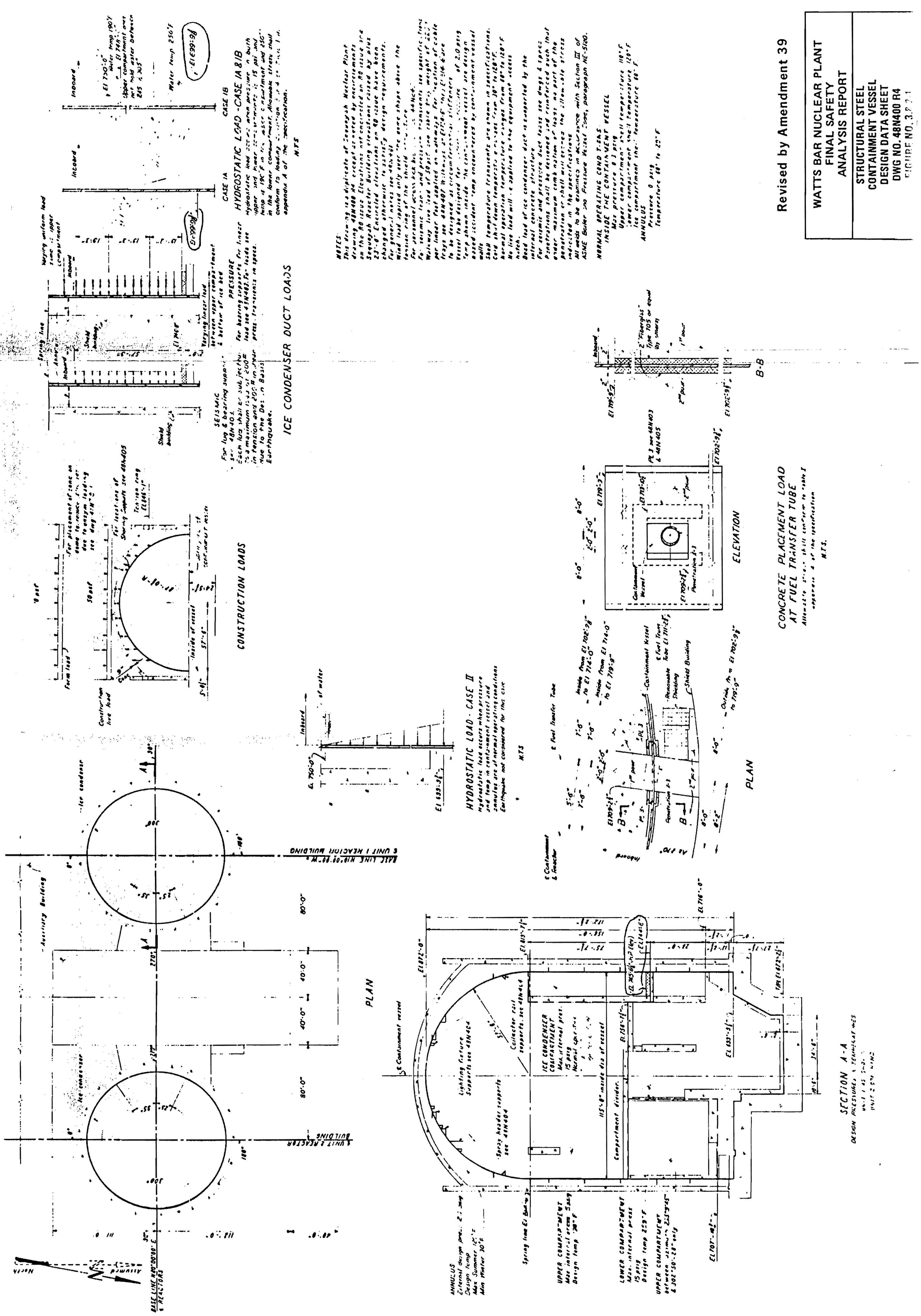


Figure 3.8.2-1 Structural Steel Containment Vessel Design Data Sheet

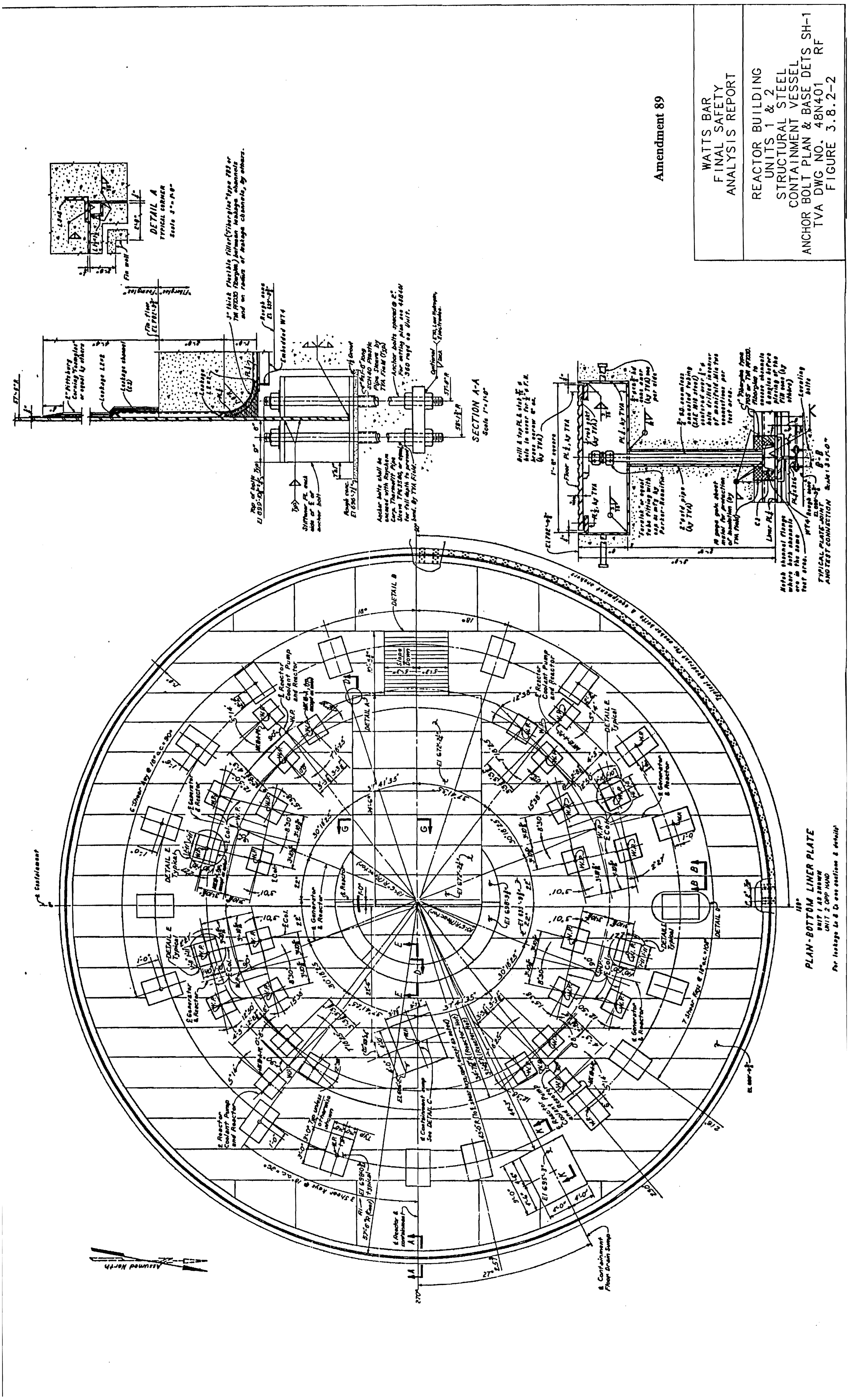


Figure 3.8.2-2 Reactor Buildings Units 1 and 2 Structural Steel Containment Vessel Anchor Bolt Plan & Base DETS - Sheet 1

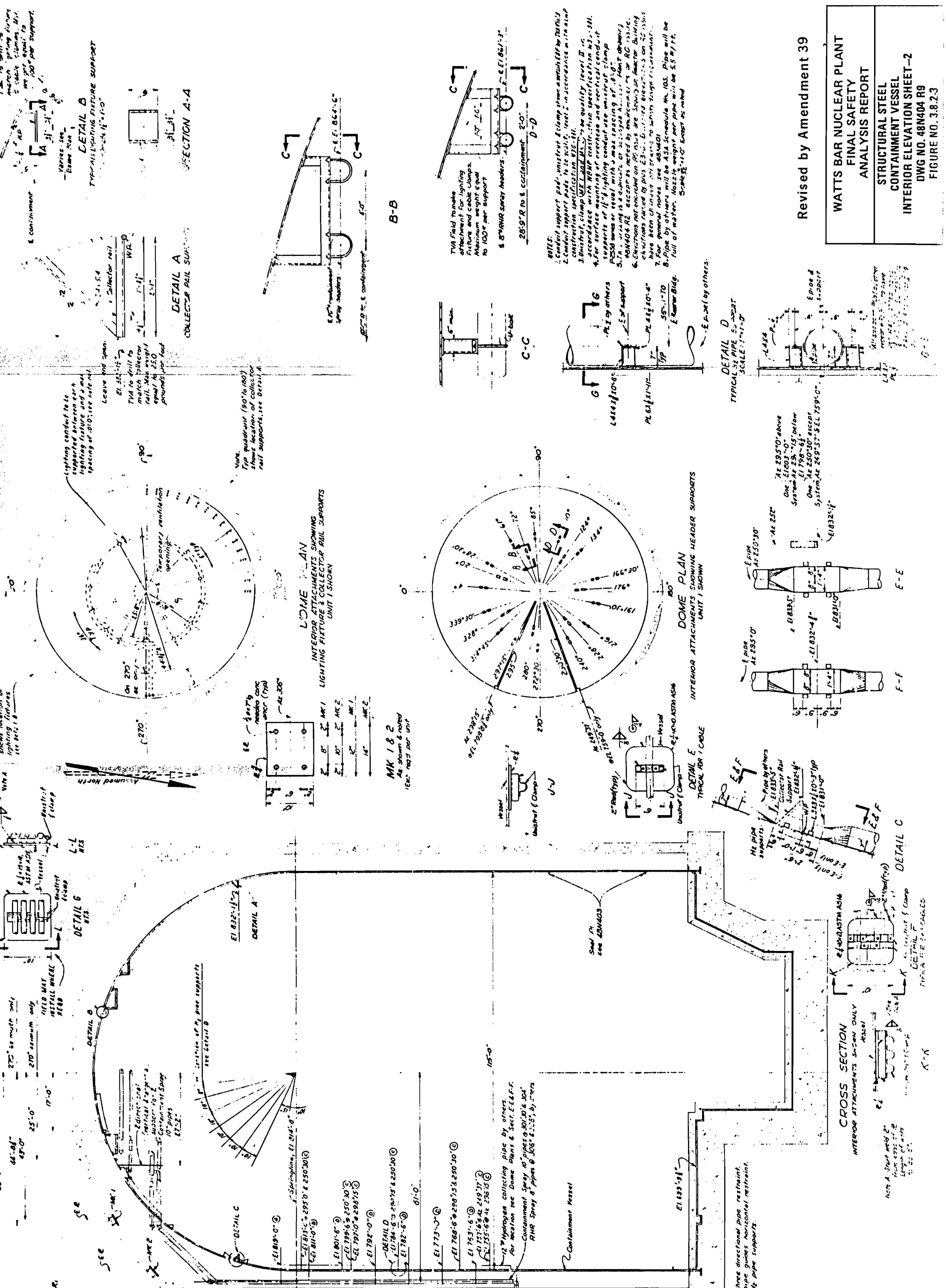


Figure 3.8.2-3 Structural Steel Containment Vessel Interior Elevation - Sheet 2

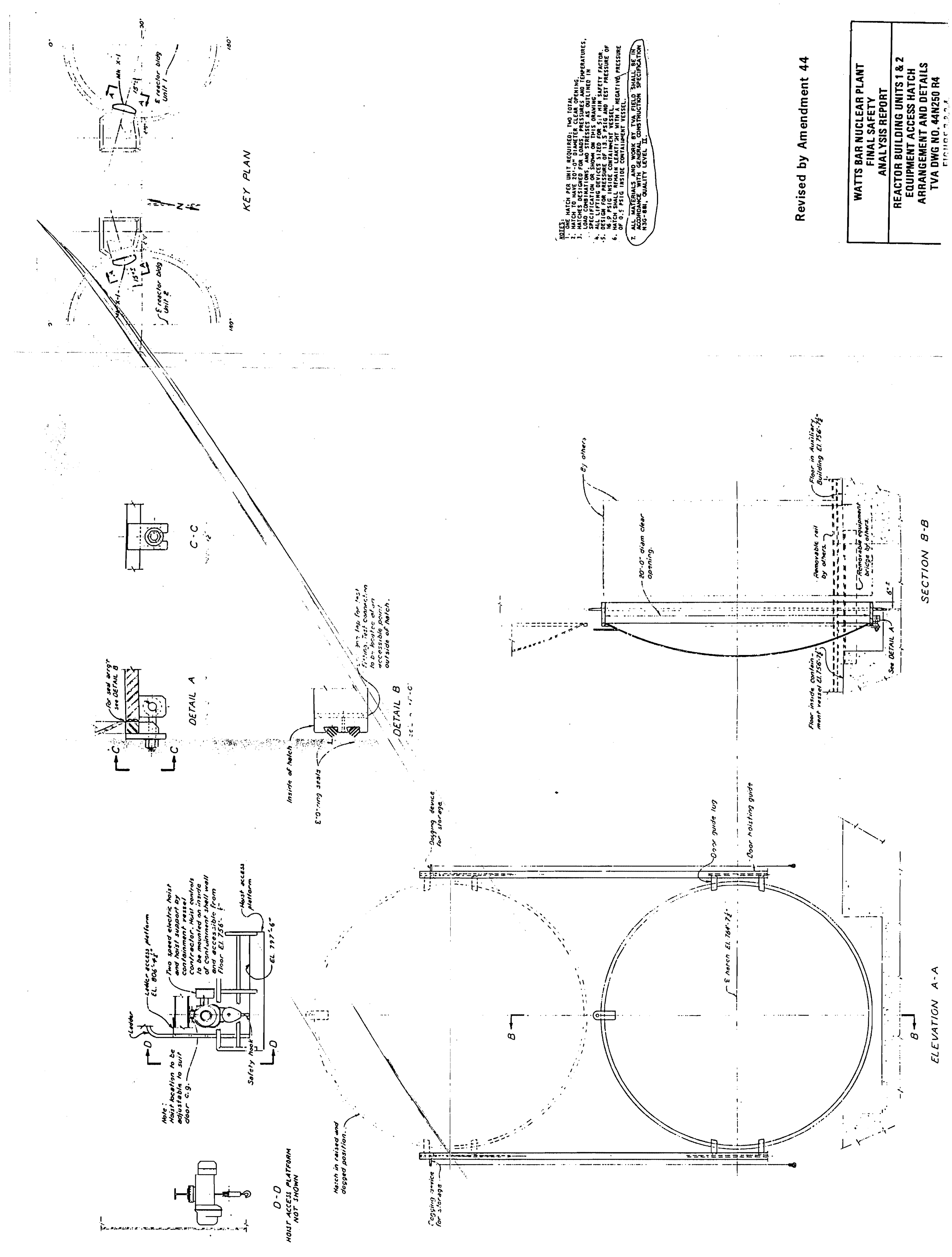


Figure 3.8.2-4 Reactor Buildings Units 1 and 2 Equipment Access Hatch Arrangement and Details

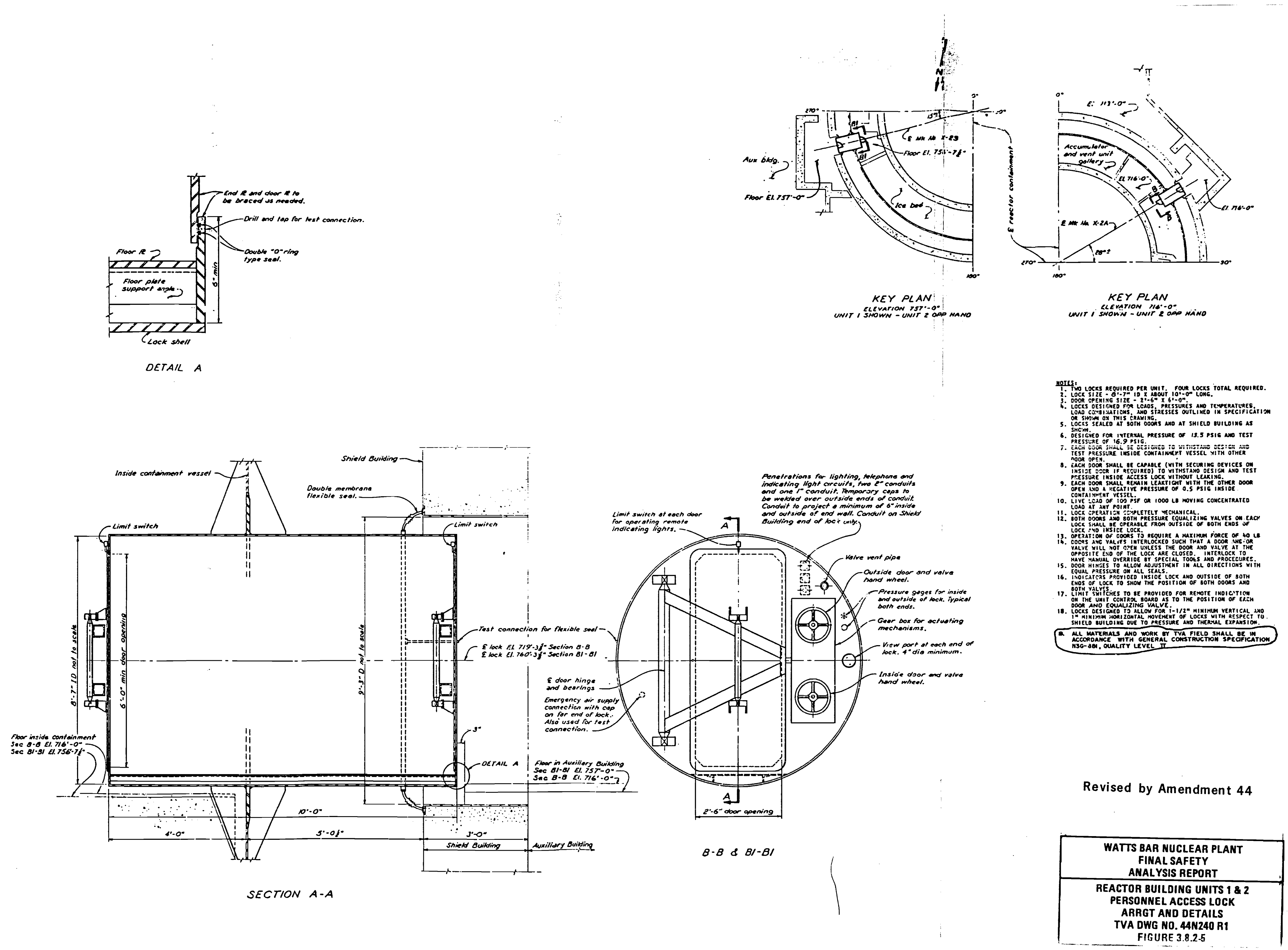


Figure 3.8.2-5 Reactor Buildings Units 1 and 2 Personnel Access Lock Arrangement and Details

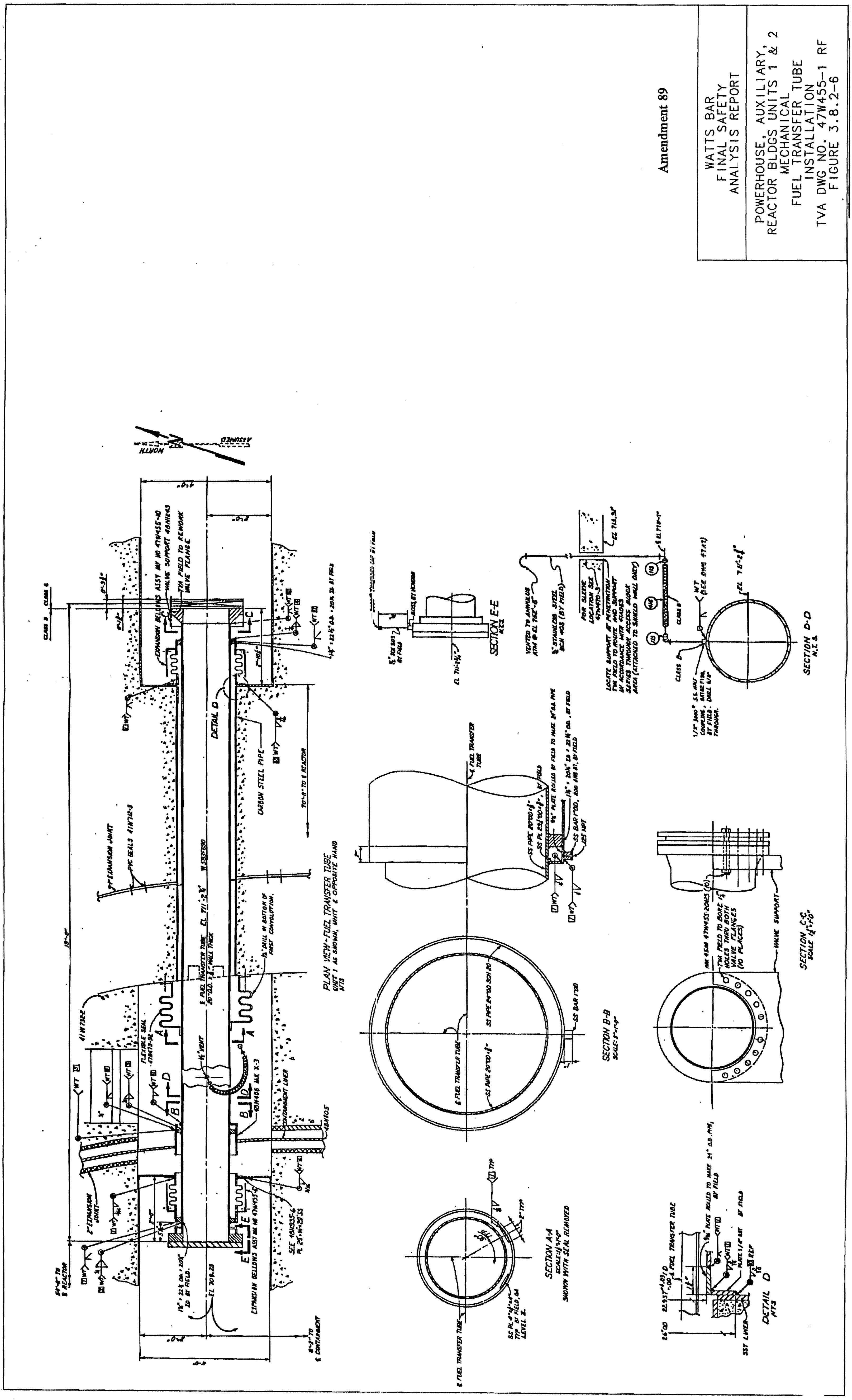


Figure 3.8.2-6 Powerhouse, Auxiliary, Reactor Buildings Units 1 and 2 Mechanical Fuel Transfer Tube Installation

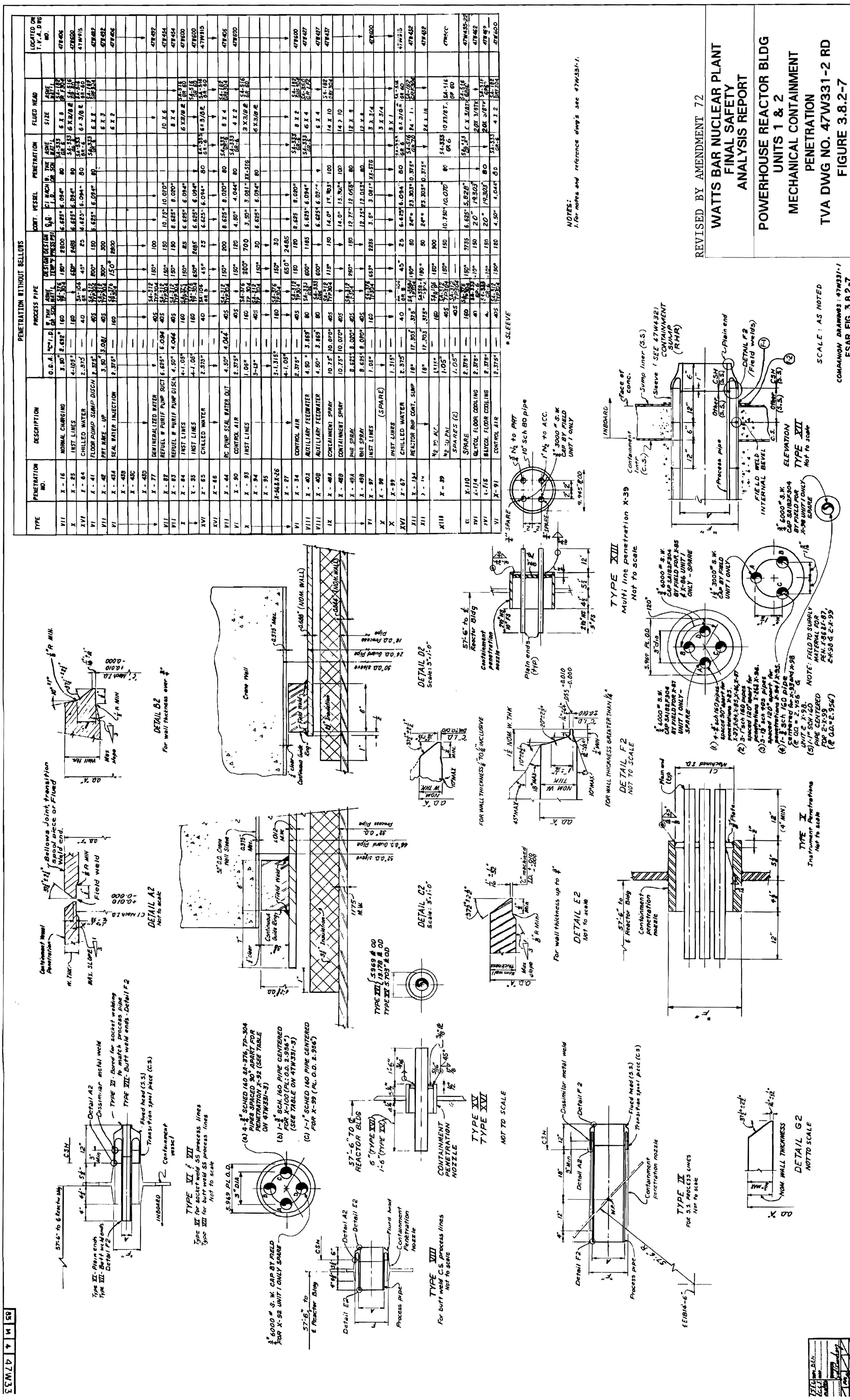


Figure 3.8.2-7 Powerhouse Reactor Buildings Units 1 and 2 Mechanical Containment Penetration

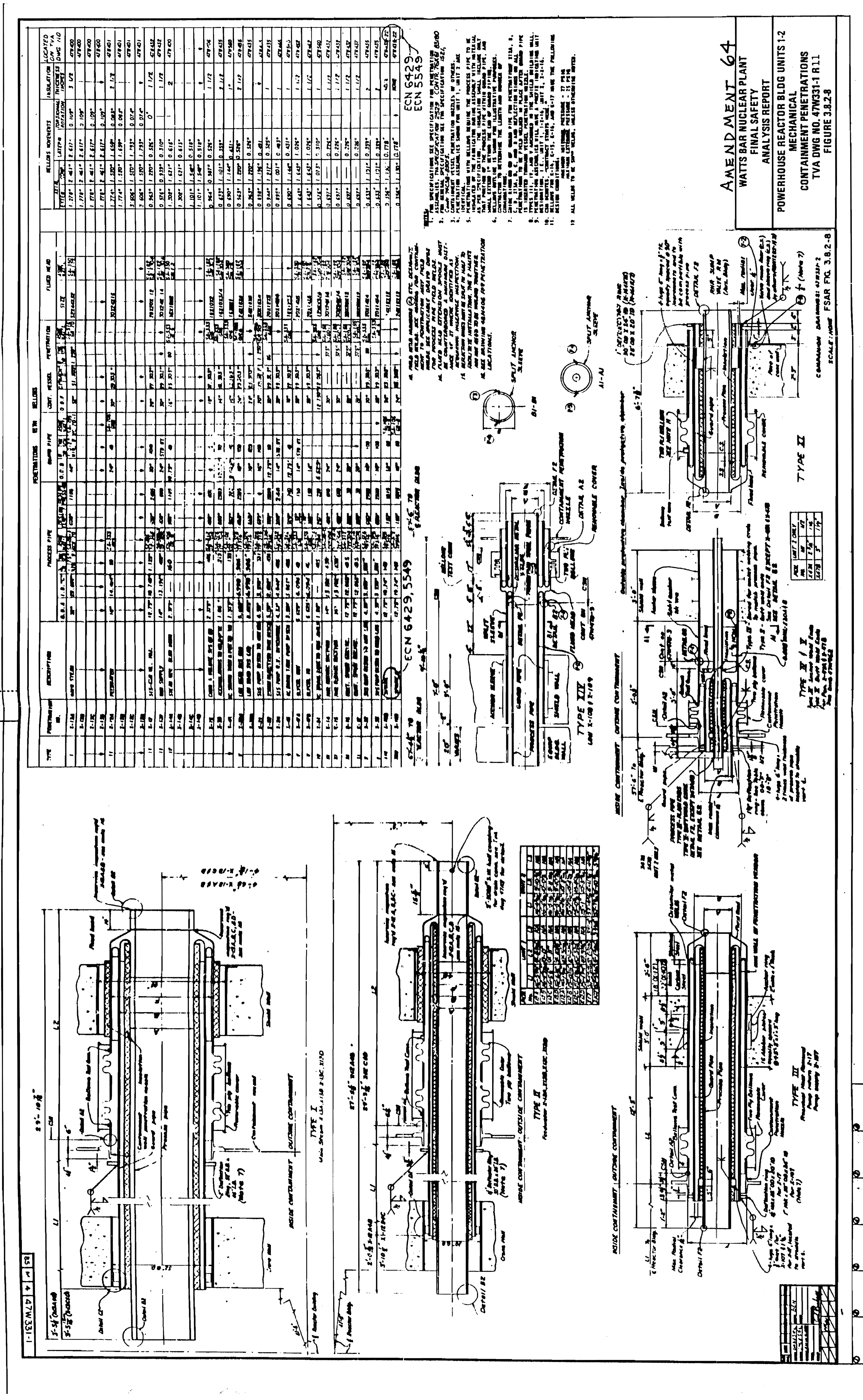
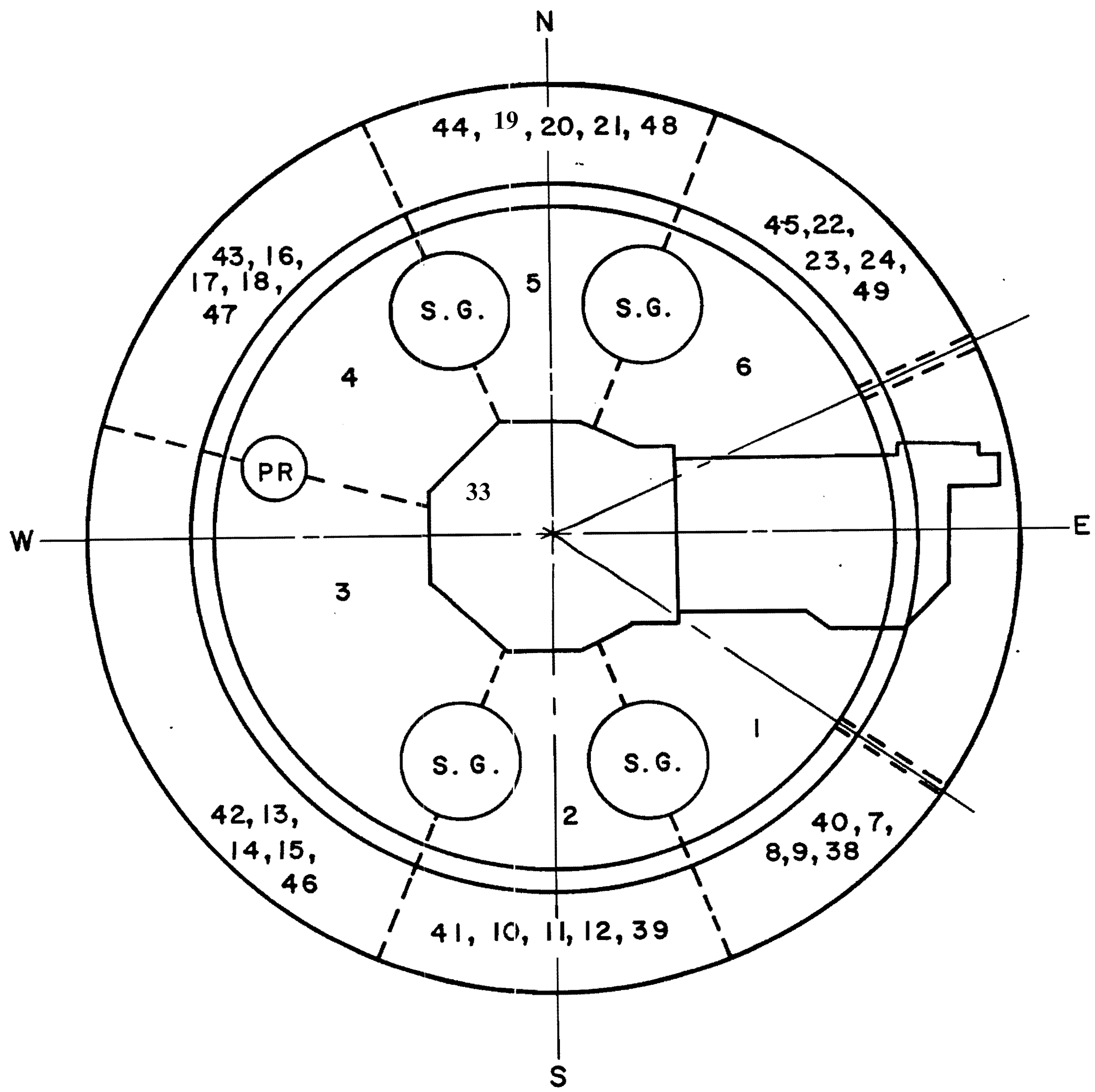


Figure 3.8.2-8 Powerhouse Reactor Building Units 1 - 2 Mechanical Containment Penetrations

Figure 3.8.2-9 Deleted by Amendment 64



WATTS BAR NUCLEAR PLANT FINAL SAFETY ANALYSIS REPORT
TMD NODAL VOLUMES Figure 3.8.2-10

Figure 3.8.2-10 TMD Nodal Volumes

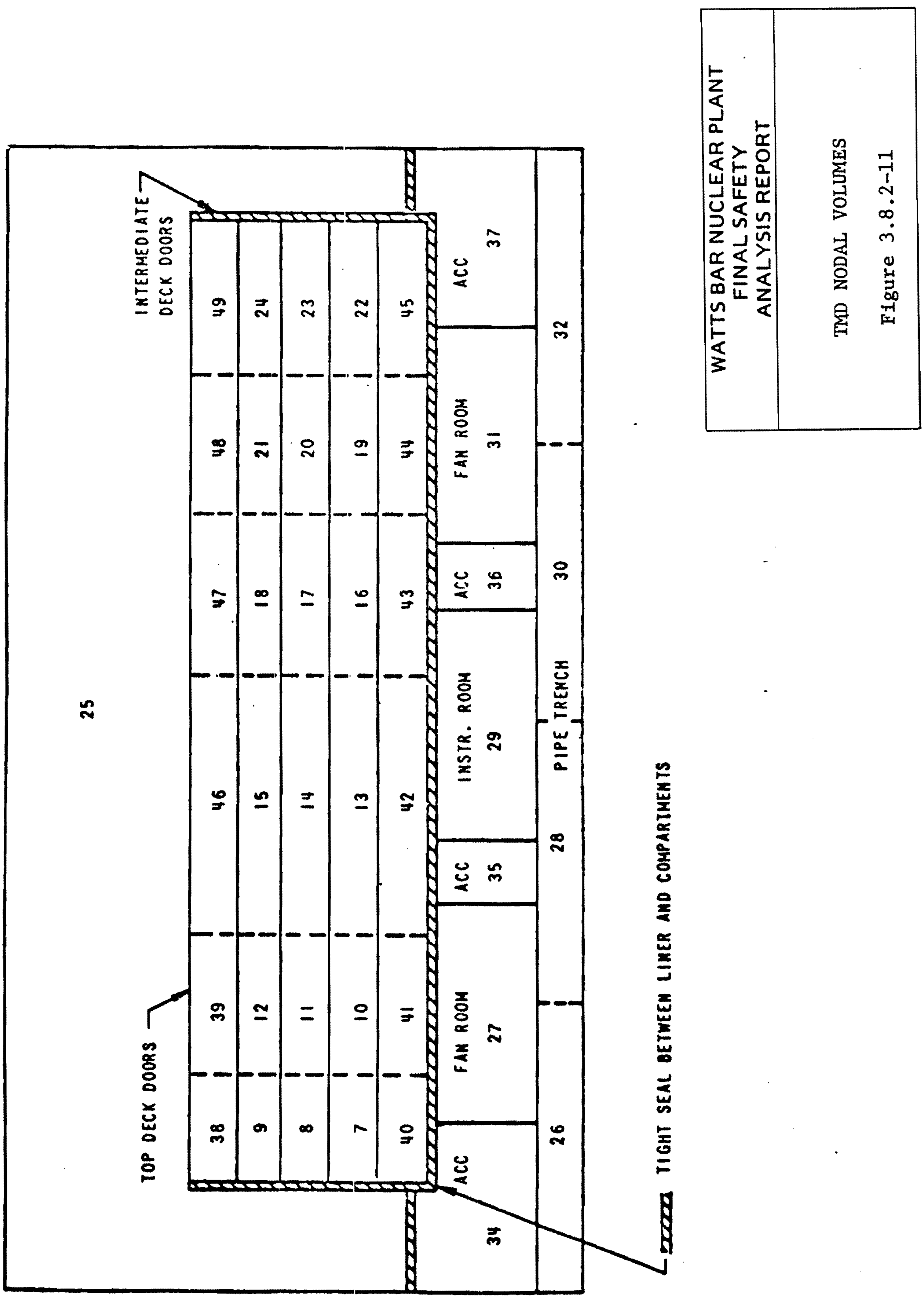
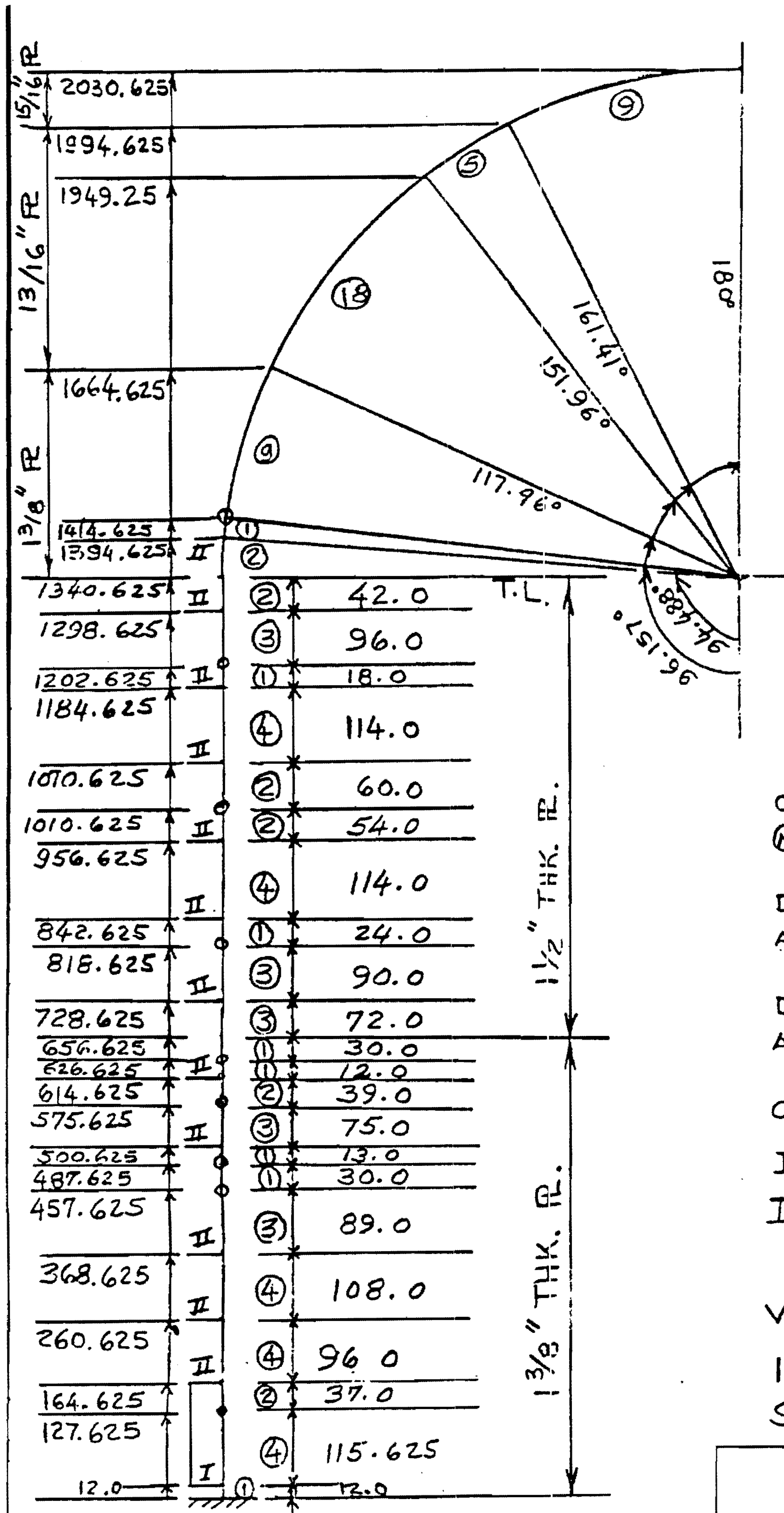


Figure 3.8.2-11 TMD Nodal Volumes



O – PRES. CHANGE LOCATIONS
Ⓝ – NUMBER OF SEGMENTS

DIMENSIONS TO THE RIGHT ARE LENGTHS OF EACH PANEL (IN.)

DIMENSIONS TO THE LEFT ARE DISTANCES ABOVE BASE (IN.)

CIRCUMFERENTIAL STIFFENERS
I PL 10" x 1"
II PL 22" x 1 3/8"

VERTICAL STIFFENER
1" x 8" @ 5°
SPACING (72 AROUND)

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
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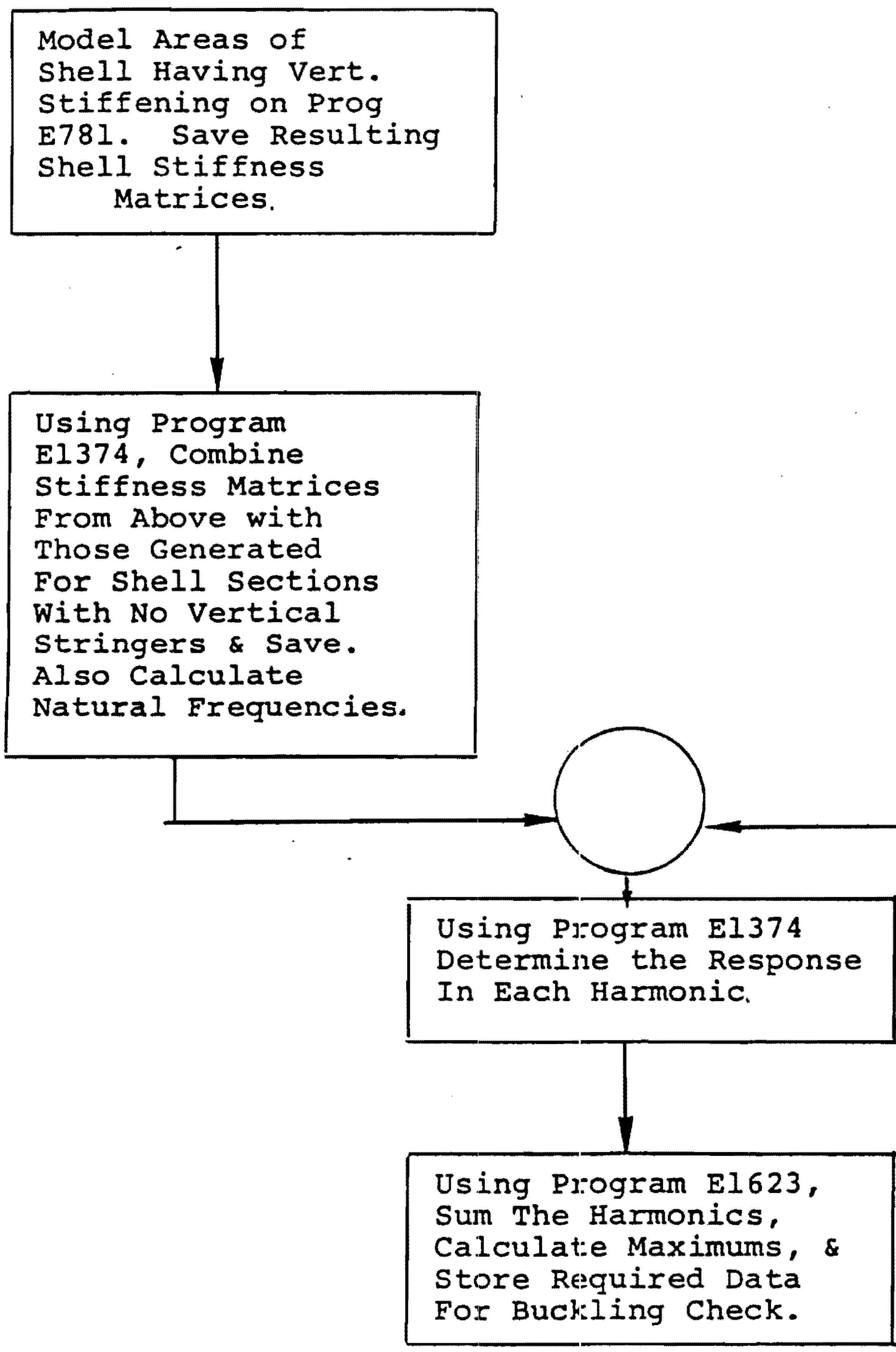
CB&I CONTAINMENT SHELL MODEL

Figure 3.8.2-12

Figure 3.8.2-12 CB & I Containment Shell Model



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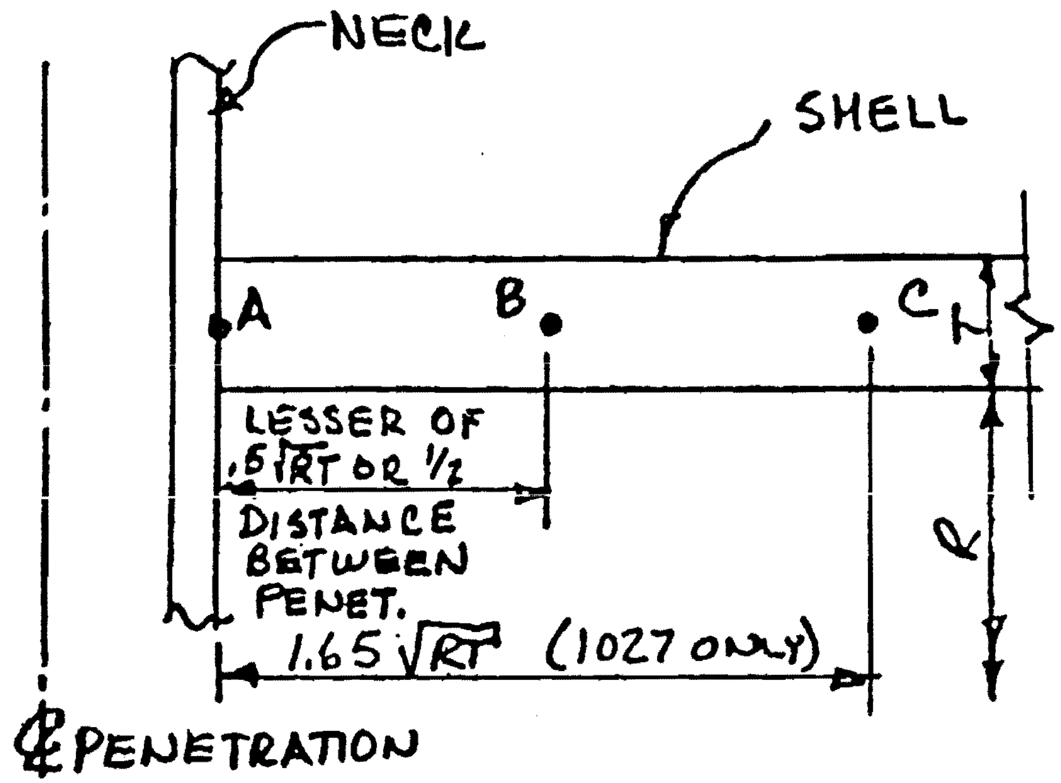
CB&I CONTAINMENT SHELL
ANALYSIS FLOW MODEL
Figure 3.8.2-13

Figure 3.8.2-13 CB & I Containment Shell Analysis Flow Model



POINT		ASSUMED STRESS				ALLOW STRESS
		CIRCUM		MERID		
		CYL	SPH	CYL	SPH	
A	SURFACE	1½ Sm	1½ Sm	1½ Sm	1½ Sm	3 Sm
	MEMBRANE	Sm	Sm	Sm	Sm	1½ Sm
B	SURFACE	1½ Sm	1½ Sm	1½ Sm	1½ Sm	3 Sm
	MEMBRANE	Sm	Sm	½ Sm	Sm	1.1 Sm
C	SURFACE	Sm	Sm	½ Sm	Sm	3 Sm
	MEMBRANE	Sm	Sm	½ Sm	Sm	1.1 Sm

WATTS BAR NUCLEAR PLANT
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STRESS REDUCTION METHOD

Figure 3.8.2-14

Figure 3.8.2-14 Stress Reduction Method

3.8.3 Concrete Interior Structure

The concrete interior structures are designed as described in Sections 3.8.3.1 through 3.8.3.8.

The evaluation and the modification of the interior reinforced concrete structures are optionally done using the ultimate strength design method in accordance with the codes, load definitions, and load combinations specified in Appendix 3.8E.

3.8.3.1 Description of the Interior Structure

3.8.3.1.1 General

This structure, shown in Figures 3.8.3-1 through 3.8.3-7, is a complex assemblage of reinforced concrete walls, slabs, and columns housed inside the SCV. It will act as a temporary containment while routing steam to and through the ice condenser in the event of a LOCA. The reactor, four steam generators, four reactor coolant pumps, pressurizer, ice condenser, reactor instrumentation, air-handling equipment, and various other support systems are located inside this structure.

The portion of this structure which separates the upper compartment from the lower is defined as the divider barrier (See Figure 3.8.1-1). The failure of any part of the divider barrier is considered critical since it would allow LOCA steam to bypass the ice condenser, thereby increasing the pressure within the steel containment. For this reason the divider barrier is designed more conservatively than the rest of the internal structure.

Since the ice condenser is both a structure and an engineered safeguard system, most detail information can be found in Section 6.5.

3.8.3.1.2 Containment Floor Structural Fill Slab

The containment floor slab is a reinforced concrete slab of 3-foot nominal thickness cast on top of the bottom liner plate. Reinforcement is provided in both faces to withstand uplift pore pressure below the liner plate and to develop restraint for uplift and rotational moments at the base of the crane wall. Earthquake shearing forces are transmitted to the base slab through shear keys located below the crane wall, through a direct tie with the reactor cavity, and through direct bearing on the base of the Shield Building wall as a result of the expanded volume of the fill slab under operating temperatures. Stresses resulting from shear forces are very low since any one of the three methods is capable of transmitting the entire shearing force.

The interior concrete structure is sufficiently keyed to the reactor cavity by its configuration of walls and slabs to provide base stability against earthquake overturning moments above the bottom liner plate at Elevation 699.28. In addition, the anchorages for the steam generators and reactor coolant pumps supports tie the containment structural fill slab to the base slab in the vicinity of the crane wall providing additional stability.

3.8.3.1.3 Reactor Cavity Wall

This 17-foot inside diameter circular wall supports and encloses the reactor vessel above the lower reactor cavity. The wall is 8-1/2 feet thick, primarily for radiation shielding and structural requirements due to the reactor support loads, and it extends from the base slab at Elevation 702.78 to Elevation 714.96 where it intersects the refueling canal floor slab. Neutron detector windows reduce the effective structural thickness to 6 feet for approximately the first 10 feet of height. The next 12 feet of height has only a 4-foot, 3-inch structural thickness due to the 3-foot, 1-inch wide by 6–foot, 6-inch high inspection cavity which surrounds the reactor vessel. Between the inspection cavity and reactor vessel is a 14-inch-wide structural wall. This is shown in Figures 3.8.3-7a through 3.8.3-7g.

14-Inch Reactor Cavity Bulkhead Wall

This wall consists of two interconnected concentric cylindrical steel shells, separated by 11 inches of concrete fill, that form the reactor cavity from Elevation 721.625 through a composite steel diaphragm to the concrete wall at radius 13.0. This is shown on Figures 3.8.3-7e through 3.8.3-7g.

The anchorage for the wall at Elevation 715.04 is the reactor support embedments directly below the wall.

Both the 14-inch wall and the diaphragm are designed to withstand the pressure and temperature transients resulting from a LOCA condition in accordance with the required factored loading combination in Table 3.8.3-1. The peak differential design pressure and temperature are 50 psi and 150° F, respectively.

3.8.3.1.4 Compartment Above Reactor

This compartment is approximately a 270° arc continuation of the reactor cavity wall. The ends of the arc intersect the two refueling canal side walls. The inside diameter of the wall is 26 feet and the thickness is 4 feet. It extends from approximately Elevation 725.15 to the bottom of the divider deck slab at Elevation 754.13. This compartment is vented to the lower compartment area, outside the wall, by six windows which reduce the wall to five columns. These columns each have a cross sectional area of 12 square feet and extend the last 4-1/2 feet of height to the bottom of the divider deck slab. This compartment is shown in Figure 3.8.3-7b.

During reactor operation this compartment is sealed across the top by the concrete and steel missile shield and is sealed across the refueling canal by a concrete and steel gate.

Seals Between Upper and Lower Compartments

The seals extend across the gap between the inside surface of each steel containment vessel and the concrete structure within each vessel. They are located along the bottom of the concrete floor under the ice condenser, at Elevations 739.5 and 751.33

between the ends of the ice condenser and the refueling canal concrete structure, and along the vertical sides of the refueling canal structure. These seals form part of the barrier between the upper and lower compartments of the containment vessels.

The seals consist of long strips of flexible elastomer coated fabric folded longitudinally with open edges butted and sewn to form two loops in cross section. Metal bars are inserted into the seal for use during attachment. These strips are field-spliced with vulcanized overlay joints or cold bond overlay to form a continuous seal.

The seals are attached to the containment vessel and the interior concrete structure using bolted clamps with bolts spaced one foot apart. These clamps grip the metal bars inserted in the seal thereby closing and sealing the gap.

These seals form part of the barrier between the upper and lower compartments of the containment vessels. During normal operating conditions, the seals prevent airflow around the ice condensers. In an accident, the seals and the other divider parts limit the amount of hot gases, steam, and vapor that can bypass the ice condensers. The seals will maintain their integrity for the first 12 hours after an accident. A small amount of leaking during this period is permissible.

The seals will maintain their integrity during earthquake conditions and effectively maintain their air seal. The seals will function effectively in a post-earthquake condition. The slack in the coated fabric seals, which was purposely provided, allows for the relative movement, between the containment vessel and the interior concrete structure, which results from earthquakes.

3.8.3.1.5 Refueling Canal Walls and Floor (Divider Barrier)

These irregular shaped walls and slabs vary in thickness and enclose an area approximately 19 feet by 32 feet. This area will be filled with water along with the compartment above the reactor during refueling operations. The water level will be about 35 feet above the canal floor slab. The reactor internals will be removed and stored in the refueling canal during refueling. Refueling canal walls and floor are shown in Figure 3.8.3-6.

3.8.3.1.6 Crane Wall

This basically 3-foot-thick, 117-foot-high cylindrical wall encloses an 83-foot inside diameter area containing the reactor, reactor coolant pumps, steam generators, and reactor coolant piping. The crane wall is 4 feet thick in some areas to satisfy structural requirements. There are four localized areas of "bumps" on the wall which are 4 feet thick. These "bumps" begin at the floor slab at Elevation 702.78 and extend to the Elevation 716.0 and are approximately 25 feet in arch length. The crane wall is also 4 feet thick between Elevation 737.42 and Elevation 756.63 over its entire circumference. This wall acts as the major support for the divider barrier slabs and walls. It also supports the floors and walls in the 13-foot annulus between it and the steel containment vessel (SCV). The 175-ton polar crane is mounted on top of this wall. Over the refueling canal the wall has a section removed leaving a curved beam 23 feet deep spanning an arc length of 41 feet between ice condenser compartment end walls. Beginning at Elevation 746.42 the crane wall has twenty-four 7-foot, 4-inch high by 6-foot, 8-inch openings for the ice condenser inlet doors. The remaining wall consists of 25 columns each having a 10-square-foot cross section. Above the operating deck floor at Elevation 756.63, the crane wall is part of the divider barrier. It is also part of the pressurizer and steam generator compartments, which constitute part of the divider barrier, and is designed to resist the same pressures as these compartments. At the top of the crane wall the steel support beams for the ice condenser bridge crane cantilever over the ice beds causing moments and forces in the crane wall. Lateral seismic loads from the ice beds are transmitted to the outer face of the crane wall.

Personnel Access Doors in Crane Wall

See Figures 3.8.3-8 through 3.8.3-11.

Four access doors in the lower half of the crane wall are provided in each Reactor Building at the following locations:

Floor Elevation	Azimuth
702.78	221°
702.78	299°
716.0	114°16' 11"
745.0	299°

The doors provide passageways 3-feet wide by 6-feet, 6-inch high through the concrete crane wall for workmen and tools. When closed, the doors seal the passageways against steam jets, pressure, and missiles that may originate from pipe rupture in the compartment inside the crane wall.

Each door is manually operated and hinged to a steel frame embedded in the concrete wall. Each door consists of a steel skin plate stiffened by horizontal framing. The skin plate is faced with a cushioning structure of vertically arranged square, steel tubing separated from the doors skin plate by a collapsible latticework of steel bars, the purpose of which is to absorb the energy of missiles striking the door. The cushioning structure is covered with sheet steel for appearance. Bearing of the door against the frame is through steel bars. An elastomer seal is attached to the periphery of the door to reduce the possibility of damage from jets to items beyond the door. Two lever-type latches operable from either side hold the door in the closed position. Hinges on the doors are provided with graphite impregnated bushings.

The doors, under normal operating conditions, provide an effective seal against airflow and can be operated and secured manually from either side. For pipe rupture accidents, the doors seal the passageways in the crane wall against missiles, jets and pressure that may originate within the crane wall enclosure, thus preventing consequent damage to the containment vessel and to piping and machinery between the crane wall and containment vessel.

The doors will maintain their integrity and seal for not less than the first 12 hours following an accident. Limited leakage during this period is permissible.

All parts of the doors, except the seals, are fireproof. Increased leakage may occur during a fire. It is assumed that a fire and an accident which require sealing will not occur simultaneously since the reactors will be shut down immediately if a fire develops.

3.8.3.1.7 Steam Generator Compartments (Divider Barrier)

Two double-compartment structures house the four steam generators in pairs on opposite sides of the building. Each structure consists of curved and straight sections of walls that vary in thickness from 2-1/2 to 4 feet. Divider barrier walls around two steam generators extend 42 feet up from the divider floor and are capped with a 3-foot-thick slab spanning over the steam generators from the crane wall. A wall between the two steam generators extends from the divider barrier walls to the crane wall, completing the double compartment. The center wall extends only 32-1/2 feet above the floor. The area above the top of this wall, except for that occupied by a beam acting as a barrier for a postulated break in a pipe, will reduce the compartment pressure buildup in a single compartment by venting the steam to the other compartment. See Figures 3.8.1-1 and 3.8.3-6.

3.8.3.1.8 Pressurizer Compartment (Divider Barrier)

This compartment separates the pressurizer from the upper compartment. Its walls project about 38 feet above the Elevation 756.63 floor where they are capped with a 3–foot-thick slab. It is similar to the steam generator compartments except its wall thickness varies from 2 to 3 feet and the volume is much smaller. See Figure 3.8.3-6.

3.8.3.1.9 Divider Deck at Elevation 756.63 (Divider Barrier)

This 2-1/2-foot-thick irregular shaped floor is the major divider barrier between upper and lower compartments. It is supported at its outer edges by the crane wall and the compartment walls for the steam generators and pressurizer. Support near the center of the building consists of the refueling canal walls and the five columns of the upper reactor compartment. This floor contains five hatches for equipment removal. The concrete covers on these hatches are designed for the same loadings as the floor. The floor outline is shown in Figure 3.8.3-3.

3.8.3.1.10 Ice Condenser Support Floor - Elevation 744.5 (Divider Barrier)

This floor extends 12-feet, 8-inches from the outside of the crane wall to the 4-inch expansion joint separating it from the steel containment vessel. A circumferential beam under its outer edge is cast with the floor. This edge beam is supported by concrete columns which extend down through the Elevation 716 floor to the fill slab at Elevation 702.78. The floor extends 300° around the outside of the crane wall between the ice condenser end walls at azimuths 245° and 305°, as shown in Figure 3.8.3-1.

3.8.3.1.11 Penetrations Through the Divider Barrier Canal Gate

The canal gate consists of three removable concrete wall elements as illustrated on Figures 3.8.3-2 and 3.8.3-4. The elements are 2-feet, 6-inches thick and span between 7-inch-deep slots formed in the walls of the refueling canal.

Control Rod Drive (CRD) Missile Shield

The CRD missile shield consists of three removable concrete slabs as illustrated on Figures 3.8.3-3 and 3.8.3-4. The slabs are 3-feet, 6-inches thick and are anchored to the divider barrier slab at Elevation 756.63 by anchor bolt assemblies. The details of the anchor bolt assemblies are shown on Figure 3.8.3-14.

Reactor Coolant Pump Access and Lower Compartment Access

Access to the reactor coolant pumps and lower compartment is provided by removable slabs as illustrated on Figure 3.8.3-6. The reactor coolant pump access slabs are approximately 10 feet in diameter and the lower compartment access slab is approximately 6 by 10 feet. Both are 2-feet, 6-inches thick and are anchored to the divider barrier slab by anchor bolt assemblies around the edges. The details of the anchor bolt assemblies are shown on Figure 3.8.3-17.

Equipment Access Hatch

This hatch consists of a removable structural steel framed hatch cover and a structural steel support frame adjacent to the containment vessel. The arrangement and details are illustrated on Figures 3.8.3-6 and 3.8.3-18. The support frame and hatch cover consist of structural steel wide flange sections covered with steel plate. To provide adequate seals between the upper and lower compartment, the side of the frame adjacent to the containment vessel was designed to span from the refueling canal wall to the divider barrier slab, a distance of approximately 5.0 feet. The hatch cover is anchored to the concrete structure by anchor bolt assemblies at each end of the cover.

Escape Hatch

The location of the hatch and the details are shown on Figure 3.8.3-12. The hatch consists of a frame embedded in the divider barrier floor with a hinged and manually operated cover consisting of skin plate stiffened by framing. Quick-acting wheels are provided for opening and closing the cover from either side. Coil springs are incorporated with the hinges to reduce the force required for opening the cover. The hatch is equipped with a limit switch which operates to give an indication in the control room of the position of the hatch cover.

Air Return Duct Penetration

The air return ducts penetrate the divider barrier at two different locations as indicated on Figures 3.8.3-2 and 3.8.3-3. One penetration is at Elevation 746.0 and the other is at Elevation 756.63. The penetrations are 4-foot, 6-inch (inside diameter) circular openings with flanges on both sides to provide attachment for the ventilating ducts. The details of the penetrations are shown on Figures 3.8.3-15 and 3.8.3-16.

Pressurizer Enclosure Manway

The location and details of the manway are shown of Figure 3.8.3-20. The manway consists of a 30-inch diameter sleeve embedded in concrete at elevation 798.0 at the top of the pressurizer compartment. The manway cover is a circular steel plate that is bolted in place to provide adequate sealing between the upper and lower compartment.

3.8.3.2 Applicable Codes, Standards and Specifications

Structural design of the interior concrete structures is in compliance with the ACI 318–71 Building Code Requirements for Reinforced Concrete, and ACI-ASME (ACI 359) Article CC 3000 document, "Standard Code for Concrete Reactor Vessels and Containments."

Reinforcing steel conforms to the requirements of ASTM Designation A615, Grade 60.

Historical Information Installation, inspection and testing requirements for plain and reinforced concrete used in the construction of Category I structures, as well as for fly ash used as an admixture in concrete, were in general accordance with the ASTM standards, ACI 318-71, ANSI N45.2.5 and Regulatory Guides 1.15 and 1.55, except for the following TVA specific requirements:

(1) Historical Information - Required Qualification Tests

(a) Fly ash - TVA uses its own specification for fly ash rather than ASTM C 618. Significant differences occur in requirements for fineness, pozzolanic activity index, and loss or ignition. ASTM C 618 has two requirements for fineness. The first, a surface area obtained by an air permeability apparatus, is not conformed to by TVA. The second, the amount retained on a No. 325 sieve, is conformed to by TVA. TVA's requirement for pozzolanic activity index with portland cement is 65% of the ASTM C 618 requirement, but TVA's procedures result in substituting fly ash for fine aggregate in a mix and thus increase the quantity of fly ash available for reaction with the cement. TVA's limit on loss on ignition is 50% of that in ASTM C 618. The most recent addition to ASTM C 618 is a limit on the product of loss on ignition and the amount retained on the No. 325 sieve. This was added when a statistical analysis indicated that it correlated with the effect of fly ash in concrete. TVA's limits on the individual items will result in conformance to the ASTM C 618 limit on the product. TVA's experience at hydro, fossil, and nuclear plants indicates that their specification for fly ash produced acceptable concrete.

(b) Water and ice - TVA complies with the suggested limits of CRD C 400 rather than the suggested limits of AASHO T-26. The suggested limits on compressive strength of mortar are the same. AASHO T-26 utilizes an autoclave soundness test developed specifically to test free lime or magnesia in cement. ASTM C 150 has a specified limit on autoclave

expansion of 0.8%, but many elements exhibit less than 0.1%. The ASTM analysis for precision indicates that repeat tests by the same operator can differ 21% by their means. This illustrates that a clear indication of unsoundness due to water will be difficult to obtain. The test for soundness is not recommended in ASTM STP 169A "Significance of Tests and Properties of Concrete-Making Materials."

(c) Concrete mixes - TVA does not conform with two of the recommendations of ACI 211. The recommended limiting water-cement ratios were developed for Type 1 cements. ACI 211 recommendations do not agree with ACI 318. Where fly ash is utilized, neither can be directly applicable. The recommendation that trial batches for strength be made at maximum slump and air contents should not be applied where statistical analysis establishes an average over-strength requirement. Use of maximum air content and slump will offset the average strength and invalidate the analysis.

(2) Historical Information - Required Inprocess Tests

(a) The construction procedure for this project is in substantial agreement with ANSI N45.2.5 frequencies for those tests required by TVA.

(b) Mixer uniformity - TVA's requirements for unit weight of air-free mortar and for coarse aggregate content are more restrictive than ASTM C 94.

(c) Compressive strength - The sampling frequency for compressive strength provided by ANSI N45.2.5 appears to be intended for a transit mix operation. TVA purchase specifications for ready-mix are even more restrictive, however, the vast majority of TVA concrete is produced in a central mix plant where the provided frequency appears excessive. TVA varied the testing frequency requirements based on the specified strength of concrete with no one sample to represent more than:

– 300 cubic yards for a specified strength of 2000 psi.

– 175 cubic yards for specified 28 day strengths of 3000 psi or more.

These test frequencies were in effect until January 1978 during the majority of concrete placement at WBN. The testing frequency requirements were then modified such that no one sample represents more than:

– 300 cubic yards for a specified strength of 2000 psi,

– 200 cubic yards for a specified strength of 3000 psi,

– 150 cubic yards for specified strengths more than 3000 psi.

For actual application, the quantities of each mix produced per shift were such that the average quantities represented by test samples were less than that specified.

(d) Aggregate - Tests are specified by ANSI N45.2.5 which appear inappropriate to certain aggregates. A carefully selected crushed limestone fine aggregate should not require testing for organic impurities. TVA required periodic reinspection of the quarry. The quarry strata and weathering effects did not change and therefore testing listed with 6month frequency in ANSI N45.2.5 were not repeated.

(e) Water and ice - (See 1.b above) The chemical tests in CRD C 400 were repeated every 2 months, and any time a change in the water was suspected. The strength test was repeated only when chemical tests results changed significantly.

(f) Fly ash was sampled every 3 truck loads and tested for fineness. Six samples were combined and tested for total requirements, see 1.a above.

(g) Cement - TVA accepted manufacturers' mill tests which represented no more than 400 tons. TVA made tests at greater intervals which checked manufacturers' strength test within 600 psi or duplicate tests were required.

(3) Historical Information - TVA's concrete acceptance does not conform to ACI 318. It does conform to ACI 214. TVA requires that no more than 10% of the strength test results be below the specified strength for specified strengths equal to or greater than 3,000 psi. For lower strength concrete, 20% of the strength test results may be below the specified strength. Such concrete is used where a batch of somewhat lower strength concrete is not critical and where hydration temperature limitations are critical. ACI 318 applies the criteria that the averages of all sets of three consecutive strength test results at least equals the specified strength and that not more than 1 of 100 strengths test results will be more than 500 psi below the specified strength. If the standard deviation of the strength test results is 500 psi, the required overstrengths from the three criteria range between 640 psi and 670 psi. TVA does not believe that the three criteria produce significantly different results. ACI 318 states that acceptability is based on no strength test result being more than 500 psi below the specified strength, but its commentary and ACI 359 point out the probability that 1 test in 100 will have results outside the standard deviation and make the ACI criteria more severe.

TVA's requirement for regular compressive strength tests at 3 days and thorough evaluation requirements if the tested concrete strength deviates from the specified limits provide reasonable assurance that the use of low strength concrete in structures is effectively prevented.

(4) Historical Information - Personnel qualifications will be maintained as required by Nuclear Quality Assurance Plan [1].

TVA considers the applicability of ANSI N45.2.6 (Section 1.1, Scope) to be limited to those personnel performing inspection, examination, and test functions. Responsibility for examination and certification of these individuals has been established. These certifications do not correspond to the levels established in ANSI N45.2.6, except for NDE personnel who are certified in accordance with SNT-TC-1A. Construction site inspection, examination, and testing personnel are selected and assigned mechanical, electrical, instrumentation, civil, material, or welding classifications. Responsible supervisors in the respective areas perform the functions identified in Table 1 as L-III in ANSI N45.2.6. Inspection, examination, and testing personnel in the various classifications perform the functioning identified in Table 1 as L-I and L-II in ANSI N45.2.6.

Bolts-Anchors set in hardened concrete were installed in accordance with comprehensive TVA specific requirements developed for the material, installation and testing of these anchors, utilizing anchor manufacturer's instructions as applicable.

Welding, non-destructive examinations, heat treatment, and all field fabrication procedures used during construction were in accordance with the ASME Boiler and Pressure Vessel Code as applicable (see Item 3 below), and the American Welding Society, (AWS) "Structural Welding Code," AWS D1.1 (see Item 5 below). Nuclear Construction Issues Group documents NCIG-01 and NCIG-02 (see Section 3.8.1.2, Item 12) may be used after June 26, 1985, to evaluate weldments that were designed and fabricated to the requirements of AISC/AWS. Unless otherwise indicated in the FSAR, the design and construction of the interior structures are based upon the appropriate sections of the following codes, standards, and specifications. Modifications to these codes, standards and specifications are made where necessary to meet the specific requirements of the structures. Where date of edition, copyright, or addendum is specified, earlier versions of the listed documents were not used. In some instances, later revisions of the listed documents were used where design safety was not compromised.

(1) American Concrete Institute (ACI)

ACI 214-77	Recommended Practices for Evaluation of Strength Test Results of Concrete
ACI 315-74	Manual of Standard Practice for Detailing Reinforced Concrete Structures
ACI 359	Standard Code for Concrete Reactor Vessels and Containments (Proposed ACI-ASME Code ACI-359 (Article CC-3300) as issued for trial use April 1973.
ACI 318-71	Building Code Requirements for Reinforced Concrete

ACI 347-68	Recommended Practice for Concrete Formwork
ACI 305-72	Recommended Practice for Hot Weather Concreting
ACI 211-1-70	Recommended Practice for Selecting Proportions for Normal Weight Concrete
ACI 304-73	Recommended Practice for Measuring Mixing, Transporting, and Placing Concrete

(2) American Institute of Steel Construction (AISC) 'Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings,' adopted February 12, 1969.

(3) American Society of Mechanical Engineers (ASME), Boiler and Pressure Vessel Code Sections II, III, V, VIII, and IX, 1971 Editions, as amended through summer 1972 Addenda.

(4) American Society for Testing and Materials (ASTM), 1975 Annual Book of ASTM Standards.

(5) American Welding Society (AWS)

"Structural Welding Code, AWS D1.1-72, with Revisions 1-73 and 2-74 except later editions may be used for prequalified joint details, base materials, and qualification of welding procedures and welders.

Visual inspection of structural welds will meet the minimum requirements of Nuclear Construction Issues Group documents NCIG-01 and NCIG-02 as specified on the design drawings or other engineering design output. See Item 14 below.

AWS D12.1-61, 'Recommended Practice for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Connections.'

(6) Crane Manufacturers Association of America, Inc. C.M.A.A. No. 70, Specification for Electric Overhead Traveling Cranes, 1971.

(7) 'Uniform Building Code,' International Conference of Building Officials, Los Angeles, 1970 Edition.

(8) Southern Standard Building Code, 1969 Edition, 1971 Revision.

(9) 'Nuclear Reactors and Earthquakes,' USAEC Report TID7024, August, 1963.

(10) American Society of Civil Engineers Transactions Volume 126, Part II, Paper No. 3269, 'Wind Forces on Structures,' 1961.

(11) Code of Federal Regulations Title 29, Chapter XVII, 'Occupational Safety and Health Standards,'Part 1910.

(12) NRC Regulatory Guides (RG)

RG 1.12 Instrumentation for Earthquakes

RG 1.31 Control of Ferrite Content in Stainless Steel Weld Metal

RG 1.10 Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures

RG 1.15 Testing of Reinforcing Bars for Category I Concrete Structures

RG 1.55 Concrete Placement in Category I Structures

(13) Structural Engineer Association of California, 'Recommended Lateral Force Requirements and Commentary,' 1968 Edition.

(14) Nuclear Construction Issues Group (NCIG)

NCIG-01, Revision 2 - Visual Weld Acceptance Criteria (VWAC) for Structural Welding

NCIG-02, Revision 0 - Sampling Plan for Visual Reinspection of Welds

The referenced NCIG documents may be used after June 26, 1985, for weldments that were designed and fabricated to the requirements of AISC/AWS.

NCIG-02, Revision 0, was used as the original basis for the Department of Energy (DOE) Weld Evaluation Project (WEP) EG&G Idaho, Incorporated, statistical assessment of TVA performed welding at WBNP. Any further sampling reinspections of structural welds subsequent to issuance of NCIG–02, Revision 2, are performed in accordance with NCIG-02, Revision 2 requirements.

The applicability of the NCIG documents is specified in controlled design output documents such as drawings and construction specifications. Inspectors performing visual weld examination to the criteria of NCIG-01 are trained in the subject criteria.

(15) TVA Reports

CEB 86-12 -Study of Long Term Concrete Strength at Sequoyah and Watts Bar Nuclear Plants

CEB 86-19-C - Concrete Quality Evaluation

(16) NRC Standard Review Plan, NUREG-0800, Rev. 2, Section 6.2.1.2, "Subcompartment Analysis".

3.8.3.3 Loads and Loading Combinations

Loading combinations and allowable stresses are shown in Table 3.8.3-1. General loads are described below.

Dead Loads

These loads consist of the weight of the structure and equipment, plus any other permanent load contributing stress such as hydrostatic pressure.

Live Loads

These are movable loads such as loads which occur during servicing equipment, crane loads, and water loads due to temporary flooding of various compartments.

Normal Temperature

These are the straight line temperature gradients which exist through member thicknesses due to differences in operating temperatures of various compartments.

LOCA Pressure

These loads are time-varying pressure differentials that will result between compartments in the event of a double-ended-break of a reactor coolant pipe, as discussed in Chapter 6. They vary in magnitude depending on the location of the pipe break. During the construction permit stage, the maximum calculated differential compartment pressures were increased by 40% in accordance with NRC requirements to account for uncertainties. At the operating license stage, the design pressures equalled or exceeded the peak calculated differential pressures. Dynamic load factors were not applied in the structural analysis except for the ice condenser support floor, the walls at the end of the ice condenser compartment, and the beam over the main steam pipe in the steam generator enclosures since the vibration period of other components in the structure is small in comparison to the rate of application and duration of the pressure loads.

LOCA Temperature

Time-varying nonlinear temperature gradients due to a LOCA cause stresses in the restrained members of this structure. These gradients will vary depending upon time and member location in relation to the pipe break. Stresses were computed for these loadings using a TVA developed program which has the same basic assumptions as the Reinforced Concrete Chimneys Code (ACI 505-54). A typical gradient for the divider floor, is shown in Figure 3.8.3-19.

Creep and Shrinkage

Creep was not considered in the design of interior concrete for the reasons outlined in Section 3.8.1.4.

Shrinkage effects were considered as outlined in Section 3.8.1.4. The peak hydration temperature of the concrete used in the interior structures was estimated to be approximately 130° F for summer placement with controlled placing temperatures of

65° F. From Figure 3.8.3-19 the average normal operating temperature for combination of shrinkage effects with other loads was 80° F resulting in a design temperature drop of 50° F. Under LOCA temperature gradients average temperatures exceed hydration temperatures and shrinkage stresses are relieved. Therefore, shrinkage effects are considered only with normal operating temperature gradients.

Operating Basis Earthquake (OBE)

Reference Section 3.7.2.

Safe Shutdown Earthquake (SSE)

This is the maximum postulated earthquake the plant is designed to withstand and still permit a safe shutdown. Reference Section 3.7.2.

Pipe Forces

These forces are the pressure jet effects that can occur due to breaks in the system's piping. They may be the jet force impinging upon the structure, or the equipment and piping anchorage forces as the result of such a jet. The static equivalent of the major equipment anchorage loadings were furnished by Westinghouse Corporation, the support designer.

Jet forces from postulated pipe ruptures, both longitudinal and transverse, were assumed to load the interior concrete structure.

$$F = (1.2pA)k$$

where:

F = Force on structure, lbs

p = Pressure, psi

A = Inside cross sectional area of pipe, in^2

k = Load factor

The effects of jet force was taken in combination with the uniform compartment differential pressure.

A minimum load factor k = 1.3 was used with both conditions based on localized yielding of the structural member and a ductility factor of 3.

The only jet force in the compartment above the reactor cavity is from the control rod opening. This is due to a pressure of 2250 Psi. The reactor coolant pipe will not produce a jet force in this area.

Missiles

The systems located inside the reactor containment have been examined to identify and classify potential missiles. The basic approach is to assure design adequacy against generation of missiles, rather than allow missile formation and try to contain their effects. Reference Section 3.5.

Ice Condenser Loads and Loading Combinations

The ice bed structure shall be designed to meet the loads described below within the behavior criteria limits presented in Section 3.8.3.5 of these criteria. The following load combinations are defined for design purposes:

(1) Dead Load + Operating Basis Earthquake Loads (D + OBE)*.

(2) Dead Load + Accident induced loads (D + DBA).

(3) Dead Load + Safe Shutdown Earthquake (D + SSE).

(4) Dead Load + Safe Shutdown Earthquake + Accident induced loads (D + SSE + DBA).

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* Includes thermal induced load and D+L

The loads are defined as follows:

Dead load (D) - Weight of structural steel and full ice bed at the maximum ice load specified.

Live Load (L) - Live load includes any erection and maintenance loads, and loads during the filling and weighing operation.

Thermal Induced Load - Includes those loads resulting from differential thermal expansion during operation plus any loads induced by the cooling of ice containment structure from an assumed ambient temperature at the time of installation.

Accident Fluid Dynamic and Pressure Loads (DBA) - Accident pressure load includes those loads induced by any pressure differential drag loads across the ice beds, and loads due to change in momentum.

Operating Basis Earthquake (OBE) - As previously defined.

Safe Shutdown Earthquake (SSE) - As previously defined.

3.8.3.4 Design and Analysis Procedures

3.8.3.4.1 General

Each component of the interior concrete structure was considered individually. Its boundary conditions and degrees of fixity were established by comparative stiffness; loads were applied, and moments, shears, and direct loads determined by either moment distribution or finite element methods of analysis. Reinforcing steel was proportioned for the component sections using the allowable stresses given in Table 3.8.3-1, the provisions of the ACI 318-71 Building Code and the proposed Standard Code for Concrete Reactor Vessels and Containments, ACI-ASME (ACI–359) Code, as issued for trial use, April 1973.

During the construction stage, a factor of 1.4 was applied to the design pressures resulting from LOCA. The structure was designed using the 40% margin and the recommendations of the ACI-ASME Joint Committee contained in Proposed Standard Code for Concrete Reactor Vessels and Containment. The results are tabulated in Table 3.8.3-2. NRC Standard Review Plan, NUREG-0800, Rev. 2, Section 6.2.1.2, Section II.B.5, permits reduction of design pressure so that the peak calculated differential pressure does not exceed the design pressure. This reduction in design pressure was utilized in the review of the concrete strength evaluation[2].

A completely independent design was performed on all portions of the divider barrier. Procedures used in this design and analysis are discussed in Sections 3.8.3.4.3 through 3.8.3.4.13.

3.8.3.4.2 Structural Fill Slab on Containment Floor

The fill slab is designed to span between walls with hydrostatic uplift pressure on 100% of its bottom face from water surface at Elevation 710. Loads from the steam generators and reactor coolant pumps are transferred directly into the base mat by continuous steel connections through the liner plate. As the base mat deflects under load, the fill slab deflects with it and is designed for these deflections. Analysis was made using the effects of crane wall uplift and rotation as well as the uplift of the support columns for the Elevation 716.0 floor. The fill slab is also designed for loads imposed upon it by the reactor coolant pipe crossover supports. Classical deflection formulas, as well as the computer code ICES STRUDL II, were used to determine moments, shears, and reactions. The finite element method of analysis was used to determine direct stresses in shear keys.

3.8.3.4.3 Reactor Cavity Wall

In the event of a circumferential split of a reactor coolant pipe at a reactor vessel nozzle, a nonsymmetric pressure occurs in the inspection cavity region. The pressures resulting from this break, a maximum of 148 psi, are applied to the design of the 8-foot, 6-inch thick and 4-foot, 3-inch thick reactor cavity wall. Large horizontal restraint forces from the steam generators and reactor coolant pumps of approximately 2,200 kips will be transmitted to the 4-foot, 3-inch thick section of wall.

A linear temperature gradient will occur during reactor operation. In the event of a LOCA the reactor is shut down and time-varying nonlinear, gradients similar to those in Figure 3.8.3-19 will occur in the wall. All temperature gradient cases are considered in the design up to a time of 48 hours following a LOCA.

Radiation generated heat on the structures is considered only for the primary shield immediately next to the reactor vessel. There the radiation generated heat is obtained as a function of position of the reactor core with respect to the structure and the temperature distribution is calculated. The effect of the temperature on the structure is then evaluated.

The average temperature of the wall during reactor operation exceeds hydration temperatures during construction. Therefore, tensile stress from hydration temperature considerations will be less than the stress induced by the temperature gradient. Long-time creep relaxation can be expected to substantially reduce temperature stresses; however, such a reduction was not utilized since the effective operating temperature differential across the wall of the reactor cavity was only 35° F. The 8-1/2-foot-thick portion of the wall is basically a thick cylinder. Formulas for stresses in thick cylinders were used to determine the ring moments and tensile stresses induced by the pressure loading and thermal gradients. The 8-1/2-foot-thick portion of the wall was also analyzed as fixed at its base, Elevation 702.78, and an analysis for vertical moments and forces was made for LOCA, reactor support loads and thermal effects. The 4-foot structural portion from the top of the 8-1/2-foot-thick ring to the top of the reactor continues in the same shape and configuration as the compartment above the reactor. This portion of the wall was modeled as a continuation of the compartment above the reactor using the computer program SAP IV (1973). The thin plate and shell element of this program was utilized. The wall was considered fixed at its base by the 8-1/2-foot-thick ring. The large openings for the reactor coolant system pipes occur in this portion of the wall and they were taken into account in the computer model. The axisymmetric pressure due to a reactor coolant pipe break was applied to the wall as well as the large concentrated forces from the steam generator and reactor coolant pump anchorages. Moments, shears, axial loads, and displacements were obtained from the computer analysis and reinforcing selection was made from these results.

Independent Design - Historical Information

The analysis of the reactor cavity was approached from the standpoint that a dynamic and inelastic type analysis might be required. However, comparison of the natural frequencies of the structure and the shock spectra of the pressure transients as forcing functions indicated that dynamic amplification was negligible. The results of static stress analyses showed that the stresses in the concrete were generally less than the cracking stress of concrete except in the immediate area of the break. Consequently, only a static elastic analysis was necessary. Analyses were performed for the pressure loads at specific times combined with support loads from reactor coolant loop and reactor pressure vessel. Thermal stresses resulting from this LOCA were also combined directly with pressure-induced stress.

The structural model used in the independent review was a three-dimensional assemblage of intersecting walls simulated by multiple layers of solid isoparametric finite elements. The general purpose computer program ANSYS, a well documented and widely used program, was used to calculate stress intensities. The reactor cavity structure was considered fixed at the intersection of the reactor cavity columns and operating deck and at the base.

A detailed analysis to determine unit stresses in the concrete and reinforcing steel was based on a cracked section and evaluated using the allowable stresses given in Table 3.8.3-1.

3.8.3.4.4 Compartment Above Reactor

This compartment, which has a design internal pressure of 32 psi, will also be subjected to water pressure when the refueling canal is full of water and refueling of the reactor is taking place. Wave effects of the water during earthquake are taken into account.

This compartment was analyzed in conjunction with the refueling canal walls using the SAP IV (1973) computer program. The reactor cavity wall, as well as the compartment above the reactor, are post-tensioned during construction. Tendons are located at approximately 45° increments around the wall at seven locations. These post–tensioning tendons are stressed as soon as the concrete in the walls has reached a proper strength. This stressing operation puts a 1,000 kips compressive load in the concrete walls at each of the tendon locations. This compressive force is taken into consideration in the reinforcing design of both the reactor cavity and the compartment above the reactor walls.

Independent Design - Historical Information

This compartment was analyzed in conjunction with the reactor cavity wall as described in Section 3.8.3.4.3. The 4-foot-thick compartment wall was designed for, a) a break at the reactor vessel nozzle in the lower cavity, b) a break in a primary coolant loop outside the reactor cavity structure along with associated support loads, c) a hydrostatic load applied to the interior face during refueling, and d) CRD mechanism restraint loads. The compressive stress due to post-tensioning of the walls was considered in the formulation of the finite element model.

3.8.3.4.5 Seals Between Upper and Lower Compartments

The design of the seals was by TVA without the use of a computer program.

The flexible coated fabric part of the seal was considered as a thin-wall half cylinder as the fabric width was sized to form an approximate semicircle when subject to internal pressure. With the semicircle, there is adequate slack in the seals to provide for relative movement between the attaching surfaces during all conditions without damage to the seals.

Earthquakes are the only natural environmental conditions which apply to the seals. The seals, being inside the containment vessel are protected from floods, wind, tornadoes, snow and ice. The seals are not in the area affected by missiles and therefore were not designed for missiles.

3.8.3.4.6 Refueling Canal Walls and Floor (Divider Barrier)

Primary Design

The canal walls and slab are designed to take the gravity and earthquake forces from the upper and lower internals storage stand. The face of the walls inside the lower compartment will be subject to a maximum LOCA pressure of 24 psi and localized jet forces due to a LOCA. The walls are subject to concentrated forces and moments from the reactor coolant pump restraints. The walls are subject to an uplift condition due to LOCA pressure acting on the divider barrier slab at Elevation 754.13. The canal walls and slab are designed for the water pressure in the canal during refueling operations. The seismic effect on this water was also considered.

The walls of the refueling canal and the compartment above the reactor were analyzed as a unit consisting of both straight and curved sections of walls. These walls were analytically modeled using the SAP IV (1973) finite element computer program. Shell curved-rectangular elements were used in the mesh assembly and spring constants were used to represent the stiffnesses of walls and slabs framing into the canal walls. Spring constants were used at the intersection of the canal walls with the crane wall, operating deck slab, and the canal floor slab.

The refueling canal slab was analyzed using the STRUDL finite element computer program. Both the rectangular and triangular flat plate element were used in the analysis.

Independent Design - Historical Information

(1) Refueling Canal Floor

The refueling canal floor slab was modeled and analyzed utilizing the SAP IV finite element program. The floor, due to its irregular shape, was analyzed in two sections. The larger area of the floor inside the crane wall radius was analyzed utilizing a finite element plate bending model. Boundary conditions were input reflecting the relative stiffnesses of the supporting walls. The slab was fixed at the crane wall boundary. A smaller segment beyond the crane wall support was analyzed using the conservative "strip" method of analysis.

SAP IV is a general structural analysis program for the static and dynamic analysis of linearly elastic structures. This program was developed and published by Bathe, Wilson, and Peterson, The College of Engineering, University of California at Berkeley, in June 1973, and has seen extensive usage since that time. The plate element used in the analysis is a quadrilateral of arbitrary geometry formed from four compatible triangles. A constant strain triangle was used to represent membrane stresses and the LCCT9 element was incorporated to represent bending behavior.

The design loads consisted of base plate forces from the upper and lower reactor internals, and fluid pressure forces from the flooded state during refueling. Earthquake and thermal gradient loading was also considered. Factored and unfactored load cases were checked and the reinforcing was sized for maximum stresses using the criteria of Table 3.8.3-1.

(2) Refueling Canal Walls

These walls were analyzed in conjunction with the reactor cavity walls as described in Section 3.8.3.4.3. Primary loads considered were, a) a break in a main steam line outside the reactor cavity with a resulting jet force and associated reactor coolant system support loads, b) a hydrostatic load during refueling with associated upper and lower internal support stand loads, and c) the effects of a main steam line break inside the reactor cavity. The model employed is described in the aforementioned Section.

3.8.3.4.7 Crane Wall

Wall Below Operating Deck

In the lower compartment the crane wall is subject to jet forces due to a possible break in the reactor coolant or main steam piping. The largest of these postulated jet forces is 2,650 kips occurring at a crossover leg between a steam generator and a reactor coolant pump. In this same area, the crane wall is subject to a large missile impingement load. Other areas of the crane wall in the lower compartment are exposed to uniform pressure differentials of approximately 23 psi.

The steam generators and reactor coolant pumps are braced laterally with restraints anchored into the crane wall. These restraints impose large concentrated loads on the wall. The largest of these loads is approximately 2,300 kips.

Crane wall temperature gradients, before and after a LOCA, were investigated. At several elevations in the crane wall, maximum and minimum vertical loads were computed using results from the "Dynamic Earthquake Analysis of the Interior Concrete Structure, prepared by TVA. In addition, various parts of the crane wall were designed to handle concentrated loads, 100 to 300 kips, resulting from breaks in small piping systems.

The crane wall was analyzed by isolating areas spanning between slabs and cross walls. Moments, shears, and axial forces were calculated using the STRUDL finite element program. Fixed-end moments were distributed between adjacent sections of wall using conventional distribution methods.

Columns between ice condenser doors are subjected to moments and forces distributed from the ice condenser floor and divider barrier floor, as well as moments, shears, and axial forces from the wall sections above and below the columns. The columns were designed for these moments, shears, and axial forces plus earthquake loads. The columns in the vicinity of the steam generator and pressurizer compartments are greatly influenced by the lateral restraints from the aforementioned equipment, which is anchored in the crane wall immediately at the top of the columns.

Personnel Access Doors in the Crane Wall

Main structural members of the doors were considered as simple beams. Energy absorbing members were considered as collapsible members. Members of the embedded frames were considered as being rigidly supported by concrete. Loads from the embedded frames are transferred to the concrete by embedded anchors.

Design of the doors and embedded frames was by TVA without the use of a computer program. Design of collapsible members on the doors was based on tests made by Oak Ridge National Laboratory. Results of these tests are recorded in their publication titled Structural Analysis of Shipping Casks, Volume 9, "Energy Absorption Capabilities of Plastically Deformed Struts Under Specified Impact Loading Conditions." Collapsible members were sized to limit loads transmitted to the embedded frame to 13,000 pounds per linear inch.

The doors were designed to function during normal conditions, earthquakes, and pipe rupture accidents.

Earthquakes are the only natural environmental condition which applies to the doors. The doors are protected from flood, wind, tornadoes, ice and snow, since they are located inside the containment vessels.

The doors will be closed any time reactor containment is required, except when a workman is passing through the access.

When containment is not required, the doors are not required to seal or to retain their integrity. Since the doors are left open only when containment is not required, seismic qualifications of the doors in the open position is not required.

Earthquake loads used in designing the doors were from accelerations determined for the crane wall at the horizontal centerline of each door by dynamic analysis of the Reactor Building for an OBE and an SSE. These acceleration loads were used as static loads since the doors are firmly secured to the wall when closed. Doors were reanalyzed using Set "B" ARS spectrum.

Some air leakage may occur at the periphery of the doors during earthquakes, but this leakage will not exceed the permissible leakage of 30 square inches per door.

Under normal conditions, seals on the doors will have a life of not less than 10 years, and the other parts of the doors will have a life of not less than 40 years. Some air leakage may occur at the periphery of the doors, but this leakage will not exceed the permissible leakage area for normal operation of 10 square inches per door.

Wall Above the Operating Deck (Divider Barrier)

Primary Design

Under accident conditions the crane wall above the operating deck is designed for maximum pressure differentials between the ice compartments outside the crane wall and the steam generator and pressurizer compartments. It is also designed for the loads imposed on the wall by the lattice frame anchorages of the ice condenser. Maximum and minimum vertical loads imposed by the earthquake analysis are combined with these loads for the maximum stress conditions. The end walls of the ice condenser and the spacing of the steam generator and pressurizer walls stiffen the crane wall to such an extent that it essentially spans horizontally between these supporting walls. The stud loadings on the wall from the lattice frame may either add to or subtract from the pressure loading depending on whether the maximum pressure is inside the crane wall or in the ice compartment. The portion of the crane wall behind the steam generator and pressurizer compartments is subject to jet impingement loads.

The steam generator and pressurizer upper lateral restraints are anchored in the crane wall and exert large forces on it. The effect of pressure on the top slab of the pressurizer and steam generator compartments causing an uplift on the crane wall was considered.

The STRUDL II frame program, STRUDL II finite element program, and the SAP IV (1973) finite element program are the principal computer programs used in the analysis of the upper crane wall above the operating deck slab.

The upper restraint of the steam generators is designed such that the crane wall only receives load from a steam line break. Seismic restraints in the radial direction are transmitted through hydraulic snubbers to the floor of the operating deck. In the other direction they are transmitted to the walls of the steam generator compartment. The two 720-kip steam generator restraint loads on the crane wall are assumed to occur coincidentally with the maximum pressure differential in the steam generator compartment.

During construction a 36-foot-wide opening, used for moving major equipment into the building, was left in the crane wall at approximately the 90° azimuth. This opening began at elevation 756.63 and extended 46 feet high. This leaves a 3-foot-wide, 17–foot-deep curved beam spanning a 37-1/2-foot arc over the opening. This beam and the permanent beam over the refueling canal are designed to carry the construction loads of the polar crane, approximately 1200 kips maximum while installing major equipment. The permanent beam is also designed to take the reactions from the cantilevered beams supporting the ice condenser bridge crane. The analysis of these beams was made using the STRUDL computer program. The top of the crane wall is designed to withstand a force in the radial direction due to the polar crane bumping into it as a result of seismic action. This seismic force from the polar crane is considered to act at any point on the circumference of the wall and approximately 2 feet below the top of the wall.

Independent Design - Historical Information

The crane wall resists two general types of loads. They are, 1) localized forces from equipment supports, pressure forces, structure discontinuities, etc., and 2) forces from gross structure motions of the interior concrete structure induced by the design earthquakes and the design basis accident (DBA). Calculations of gross forces in the interior concrete structure due to a design earthquake are described in Section 3.7.2.1.1. The lumped mass cantilever beam model used in the seismic analyses was loaded by a time-dependent forcing function representing the nonaxisymmetric pressure loads from a DBA. The analysis used modal superposition and determined the total responses in the time domain. The results of the analysis consisted of gross overturning moments and shears and accelerations, deflections, and acceleration response spectra at various elevations.

Portions of the crane wall subjected to very isolated forces were isolated and designed as substructures. Boundary conditions were always chosen to give conservative results.

The forces at the junction of crane wall and other divider barrier components such as the operating deck, ice condenser floor, and end walls were included in the design.

The crane wall at the ice condenser inlet doors between elevations 746.42 to 753.63 were designed as beam columns. The columns are subjected to both localized and gross motion forces. Localized loads resulted from the steam generator support loads, pressure forces, and interactions from the operating deck and the ice condenser floor. The column design was verified by both working stress and ultimate strength methods.

The portion of the crane wall within the steam generator compartment was analyzed as part of the steam generator enclosure. This portion was modeled using the MARC–CDC nonlinear finite element computer program. The element utilized was the 20-node isoparametric solid.

The loadings on the wall consisted of reaction loads from the steam generator supports near the crane wall columns and pressure loads from postulated breaks in the main steam line. The crane wall columns were designed to resist maximum moments, shears, and axial loads due to the local forces on the crane wall as well as forces due to gross motions of the interior concrete. The crane wall segment of the steam generator compartment model was extended past the juncture of the enclosure and the crane wall to minimize boundary effects on the solution. Boundary conditions assumptions were selected to provide conservative stresses. The crane wall within the pressurizer compartment was designed by a similar method.

3.8.3.4.8 Steam Generator Compartments (Divider Barrier)

These compartments are designed to resist a maximum of 38 psi differential pressure on the wall common with the upper compartment and 26 psi differential pressure on the center wall that would result following a main steam pipe break inside any single compartment. The center wall is also designed for the effect of a 1160-kip jet force that would result from a main steam pipe break. Also accounted for are thermal effects accompanying a pipe break (see Figure 3.8.3-19).

The compartments span mainly in the horizontal direction resulting in tensile stresses and horizontal moments in the walls near the center of their height. Close to the ends of the compartments, discontinuity stresses result in the vertical direction, similar to those of a flat head cylinder.

The STRUDL frame program was used to find the maximum horizontal forces in the walls by modeling a vertical 1-foot height of walls including a 113-degree sector of crane wall. Short chord lengths were used to represent curved sections of walls. Manual calculations were done at the top and bottom of the wall which is common to the upper compartment to investigate the effects of the slabs restraining the wall. In addition, the flat plate finite element STRUDL computer program was used to analyze the center wall for moments and shears in both directions. The top slab was analyzed using stiffened members in the flat plate finite element STRUDL program. The inverted "tee"-shaped beam which stiffens the top slab and which is located at the top of the center wall was analyzed for the dynamic effects of a main steam pipe breaking and striking the flange of the beam.

Independent Design - Historical Information

(1) Roof Slab

The steam generator enclosure roof slab was analyzed as a thick plate using the three-dimensional 20-node isoparametric solid element available in the MARC-CDC finite element program. The T-beam stiffener attached to the inside of the slab was included in this finite element model.

The MARC-CDC finite element program is based on research work carried out by Professor Pedro V. Marcal of Brown University and colleagues at the University of London. In 1969 the program was released commercially by the Marc Analysis Research Corporation. The Control Data Corporation has recently documented the program and offered it for general usage through their computer system. A sample problem was run to verify the results obtained from the program.

The boundary conditions of the roof slab were approximated by modeling the actual stiffness of the steam generator enclosure wall and crane wall.

The design loads considered originated from a postulated rupture in the main steam line in a steam generator compartment. Differential pressures, jet force and pipe reaction resulting from the postulated break were checked for factored and unfactored load cases. The T-beam beneath the roof slab was designed to resist pipe whip forces from a postulated guillotine break in the main steam line. Moments, shears, and torsional stresses were checked to ensure adequate reinforcing utilizing the stress criteria of Table 3.8.3-1.

(2) Enclosure Walls and Separation Wall

The steam generator enclosure wall, separator wall, and a segment of the crane wall were modeled utilizing the three-dimensional 20-node isoparametric element available in the MARC-CDC nonlinear finite element program.

Spring constants simulating the crane wall columns were obtained by analyzing a small portion of the crane wall. The walls of the steam generator enclosure were then modeled to include appropriate boundary conditions at the intersection of the operating deck, upper crane wall, and the crane wall columns.

The loadings considered were of two basic types: a) a pressure load obtained from a hypothetical DBA, and b) snubber and embedment plate loads caused by combined LOCA and seismic action. Live loads, dead loads, seismic loads, and thermal loads were also considered.

Flexural, axial, shear, and torsional stress levels resulting from the factored and unfactored loading combinations were evaluated using the allowable stress criteria of Table 3.8.3-1. Reinforcement was selected based on these criteria.

3.8.3.4.9 Pressurizer Compartment (Divider Barrier)

Primary Design

The compartment is designed to resist a 50 psi differential pressure. Methods of analysis were similar to those of the steam generator compartments.

Independent Design - Historical Information

The enclosure wall and the adjacent crane wall were modeled using solid isoparametric finite elements. The computer program utilized was the previously documented SAP IV. The crane wall model was extended beyond its juncture with the enclosure wall to minimize boundary effects. Boundary conditions at the intersection of the enclosure wall with adjacent elements of the interior concrete structure were chosen to provide conservative stress results.

The roof slab was modeled as a thick plate using the ANSYS computer program, a widely used and previously documented program. Boundary conditions at the junction with the crane and enclosure walls were chosen to provide maximum stress levels.

With these stress levels, a detailed analysis to determine unit stresses in the concrete and reinforcing steel was based on a cracked section and evaluated using the allowable stresses given in Table 3.8.3-1.

3.8.3.4.10 Operating Deck at Elevation 756.63 (Divider Barrier)

Primary Design

The floor is designed to carry a 24 psi upward pressure and thermal effects due to a LOCA plus the jet pressure of 340 psi acting over a local area. Upward loads from the missile shield are taken by this floor around the reactor cavity where the shield is bolted down.

This floor is designed for a 1,000-psf live load. This loading suffices for several concentrated loads from the reactor head setdown which occur during refueling and periodic maintenance of the equipment.

During construction, the floor has no edge support at the steam generator and pressurizer compartment walls, since the first lifts of these walls are carried by the floor. This special construction condition was examined separately using a 300-psf design live load in addition to the wet weight of the first 45-foot pour of walls.

The floor analyses were made using the STRUDL finite element program for flat plates utilizing both rectangular and triangular elements to assemble the irregular shape.

Anchorage for the upper steam generator restraints is provided in this floor. These anchorage points have approximately a 5,500-kip design force due to a LOCA combined with SSE. This force is horizontal and applied at points where openings create horizontal single span beams. These beams were analyzed using manual methods.

Independent Design - Historical Information

The operating deck was analyzed using four finite element models to represent this irregularly shaped slab. Two models were used to analyze the larger segment from 0° to 180° while two additional models were used to analyze the segments adjacent to the refueling canal walls. Loads normal to the surface and inplane loads act on the operating deck. Stresses from concentrated snubber (inplane) loads were determined from a plane stress finite element analysis. These snubbers act as the lateral supports for the steam generators. In addition to the snubber loads, pressure loads and jet forces from postulated pipe ruptures acting normal to the operating deck were determined from a plate bending finite element model. Dead, live, seismic, pipe support, and construction loads were added to the differential pressure and snubber forces to complete the factored and unfactored load cases.

The SAP IV plate bending and plane stress elements were used to calculate moments, shears, reactions, and inplane stresses. Boundary conditions at the crane wall, steam generator enclosure, pressurizer enclosure, reactor cavity, and refueling canal wall junctions were chosen to give the most conservative results. Using the calculated moment, shears, and axial forces, stresses in the reinforcement for an assumed cracked section were checked against the stress criteria of Table 3.8.3-1.

3.8.3.4.11 Ice Condenser Support Floor Elevation 744.5 (Divider Barrier)

Primary Design

The finite element program SAP IV (1973) was primarily used in the analysis and design of the Elevation 744.5 floor. The outer circumferential beam was represented along with the floor by using a combined flat plate and grid member system. The supporting columns were modeled using spring constants for both rotation and deflection. Shear and moment values were obtained from the computer program at the crane wall, ice condenser end walls, and supporting columns. Reinforcing selections were made from these results.

Independent Design - Historical Information

The ice condenser floor was analyzed as a series of circumferentially beam-stiffened curved slabs with a continuous support along the inner radius and ends. These slabs were supported along the outer radius by flexible columns. The analysis was made utilizing SAP IV, a previously documented computer program. Models were generated for segments of the ice condenser floor at 0°, 90°, 44°, 180°, and 230°. Boundary conditions between the segments were input to provide maximum forces at the boundary and midspan.

The various segments of the ice condenser floor were dynamically analyzed to determine natural frequencies. Next, time-varying differential pressure loads were used to calculate dynamic load factors associated with the natural frequencies of each segment. The dynamic load factors obtained were then applied to the maximum differential pressures in addition to the factors for the ice condenser structure support loads.

The loadings evaluated included concentrated forces and moments from the ice condenser support system, jet forces resulting from change in momentum of the steam flow, and LOCA induced pressure differentials. The ice condenser support system loads resulted from a combination of seismic accelerations and drag on the ice baskets by the channeled gases. Normal and factored load cases were examined to determine maximum moments and shears. Stresses in the concrete and reinforcement were evaluated against the criteria of Table 3.8.3-1.

3.8.3.4.12 Ice Condenser

Analysis, meeting the criteria presented in Section 3.8.3.5, has been done on the basis of elastic system and component analyses. Limit load analysis was used as an alternate to the elastic analysis. Limit loads are defined using limit analysis by calculating the lower bound of the collapse load of the structure. Load factors are applied to the defined design basis loads and compared to the limit loads. The load factors determined for design basis load are used to provide margins of safety of the structure against collapse. A load factor of 1.43 was used when considering the mechanical loads due to dead weight and OBE. A load factor of 1.3 was used for (D + SSE) and (D + DBA). The material was assumed to behave in an elastic-perfectly-plastic manner where strain-hardening effects are neglected. The minimum specified yield strength was used. Mechanical plus thermal induced load combination and fatigue was analyzed in an elastic basis and satisfy the limits of Section 3.8.3.5. The stress analyses and results are described in Sections 3.7 and 6.7.

Experimental or Test Verification of Design

In lieu of analysis, experimental verification of design using actual or simulated load conditions was used.

In testing, account was taken of size effect and dimensional tolerances (similitude relationships) which exist between the actual component and the test models, to assure that the loads obtained from the test are a conservative representation of the load carrying capability of the actual component under postulated loading. The load factors associated with such verification are: 1.87 for D + SSE, 1.43 for D + DBA or D + SSE, and 1.3 for (D + SSE) or 1.3 for (D + DBA).

A single test sample is permitted but in such cases test results were derated by 10%. Otherwise, at least three samples were tested and the design was based on the minimum load carrying capability.

Additional analysis results are found in Section 6.7.

3.8.3.4.13 Penetrations Through the Divider Barrier

Canal Gate

Primary Design

The canal gate sections are designed to span as simply supported beams across the refueling canal; a clear span of some 19 feet. Hand calculations using conventional methods were used for this design. The canal gate was designed to withstand a 39-psi pressure differential between the compartment above the reactor and the upper compartment due to a LOCA. The effect of seismic action on the canal gate sections is considered as well as the effect of maximum temperature differential across the gate.

Independent Design - Historical Information

The canal gate was designed as a simply supported plate spanning between the two refueling canal walls. The canal gate is required to maintain integrity between the upper and lower compartments during a LOCA and was designed for the maximum probable differential pressure. The effects of seismic and thermal action were evaluated.

Moments and shears were calculated using conventional hand methods. The evaluation of concrete and reinforcing steel stresses was based on a cracked section and the allowable stresses of Table 3.8.3-1.

Control Rod Drive (CRD) Missile Shield

Primary Design

The CRD missile shield sections are designed to span as simply supported slabs across the compartment above the reactor. The slabs are held down at the ends by anchor bolts embedded in the operating deck slab. The missile shield is designed to withstand a maximum differential pressure of 39 psi between the compartment above the reactor and the upper compartment due to a LOCA. The missile shield is subject to loading from the CRD mechanism as a missile. An accompanying jet force due to pressure escaping through the head of the reactor is also considered. The slabs are investigated for the maximum penetration resulting from the missile effects of the control rod drive shaft. The underside of the slab is faced with a 1-inch-thick steel plate to aid in resisting missile penetration. The penetration depths are calculated by use of the Petry formula and a formula by C. V. Moore, "The Design of Barricades for Hazardous Pressure Systems," Nuclear Engineering and Design 5 (1967), 81-97, North-Holland Publishing Company, Amsterdam. The calculated penetration depth is 2.2 inches into the 3-foot, 6-inch thick slab. The effect of maximum temperature differential across the missile shield is also considered in the design.

Independent Design - Historical Information

The missile shield sections were analyzed as simply supported slabs spanning the compartment above the reactor. This shield must resist the maximum probable differential pressure from a LOCA to maintain integrity between the lower and upper compartments. Additionally, it must resist certain missiles from the control rod drive mechanism. Penetration into the steel plate and concrete were calculated and equivalent static loads for the impacting missiles were calculated and evaluated. Thermal stresses resulting from temperature differentials between lower and upper volumes were considered in the stress evaluation.

Concrete and reinforcing steel stresses were determined considering a cracked section and the stress allowables of Table 3.83-1.

Reactor Coolant Pump Access and Lower Compartment Access Hatches

Primary Design

The reactor coolant pump access and lower compartment access slabs are designed to span simply supported between anchor bolt. The slabs are designed for both downward and upward loads acting on them. The downward loads are dead load and a 1,000-psf live load. For upward loads, the slabs are designed to carry a 24-psi differential pressure between the lower and upper compartments due to a LOCA. A jet impingement loading associated with this LOCA is also considered. The effect of maximum temperature differential as well as seismic effects on the slabs are accounted for in the design.

Independent Design - Historical Information

The reactor coolant pump access hatch and the lower compartment equipment access hatch were analyzed and designed as simply supported circular and rectangular plates. Maximum moments and shear forces were obtained from a plate bending analysis. Dead, live, seismic, and thermal loads were combined with differential pressures and jet forces due to a postulated LOCA to give controlling factored and unfactored load cases. Shear stresses at the periphery of the hatch openings in the operating deck and stress levels in the perimeter anchor bolts were checked to ensure compliance with criteria of Table 3.8.3-1.

Equipment Access Hatch

The hatch cover is designed to span as a simply supported beam between the anchor bolt assemblies with the anchor bolts designed to withstand a load at least 5% greater than that calculated for the end reactions resulting from the actual load on the hatch. The allowable stresses are given in Table 3.8.3-6.

Escape Hatch

Structural components of the hatch have been designed such that, the allowable stresses given in Table 3.8.3-7 will not be exceeded.

Air Return Duct Penetrations

The controlling design condition is Design Basis LOCA Pressure Dead Load + Safe Shutdown Earthquake Loads. Maximum differential temperature is considered in the design, but does not occur coincidentally with a jet force or the maximum differential Design Basis LOCA Pressure. Calculated and allowable stresses are given in Table 3.8.3-8.

3.8.3.5 Structural Acceptance Criteria

3.8.3.5.1 General

Structure

Calculated and allowable stresses for the principal concrete structure are listed in Table 3.8.3-2. Locations of these areas are shown referenced to Figures 3.8.1-1, 3.8.3-6 and 3.8.3-7. The working stress design method was used for finding stresses.

3.8.3.5.2 Structural Fill Slab on Containment Floor

During the original design (construction permit) phase with 40% added to LOCA pressure, the controlling combination was "Abnormal/Severe Environmental." See Table 3.8.3-2.

Shear transfer through the fill slab is discussed in Section 3.8.3.1.2.

3.8.3.5.3 Reactor Cavity Wall and Compartment Above Reactor

Loading combinations 1 through 7 in Table 3.8.3-1 were considered in the design. During the original design (construction permit) phase with calculated values of LOCA pressure load increased by 40%, the controlling load combinations are "Abnormal" and "Abnormal/Severe Environmental." See Table 3.8.3-2. The 4-foot, 3-inch structural thickness provided adequate depth for limiting peripheral shear stresses due to anchorage loads on the steam generator and reactor coolant pump restraints.

Earthquake shears for the interior structures were distributed to the various walls in proportion to their rigidity. This term is defined as $1/\Delta$ where delta is the deflection due to a unit force, and it includes deflection due to bending and shear. The procedure is outlined in "Analysis of Small Reinforced Concrete Buildings" by the Portland Cement Association.

The columns at the top of the compartment above the reactor wall were designed for the effect of earthquake shears by proportioning shear to them based upon their rigidity as previously described.

3.8.3.5.4 Refueling Canal Walls and Floor

Loading combinations 1 through 7 in Table 3.8.3-1 were examined. During the original design (construction permit) phase with calculated values of LOCA pressure load increased by 40%, the controlling load combinations are "Abnormal" and "Abnormal/Severe Environmental." See Table 3.8.3-2.

3.8.3.5.5 Crane Wall

The crane wall was analyzed for loading combinations 1 through 7 in Table 3.8.3-1. During the original design (construction permit) phase with calculated values of LOCA pressure load increased by 40%, the controlling load combination is "Abnormal/Severe Environmental." See Table 3.8.3-2.

Earthquake shears were calculated utilizing the procedure discussed in Section 3.8.3.5.3.

Shear reinforcement was required in many areas of the crane wall for radial shears generated by jet impingement loading, as well as pipe and equipment restraint reactions, missile impingement loading and pressure due to LOCA.

3.8.3.5.6 Steam Generator and Pressurizer Compartment

Loading combinations 1 through 7 in Table 3.8.3-1 were examined. During the original design (construction permit) phase with calculated values of LOCA pressure load increased by 40%, the controlling load combination is "Abnormal" for the pressurizer compartment and "Abnormal/Severe Environmental" and "Abnormal/ Extreme Environmental" for the steam generator compartment. See Table 3.8.3-2.

3.8.3.5.7 Operating Deck at Elevation 756.63

Loading combinations 1 through 7 in Table 3.8.3-1 were examined for the floor. During the original design (construction permit) phase with calculated values of LOCA pressure load increased by 40%, the controlling load combination is "Abnormal/Severe Environmental." See Table 3.8.3-2.

3.8.3.5.8 Ice Condenser Support Floor Elevation 744.5

Loading combinations 1 through 7 in Table 3.8.3-1 were examined in the design of the floor. During the original design (construction permit) phase with calculated values of LOCA pressure load increased by 40%, the controlling load combination is "Abnormal." See Table 3.8.3-2.

3.8.3.5.9 Penetrations Through the Divider Barrier

Canal Gate and Control Rod Drive (CRD) Missile Shield

Loading combinations 1 through 7 in Table 3.8.3-1 were examined. During the original design (construction permit) phase with calculated values of LOCA pressure load increased by 40%, the controlling load combination is "Abnormal/Severe Environmental." See Table 3.8.3-2.

Reactor Coolant Pump and Lower Compartment Access Hatches

Loading combinations 1 through 7 in Table 3.8.3-1 were examined. During the original design (construction permit) phase with calculated values of LOCA pressure load increased by 40%, the controlling load combination is "Abnormal/Severe Environmental."

Escape Hatch

Loading combinations 1 through 7 in Table 3.8.3-1 were examined. During the original design (construction permit) phase with calculated values of LOCA pressure load increased by 40%, the controlling load combination is "Abnormal/Severe Environmental."

3.8.3.5.10 Personnel Access Doors in Crane Wall

Allowable stresses for non-collapsible members for load combinations used for the various parts are given in Table 3.8.3-3. Normal load conditions are shown for mechanical members only. Loads on structural members during normal conditions are negligible and therefore are not shown on Table 3.8.3-3. For normal load conditions, factors of safety for mechanical parts are 5 to 1 on ultimate. For limiting conditions such as SSE and a pipe rupture accident, stresses do not exceed 0.9 yield.

For collapsible members during a pipe rupture accident, stresses exceed yield and members are plastically deformed. Plastic deformation of energy absorbing members does not affect the sealing integrity of the doors.

3.8.3.5.11 Seals Between Upper and Lower Compartments

Under normal and earthquake conditions, there are no loads on the seals. However, the seals are subject to radiation, as outlined previously, during normal operating conditions. The seal has been tested under accident pressures and temperatures after undergoing heat aging to 40 years equivalent age, and irradiation to 40 years normal operation plus accident integrated doses in order to qualify it for the life of the plant.

The seals are not required to maintain their integrity during a fire. It is assumed that a fire and an accident which require sealing will not occur simultaneously since the reactor will be shut down immediately if a fire develops.

3.8.3.5.12 Ice Condenser

Table 3.8.3-4 provides a summary of the allowable limits to be used in the design of the ice condenser components.

For all cases the stress analysis was performed by considering the load combinations producing the largest possible stress values.

Stress Criteria

The stress limits for elastic analysis are:

(1) D + OBE

Stress shall be limited to normal AISC, Part I Specification allowables (S). The members and their connections shall be designed to satisfy the requirements of Part I, Sections 1.5, 1.6, 1.7, 1.8, 1.9, 1.10, 1.15, 1.16, 1.17, 1.20, 1.21 and 1.22 of the AISC Specification (stress increase in Sections 1.5 and 1.6 is disallowed for these loads). Where the requirements of Section 1.20 are not met, differential thermal expansion stresses shall be evaluated and the maximum range of the sum of mechanical and thermal induced stresses shall be limited to three times the appropriate allowable stresses provided in Section 1.5 and 1.6 of AISC Specification.

(2) D + SSE, D + DBA

Stresses shall be limited to normal AISC Specification allowables given in Sections 1.5 and 1.6, increased by 33% (1.33S). No evaluation of thermal induced stresses or fatigue is required. In a few areas, where the stresses exceed 1.33S but are below I.5S, specific justification is provided on a case by case basis.

(3) D + SSE + DBA

Stresses shall be limited to normal AISC Specification allowables given in Sections 1.5 and 1.6, increased by 65% (1.65S). No evaluation of thermal induced stresses or fatigue is required.

For all cases, direct (membrane) mechanical stresses shall not exceed $0.7S_u$, where S_u is the ultimate tensile strength of the material.

The summary of the ice condenser allowable limits is given in Table 3.8.3-4.

3.8.3.6 Materials, Quality Control and Special Construction Techniques

General

Refer to Section 3.8.1.6.

3.8.3.6.1 Materials

Refer to Section 3.8.1.6.1 with the following additions.

Concrete

Aggregates for radiation-shielding concrete which was used in limited locations conformed to ASTM C 637-73.

The specified strengths of concrete used for interior concrete structures were 3000 psi, 4000 psi, 5000 psi, and 8000 psi.

Reinforcing Steel

Prestress steel which was used in the reactor cavity walls conformed to ASTM A 421–65.

Personnel Access Doors in Crane Wall

ASTM standards were used for all material specifications and certified mill test reports were provided by the contractor for materials used for all load carrying members.

Seals Between Upper and Lower Compartments

The seals consist of long strips of flexible elastomer coated fabric with both edges hemmed to form pockets into which metal clamp bars are inserted. The coated fabric is two ply dacron coated on both sides with an elastomer (ethylene-propylene-dienepolymer). The elastomer is compound E603 or E603A by the Presray Company.

Escape Hatches in Elevation 756.63 Floor

ASTM standards were used for all material specifications and certified mill test reports were provided by the contractor for materials used for all load carrying members.

3.8.3.6.2 Quality Control

Concrete

The quality control requirements were essentially the same as in Section 3.8.1.6.2. Some concrete did not meet specification requirements. This was evaluated and documented in Reference [2]. Results have been documented in affected calculation packages and drawings.

Personnel Access Doors in Crane Wall, Escape Hatches in Elevation 756.63 Floor

Design by TVA and erection by TVA were in accordance with TVA's quality assurance program. Design and fabrication by the contractor were in accordance with the contractor's quality assurance program which was reviewed and approved by TVA's design engineers. The contractor's quality assurance program covers the criteria in Appendix B of 10 CFR 50. Fabrication procedures such as welding and nondestructive testing were included in Appendices to the contractor's quality assurance program.

ASTM standards were used for the material specifications and certified mill test reports were provided by the contractor for materials used for the load carrying members.

Seals Between Upper and Lower Compartments

The flexible elastomer coated fabric used for seals was certified by a qualified rubber technologist as being adequate for the normal and accident conditions. In addition, certified mill test reports were provided by the contractor for materials used for the load carrying members.

The seal has been tested by the original seal supplier under contract with TVA. The test was designed to evaluate seal specimens under simulated accident temperature and pressure conditions in a configuration emulating actual plant as-constructed installation. The test specimens, which were fabricated from seal material removed from the Unit 1 containment, were heat aged to 40 years equivalent age, and irradiated to 40 years normal operation plus accident integrated doses prior to testing. This testing process represented the material properties that would exist following a design basis accident at the end of a 40 year plant life.

3.8.3.6.3 Construction Technique - Historical Information

No unusual construction procedures were employed in the construction of the interior structures.

3.8.3.6.4 Ice Condenser

Structural steels for ice condenser components are selected from the various steels listed in the AISC Specification or Code. When materials such as steel sheets, stainless steel or nonferrous metals are required and are not obtainable in the AISC Code, these materials are chosen from ASTM Specifications. Proprietary materials such as insulating materials, gaskets and adhesives are listed with the manufacturers' name on the component drawings.

Material certifications for chemical analysis and tensile properties were required with testing procedure and acceptance standards meeting the AISC or ASTM requirements.

Because the concept of nonductile fracture of ferritic steel is not a part of the AISC Code, and Westinghouse recognizes its importance in certain ice condenser components where heavy plates and structurals are used, such as the lower support structure, Charpy V-notch (CVN) energy absorption requirements are stipulated as shown in Table 3.8.3-5.

These criteria apply to the design of the following ice condenser components:

(1) Ice basket and coupling.

(2) Lattice frame and columns including attachments and bolts.

(3) Structural steel supporting structures comprising the lower support structure, door frames and bolts.

(4) Wall panels and cooling duct support studs attached to the crane wall and walls.

(5) The supports of auxiliary components which are located within the ice condenser cavity but which have no safety function.

The various candidate materials, i.e., steel sheets, structural shapes, plates and bolting used in the ice condenser system were selected on the following bases:

(1) Provide satisfactory service performance under design loading and environment and pressure or construction performance.

(2) Assure adequate fracture toughness characteristics at ice condenser design conditions.

(3) Be readily fabricated, welded, and erected.

(4) Be readily coated for corrosion resistance when required.

The candidate materials are of high quality and were made by steel-making practices to be specified by Westinghouse. Principal candidate materials meeting the above bases are listed below. Other materials for specific applications are selected on a case-by-case basis.

Sheets

Carbon steel sheets are commercial quality (CQ), drawing quality (DQ), or drawing quality-special killed (DQSK). The selection of the quality depends upon the part being formed. When higher strength, structural quality sheets are required, ASTM specification A607 is used. AISI Type 409 modified stainless steel is a potential alternate sheet material for the ice baskets.

The ice baskets were made from perforated sheet material. The wall duct panels were made from sheet material and the cradle supports from structural sections and plates.

Structural Sections, Plates and Bar Flats

Structural sections, plates and bar flats are generally highstrength, low alloy steel selected for suitable strength, toughness, formability and weldability.

The high-strength low-alloy steels are A441, A588, A572 or A633. These steels are readily oxygen cut and possess good weldability.

Bolting

High-strength alloy steel Type A320 L7 bolting for low temperature service is used for the lower support structure. Stocked bolting made from A325, A449 and ASTM A354, Grade BD (SAE J429, Grade 8) materials are used for other parts. The above bolts met CVN 20 ft-lb at -20° F, for sizes greater than 1 inch in diameter.

Nonmetallic materials such as gaskets, insulation, adhesives and spacers are selected for specific uses. Freedom from detrimental radiation effects is required.

All structural welding was in accordance with the AWS Structural Code for Welding, D1.1 (AWS Code). The AWS Code is an overall welding system for the design of welded connections, technique, workmanship, qualification and inspection for buildings, bridges, and tubular structures. (See Section 3.8.3.2, Item 5).

The quality of welds for the ice condenser system is based on Paragraph 9.25 of the AWS Code. (See Section 3.8.3.2, Item 5).

Resistance welding was in accordance with AWS, Recommended Practices for Resistance Welding, C1.1.

Magnetic particle examination was performed on at least 5% of the welds in each critical member of the lower support structure. Magnetic particle or liquid penetrant examinations, where applicable, were performed on 5% of the welds in each critical member of the balance of the ice condenser structure. The welds selected for examination were designated in the Design Specifications. The nondestructive examination methods and acceptance standards are given in Section 6 and Paragraph 9.25, Quality of Welds, of the AWS Code. (See Section 3.8.3.2, Item 5).

3.8.3.7 Testing and Inservice Surveillance Requirements

Testing of the interior concrete structures was not planned. A completely independent design has been prepared for divider barrier features in order to ensure that during a LOCA the escaping steam will not bypass the ice condenser.

Personnel Access Doors in Crane Wall

Periodic visual inspections of the doors are to be made. Parts inspected during the visual inspection are to include all bolted connections, structural members for paint deterioration, latches, hinges, and elastomer seals. The seals are to be inspected for cracks, blemishes, or any other indications of deterioration of the elastomer and for proper seating at the sealing surfaces.

Seals Between Upper and Lower

On periodic unit shutdowns, visual inspections of the seals are to be made. Parts inspected are to include all bolted connections, clamp bars, metal to fabric joints and the elastomer-coated fabric. The seals are to be replaced if they show any evidence of deterioration.

Escape Hatches in Elevation 756.

Periodic visual inspections of the hatch covers are to be made. Parts inspected during the visual inspection are to include all bolted connections, structural members for paint deterioration, latching mechanisms, hinges, limit switches and elastomer seals.

The escape latch seals are to be carefully inspected for cracks, blemishes, or any other indications of deteroration of the elastomer and for properly seating at the sealing surfaces.

3.8.3.8 Environmental Effects

The atmosphere in the ice bed environment is at 10 °F and the absolute humidity is very low. Therefore, corrosion of uncoated carbon steel is negligible.

To ensure that corrosion is minimized while the components of the ice condenser are in storage at the site or in operation in the containment, components are galvanized, painted, protective coatings installed, or placed in a protective container. Galvanizing is in accordance with ASTM A123 or A386.

Materials such as stainless steels with low corrosion rates shall be used without protective coatings.

Corrosion has been considered in the detailed design of the ice condenser components, and it has been determined that the performance characteristics of the ice condenser materials of construction are not impaired by long-term exposure to the ice condenser environment.

Since metal corrosion rates are directly proportional to temperature and humidity, corrosion of ice condenser components at operating temperatures has been assumed to be almost nonexistent. Data available in the open literature does not reflect the exact temperature range and chemistry conditions that are expected to exist in the ice condenser, but does indicate that corrosion rates decreased with decreasing temperatures for the materials and conditions being considered. Although the data in the literature indicated that corrosion of components is not expected, Westinghouse has chosen to employ several preventive measures in the construction of the ice condenser system. To inhibit corrosion, galvanizing is used on the ice baskets. Westinghouse has performed tests which show that galvanized material would not be expected to fail due to corrosion during a 40-year exposure to a 5 - 15° F ice condenser refrigerated air environment. Other structural members are galvanized, protected by corrosion resistant paints that meet the requirements of ANSI 101.2-1972 (Protective Coatings [Paints] for Light Water Nuclear Reactor Containment Facilities) as a minimum, or were constructed of stainless steel, or self-passivating steel. Heavy plate and structural fabrications may be installed in the blasted and/or bare condition.

REFERENCES

(1) TVA Nuclear Quality Assurance Plan, TVA-NQA-PLN89-A.

(2) Concrete Quality Evaluation, CEB-86-19-C.

Table 3.8.3-1 Loading Combinations, Load Factors And Allowable Stresses For Interior Concrete Structure (Sheet 1 of 2)

Loading Combination	Load												Allowable Stresses		
	D	$L^{(1)}$	T_o	T_a	P_a	E	E'	R_o	R_a	Y_j	Y_r	Y_m	fs	fc	V_c (4)
1 Normal	1.0	1.0	1.0			1.0		1.0					0.5fy(2)	0.45fc'(3)	.5(3.5/f'c)
2 Equivalent Test	1.0	1.0			1.0								0.67 fy	0.45 fc'	.5(3.5/f'c)
3 Extreme Environmental	1.0	1.0	1.0				1.0	1.0					0.9 fy	0.75 fc'	3.5/f'c
4 Abnormal	1.0	1.0		1.0	1.5				1.0				0.9 fy	0.75 fc'	3.5/f'c
5 Abnormal	1.0	1.0		1.0	1.25				1.0	1.0	1.0	1.0	0.9 fy	0.75 fc'	3.5/f'c
6 Abnormal/severe Environmental	1.0	1.0		1.0	1.25	1.25			1.0	1.0	1.0	1.0	0.9 fy	0.75 fc'	3.5/f'c
7 Abnormal/extreme Environmental	1.0	1.0		1.0	1.0		1.0		1.0	1.0	1.0	1.0	0.9 fy	0.75 fc'	3.5/f'c

(1) Includes All Temporary Construction Loading

(2) fy = Specified Yield Strength Of Reinforcement

(3) fc' = Specified Compressive Strength Of Concrete

(4) V_c = This Is Maximum Allowable Shear Stress, Carried By The Concrete, Which May Be Reduced Depending On The Section And Type Of Loading, Ref. Aci 359 As Issued For Trial Use April 1973.

Load Nomenclature:

D - Dead Loads, Or Their Related Internal Moments And Forces

L - Live Loads, Or Their Related Internal Moments And Forces

T_0 - Operational Temperature Loads

T_a - Accident Temperature Loads

Table 3.8.3-1 Loading Combinations, Load Factors And Allowable Stresses For Interior Concrete Structure (Sheet 2 of 2)

P_a - Accident Maximum Differential Pressure

E - Operational Basis Earthquake

E' - Safe Shutdown Earthquake

R_0 - Pipe Reaction During Operating Conditions

R_a - Pipe Reaction Due To Increased Temperature Resulting From The Design Accident

Y_j - Jet Impingement Due To Fluid Discharge From Broken Pipe

Y_r - Pipe Reaction Due To Fluid Discharge From Broken Pipe

Y_m - Missile Impingement Load

Table 3.8.3-2 Summary of Representative Maximum Stresses for the Interior Concrete Structure[1, 5]

(Page 1 of 2)

Design Feature	Load	Flexure[2,3]						Shear[4]		
For Location See Figures 3.8.3-1 Thru 7	Comb.[6]	f_c ACTL	f_s ACTL	f_s' ACTL	f_cALL	f_sALL	f_s' ALL	vACTL	vALL	V_c
Pressurizer Compt At Crane Wall	4	3.5	49.3	9.1	3.75	54	54	148	168	106
St Gen Compt, Sidewall At Crane Wall	6	2.4	49.2	24.0	3.75	54	54	254	290	106
St Gen Compt, Center Wall At Crane Wall	7	2.3	52.6	15.1	3.75	54	54	82	160	79
Ice Cond. Compt, End Wall At Crane Wall	4	3.5	39.9	18.4	3.75	54	54	207	275	177
Fill Slab El 702.78 At Crane Wall	6	3.1	50.9	13.5	3.75	54	54	306	319	155
Floor El 756.63 At St Gen Compt Wall	6	2.9	49.5	22.9	3.75	54	54	489	530	164
Crane Wall At El 702.78 Fill Slab	6	2.9	51.1	8.7	5.06	54	54	454	462	67
Crane Wall At Ice Cond Columns	6	3.6	52.2	12.9	6.0	54	54	776	972	99
Crane Wall At St Gen Compt Sidewall	6	2.4	51.0	10.3	5.06	54	54	681	850	164
Reactor Cavity Wall-4.25 Feet Thickness	6	1.9	46.7	5.6	5.06	54	54	481	485	147
Compt Above Reactor-reactor Cavity Columns	4	1.2	40.4	2.4	5.06	54	54	532	681	97
Refueling Canal Wall At Canal Floor Slab	6	1.1	34.9	2.2	5.06	54	54	351	473	144
Refueling Canal Floor Slab	1	0.6	27.4	0.8	3.04	30	30	163	168	60
Ice Cond Support Floor-el 744.5	4	4.2	49.0	13.0	6.0	54	54	569	594	179
Canal Gate	6	4.2	50.2	19.1	5.06	54	54	341	347	164
Control Rod Drive (Crd) Missile Shield	6	4.0	50.5	16.7	5.06	54	54	295	311	164
Reac Cool Pump & Lower Compt Access Hatches	6	2.0	46.1	4.4	5.06	54	54	376	408	164

Table 3.8.3-2 Summary of Representative Maximum Stresses for the Interior Concrete Structure[1, 5]

(Page 2 of 2)

Notes:

(1) FLEXURAL STRESSES ARE IN KIPS PER SQ IN (KSI) SHEAR STRESSES ARE IN POUNDS PER SQ IN (PSI)

(2) fc_{ACTL}, fs_{ACTL}, fs'_{ACTL} - THE ACTUAL CALCULATED STRESS IN THE CONCRETE, TENSION REINFORCING STEEL AND COMPRESSION REINFORCING STEEL, RESPECTIVELY.

(3) fc_{ALL}, fs_{ALL}, fs'_{ALL} - THE ALLOWABLE STRESS IN THE CONCRETE, TENSION REINFORCING STEEL AND COMPRESSION REINFORCING STEEL, RESPECTIVELY.

(4) V_{ACTL} - THE ACTUAL CALCULATED SHEAR STRESS IN THE STRUCTURE.

V_{ALL} - THE TOTAL ALLOWABLE SHEAR STRESS THE SECTION CAN CARRY TO INCLUDE THE ALLOWABLE SHEAR STRESS CARRIED BY THE CONCRETE AS WELL AS THAT PROVIDED BY SHEAR REINFORCING.

V_c - THE ALLOWABLE SHEAR STRESS CARRIED BY THE CONCRETE ONLY.

(5) THIS TABLE DOES NOT REFLECT THE EVALUATIONS DOCUMENTED IN REPORT CEB 86-19-C. TABULATED STRESSES ARE FROM THE ORIGINAL CALCULATIONS. CHANGES HAVE BEEN DOCUMENTED IN CALCULATION PACKAGES.

(6) FOR LOAD COMBINATION DEFINITIONS, REFER TO TABLE 3.8.3-1.

Table 3.8.3-3 Personnel Access Doors in Crane Wall Loads, Loading Combinations, And Allowable Stresses (Page 1 of 3)

Normal operating conditions are as follows:

Pressure	-	Negligible
Temperature	-	30° to 120°F
Radiation	-	2.0x 10^7 rads for 40 year life

The effect of pipe rupture accidents on the doors varied with the location and intensity of the accidents. The three types of pipe accidents producing maximum effect on the doors and conditions accompanying these accidents are as follows:

a. Accidents Without Jets or Missles Hitting the Doors

Temperature	327° F for first hour 225° F for next 11 hours
Radiation	8.7 x 10^7 rads total for 12 hours
Pressure	12 psig acting from inside crane wall for 12 hours

b. Accidents With Jet Hitting a Door

Temperature	700°F maximum
Force and impact	As produced by maximum jet
Radiation	4.8 x 10^6 rads per hour (gamma) 2.5 x 10^7 rads per hour (beta)

Duration of maximum temperature and maximum force from jet is for not more than 10 seconds and then gradually decreases. Pressure and temperature after maximum temperature and force are as outlined in (a) above.

c. Accidents with Missile and Jet from the Same Source Striking a Door

Temperature	700° F maximum
Force and impact	As produced by jet and missile
Radiation	4.8 x 10^6 rads per hour (gamma) 2.5 x 10^7 rads per hour (beta)

Duration of maximum temperature and maximum force from jet is for not more than 10 seconds and then gradually decreases. Pressure and temperature after maximum temperature and force are as outlined in (a) above.

Table 3.8.3-3 Personnel Access Doors in Crane Wall Loads, Loading Combinations, And Allowable Stresses (Page 2 of 3)

Potential missiles which the doors were designed to withstand are as follows:

Temperature element A, without well, boss, and pipe

Temperature element B, with well, boss, and pipe

Temperature element C, without well

Temperature element D, with well

Reactor coolant pump terperature element

Pressurizer temperature detector

Pressurizer heater

2-inch check valve (boron injection)

3/4-inch globe valve (sampling system)

(flow transmitters)

(pressure transmitters)

3/4-inch air-operated valve (head gasket monitoring)

1-inch manually-operated globe valve (excess letdown)

Structural Door and Frame Assembly

	Allowable stresses (psi)[1]		
Load Combinations	Tension	Compression[3]	Shear
I. With door closed or open: Dead load plus OBE	0.6 F_y	0.6 F_y	0.4 F_y
II. With door closed or open: Dead load plus SSE	0.9 F_y	0.9 F_y	0.6 F_y
III. With door closed: Dead load plus SSE plus 12 psig from inside of crane wall	0.9 F_y	0.9 F_y	0.6F_y
IV. With door closed: Dead load plus SSE plus Load from maximum jet hitting doors at 615 psi	0.9F_y	0.9 F_y	0.6 F_y
V. With door closed: Dead load plus SSE plus Load from missile with maximum energy 6900 lb/ft hitting door plus jet from that missile source at 295 psi	0.9 F_y	0.9 F_y	0.6 F_y

Table 3.8.3-3 Personnel Access Doors in Crane Wall Loads, Loading Combinations, And Allowable Stresses (Page 3 of 3)

	Allowable Stresses (psi)(1)		
Load Combination	Tension	Compression(3)	Shear
I. With door closed or open: Dead load plus Operator force of 75 pounds	Ult / 5	Ult / 5	2 x Ult / 15
II. With door closed or open: Dead load plus OBE	$0.6F_y$	$0.6F_y$	$0.4F_y$
III. With door closed or open: Dead load plus SSE	$0.9F_y$	$0.9F_y$	$0.6F_y$
	Mechanical Parts Allowable Stresses (psi)(1)		
IV. Load Combinations	Tension	Compression(3)	Shear
With door closed: Dead load plus SSE plus Load from maximum jet hitting doors at 615 psi	$0.9F_y$	$0.9F_y$	$0.6F_y$
V. With door closed: Dead load plus SSE plus Load from missile with maximum energy (6900 lb/ft) hitting door plus jet from that missile source at 295 psi	$0.9F_y$	$0.9F_y$	$0.6F_y$

Notes:

(1) Listed allowable stresses are for non-collapsible members only. Collapsible members are plastically deformed.

(2) Earthquake loads act in one horizontal direction only at any given time and act in vertical and horizontal directions simultaneously.

(3) The value indicated for the allowable compression stresses is the maximum value permitted when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_y when buckling controls.

Table 3.8.3-3
Personnel Access Doors in Crane Wall Loads, Loading Combinations, And Allowable Stresses

$$F_{cr} = F_Y\left(1 - \frac{\frac{Kl}{r}^2}{2C_c^2}\right) \text{ when } \frac{Kl}{r} \le C_c \qquad 1$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \frac{Kl}{r} > C_c \qquad 2$$

Table 3.8.3-4 Ice Condenser Allowable Limits(3)

Load Combination	Elastic Analysis Mechanical(2)	Elastic Analysis Mechanical and Thermal	Fatique	Limit Analysis(1) (Load Factors)	Test (Load Factors)
D + OBE	S(4)	3S	AISC Part I	1.43	1.87
D + DBA	1.33 S	N.A.	N.A.	1.30	1.43
D + SSE	1.33 S	N.A.	N.A	1.30	1.43
D + SSE + DBA	1.65 S	N.A.	N.A	1.18	1.30

Notes:

(1) For mechanical loads only. Mechanical plus thermal expansion, combination and fatigue shall satisfy the elastic analysis limits.

(2) Membrane (direct) stresses shall be no larger than 0.7 S_u (70% of ultimate stress).

(3) For particular components that do not meet these limits specific justification shall be provided on a case by case basis.

(4) S = Allowable stresses as defined in Sections 1.5 and 1.6 of the AISC Part I Specification.

Table 3.8.3-5
(Page 1 of 2)
Selection of Steels in Relation to Prevention of Non-Ductile Fracture of Ice Condenser Components[1]

	Section Thickness	
Properties	5/8 inch thick and under	over 5/8-inch thickness
Energy Absorption Level	None required	i) 20 ft-lb CVN[2] at -20°F for steel over 36,000 psi yield strength
		ii) 15 ft-lb CVN[2] at -20°F for steel under 36,000 psi yield strength
Heat Treatment	None required	i) Normalizing
	Steel can be used in the hot rolled condition	ii) Quench and Temper
Type of Steel	i) Rimmed[3]	i) Killed
	ii) Semi-killed[4]	ii) Killed-fine grain practice
	iii) Killed[4,5]	
	iv) Killed - fine grain practice	

Table 3.8.3-5
(Page 2 of 2)
Selection of Steels in Relation to Prevention of Non-Ductile Fracture of Ice Condenser Components[1]

General Notes:

(1) Hot rolled, normalized or quenched and tempered steels are used where applicable.

(2) Charpy V-notch (CVN) impact testing shall be performed in accordance with the requirements of ASTM A370.

(3) Rimmed steel shall be used only for carbon steel sheet products.

(4) These type steels shall be applied for components which remain within AISC Code stress limits for all load conditions.

(5) Killed steels for above AISC Code stress limits shall be upgraded by heat treatment, e.g., bolting.

Table 3.8.3-6 Equipment Access Hatch Summary of Allowable Stresses for Design Condition

	$I^{(2)}$, $II^{(2)}$ Allowable	$III^{(2)}$ Allowable
Bending stress in structural shapes and plates (F_y = 36,000 psi)	21,600 psi (0.60 F_y)	32,400 psi (0.90 F_y)
Shear stress in structural shapes and plates (F_y = 36,000 psi)	14,400 psi (0.40 F_y)	21,600 psi (0.60 F_y)
Tensile stress in anchor bolts (F_y = 36,000 psi)	19,800 psi (0.55F_y)	31,700 psi (1.6(0.55)F_y)
Bearing stress under anchor bolt end plate (Fc' = 5,000 psi)	1,250 psi (0.25 Fc'[(1)])	

Notes:

(1) See Table 1002(a), ACI 318-63 Code

(2) I = DL + L1 or DL + L2

II = DL + L1 + OBE or DL + L2 + OBE

III = DL + L1 + SSE or DL + L2 + SSE

L1 = Live load of 14,000 lb (loaded weight of forklift)

L2 = Live load of 15 psi pressure from below (LOCA)

Table 3.8.3-7
Escape Hatch - Divider Barrier Floor Load Combinations - Allowable Stresses Structural Parts - (F_y - 36,000 psi) (Sheet 1 of 2)

		Allowable Stress (psi)		
No.	Load Combinations	Tension	Compression(2)	Shear
	Hatch Closed			
I.	Dead load Live load at 100 lb/ft^2 Load from latching device	18,000 (0.5 F_y)	18,000 (0.5 F_y)	12,000 (0.33F_y)
II.	Dead load Live load of 15 psi from below Load from latching device SSE(1)	25,900 (0.72 Fy)	25,900 (0.72 Fy)	17,300 (0.48 F_y)
	Hatch Open			
III.	Dead load OBE(1)	22,000 (0.6F_y)	22,000 (0.6Fy)	14,400 (0.4Fy)
IV.	Dead load SSE(1)	25,900 (0.72 F_y)	25,900 (0.72 F_y)	17,300 (0.48 F_y)
Mechanical Parts(3) (Excluding Springs)				
	Hatch Closed			
I.	Dead load Live load at 100 lb/ft^2 Load from latching device	$\frac{\text{Ultimate}}{5}$	$\frac{\text{Ultimate}}{5}$	$\frac{2}{3} \times \frac{\text{Ultimate}}{5}$
II.	Dead load Live load of 15 psi from below Load from latching device SSE	0.72 yield	0.72 yield	$\frac{2}{3}$ x 0.72 yield
	Hatch Open			
III.	Dead Load OBE	0.6F_y	0.6F_y	0.4F_y
IV.	Dead Load SSE	0.9F_y	0.9F_y	0.6F_y

Notes:

(1) Acts in one horizontal direction only at any given time and acts in vertical and horizontal directions simultaneously.

(2) The value given for allowable compression stress is the maximum value permitted, when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_y when buckling controls.

Table 3.8.3-7
(Sheet 2 of 2)
Escape Hatch - Divider Barrier Floor Load Combinations - Allowable Stresses
Structural Parts - (F_y - 36,000 psi)

$$F_{cr} = F_Y\left(1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right) \text{when} \qquad \frac{kl}{r} \le C_c \qquad 1$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{when} \qquad \frac{kl}{r} > C_c \qquad 2$$

(3) Pins and shafts, bolts and nuts, bushings, and seals.

Table 3.8.3-8 Air Return Duct Penetration Summary of Stresses for Controlling Design Condition DB LOCA - DI + SSE

	Calculated	Allowable
Bending stress in structural shapes and plates (F_y = 36,000 psi)	17,900 psi	21,600 psi (0.60 F_y)
Tensile stress in structural shapes and plates (F_y = 36,000 psi)	1,890 psi	21,600 psi (0.60 $F_{y)}$
Headed concrete anchors (shear) (f`’s = 60,000 psi)	17,000 psi	27,000 psi (0.45 f`’s)

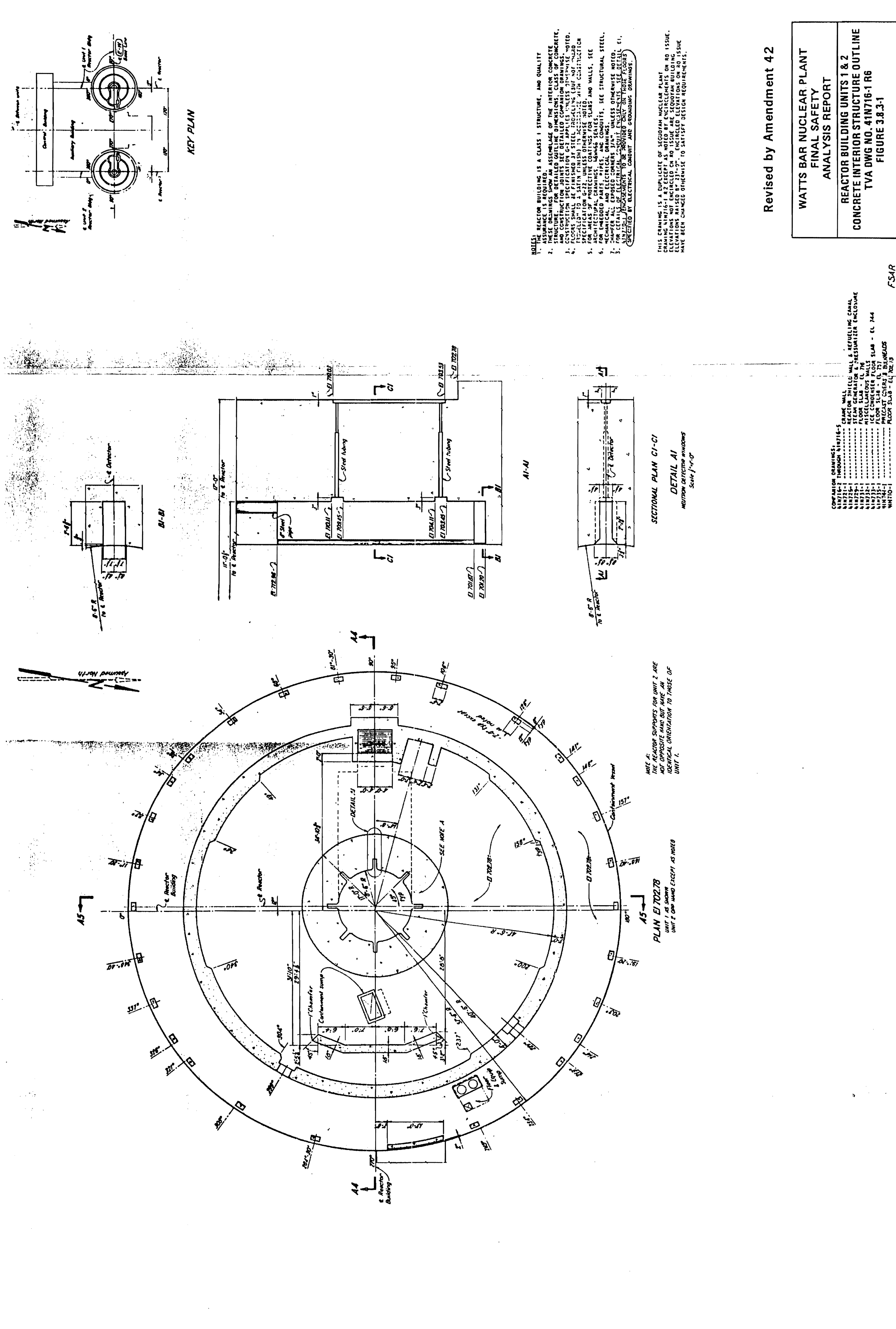


Figure 3.8.3-1 Reactor Building Units 1 & 2 Concrete Interior Structure Outline

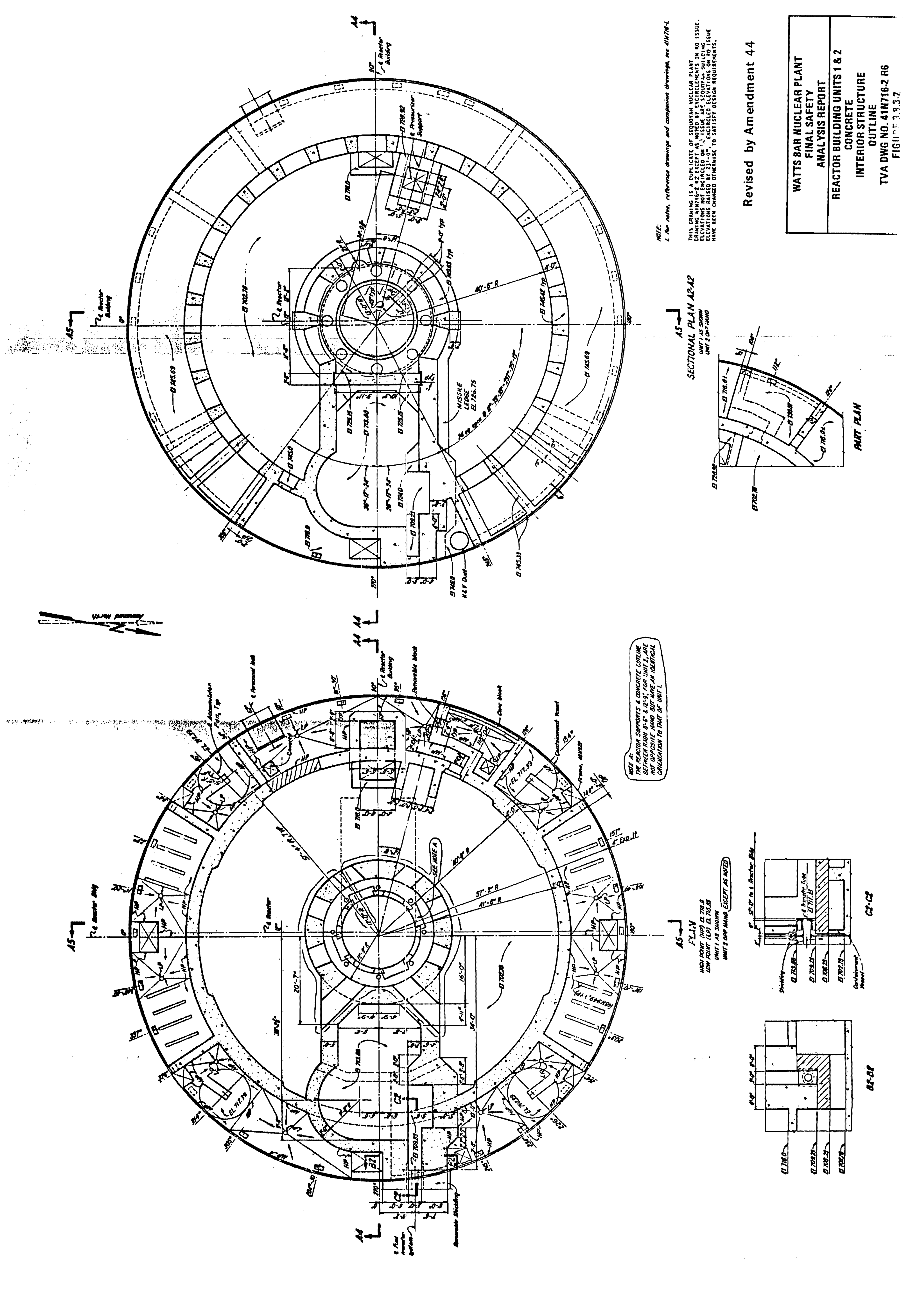


Figure 3.8.3-2 Reactor Building Units 1 & 2 Concrete Interior Structure Outline

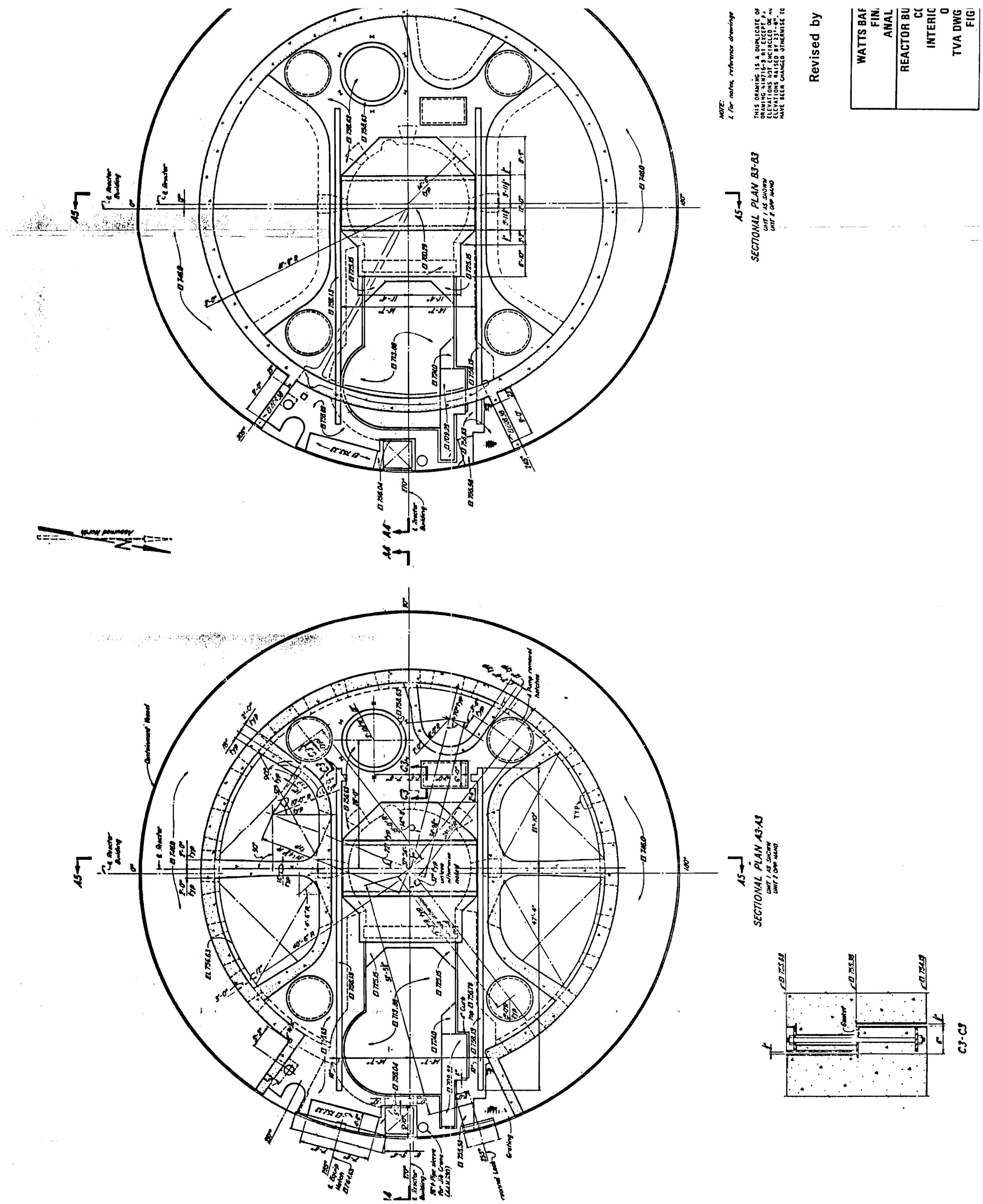


Figure 3.8.3-3 Reactor Building Units 1 & 2 Concrete Interior Structure Outline

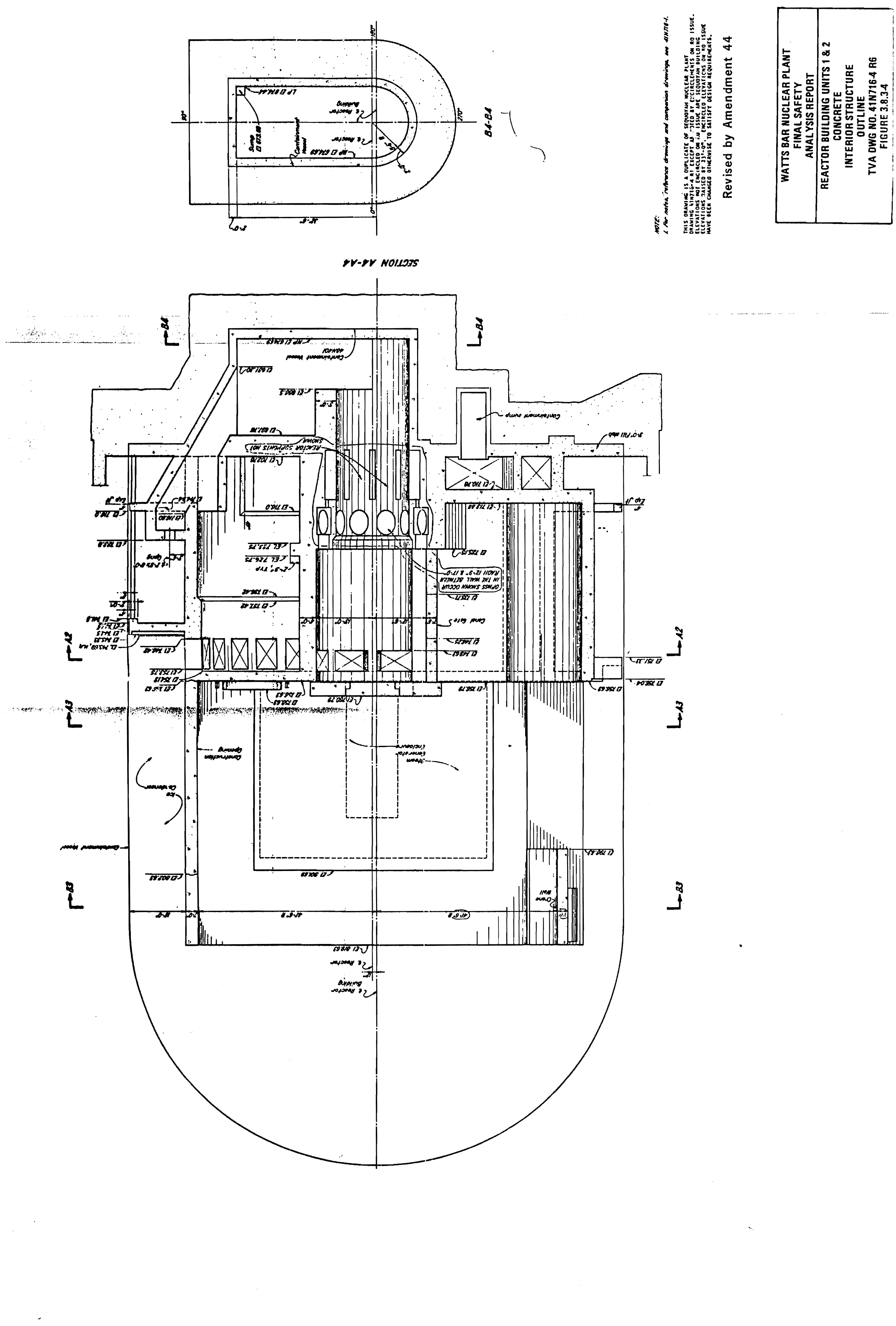
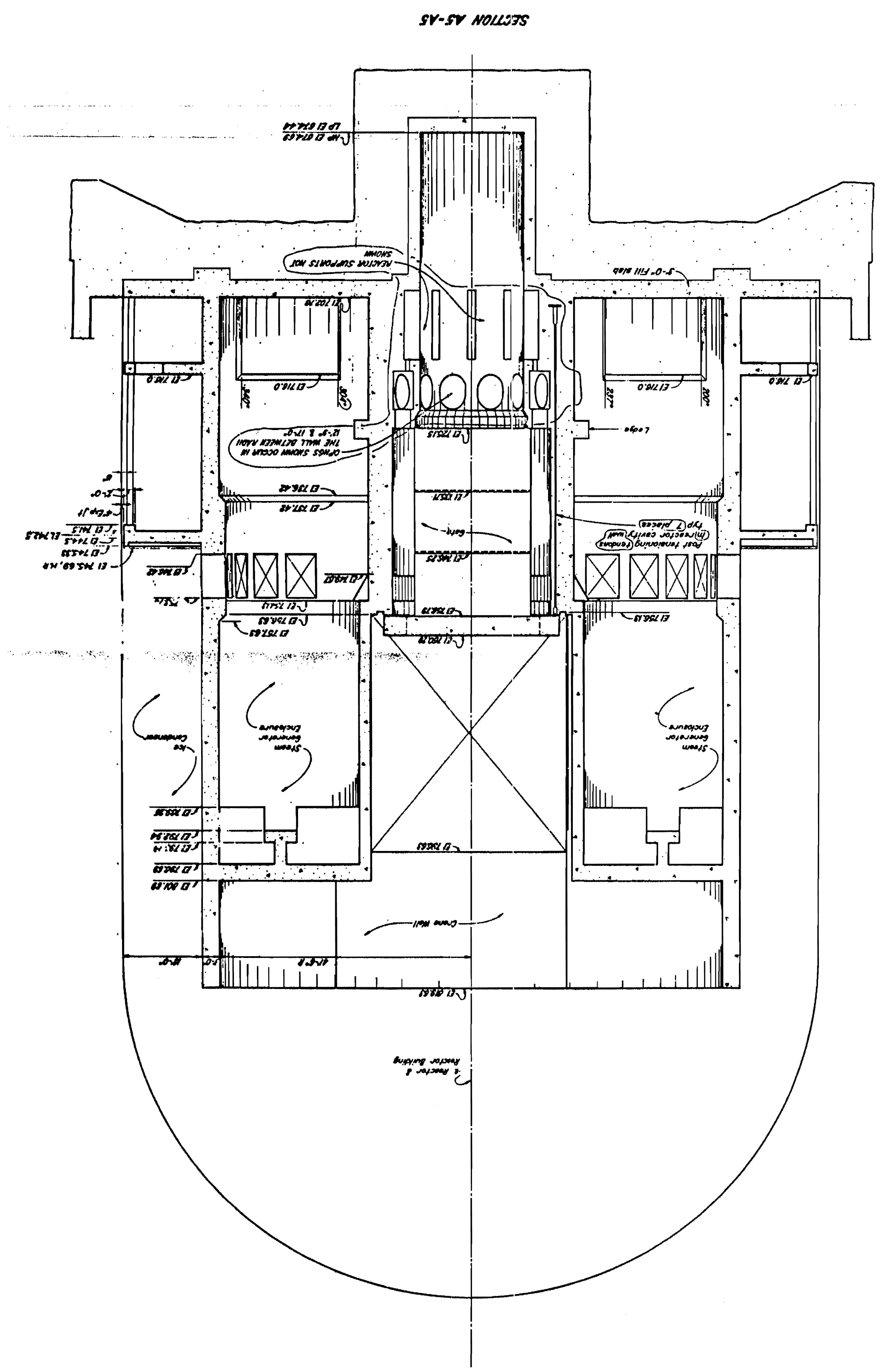


Figure 3.8.3-4 Reactor Building Units 1 & 2 Concrete Interior Structure Outline



NOTE:
1. For notes, reference drawings and companion drawings, see 41N716-1.

THIS DRAWING IS A DUPLICATE OF SEQUOYAH NUCLEAR PLANT DRAWING 41N716-5 R2 EXCEPT AS NOTED BY ENCIRCLEMENTS ON RO ISSUE. ELEVATIONS NOT ENCIRCLED ON RO ISSUE ARE SEQUOYAH BUILDING ELEVATIONS RAISED BY 23'-0". ENCIRCLED ELEVATIONS ON RO ISSUE HAVE BEEN CHANGED OTHERWISE TO SATISFY DESIGN REQUIREMENTS.

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Figure 3.8.3-5 Reactor Building Units 1 & 2 Concrete Interior Structure Outline

Figure 3.8.3-6 Plan-Upper Compartment; 1Ft. = 0.3048m

Figure 3.8.3-7 Plan-Lower Compartment; 1Ft. = 0.3048m

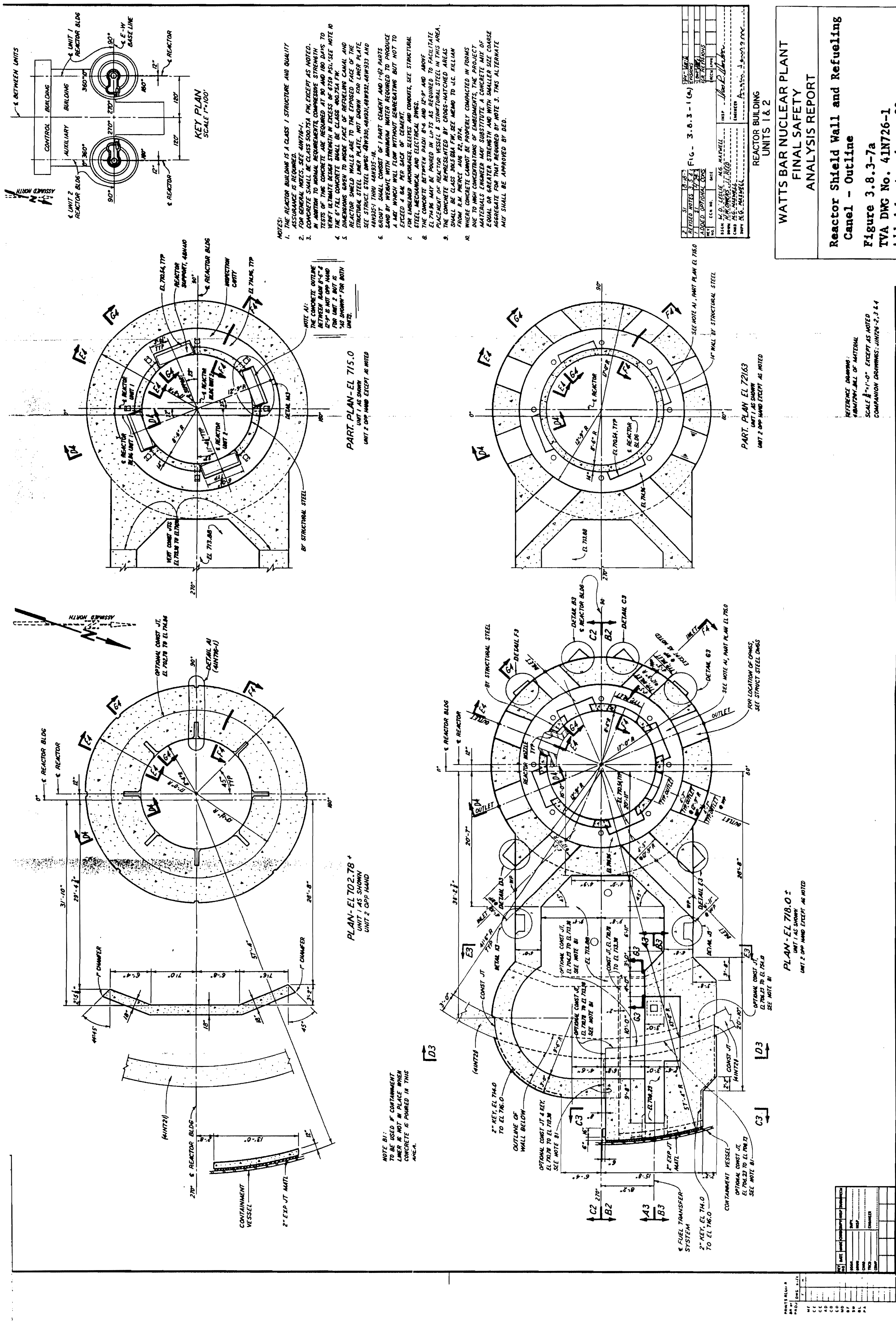


Figure 3.8.3-7a Reactor Shield Wall and Refueling Canal - Outline

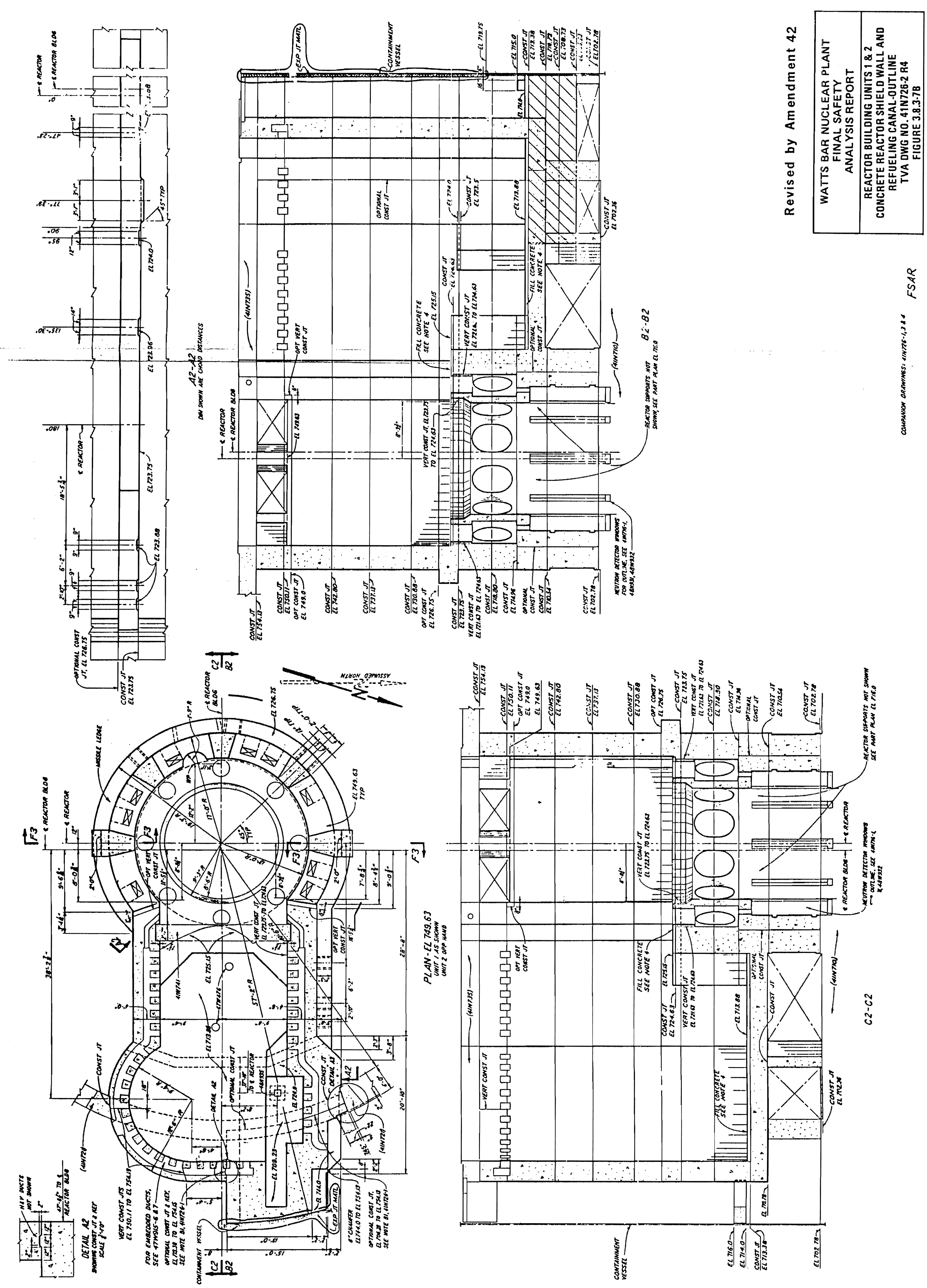


Figure 3.8.3-7b Reactor Building Units 1 & 2 Concrete Reactor Shield Wall and Refueling Canal - Outline

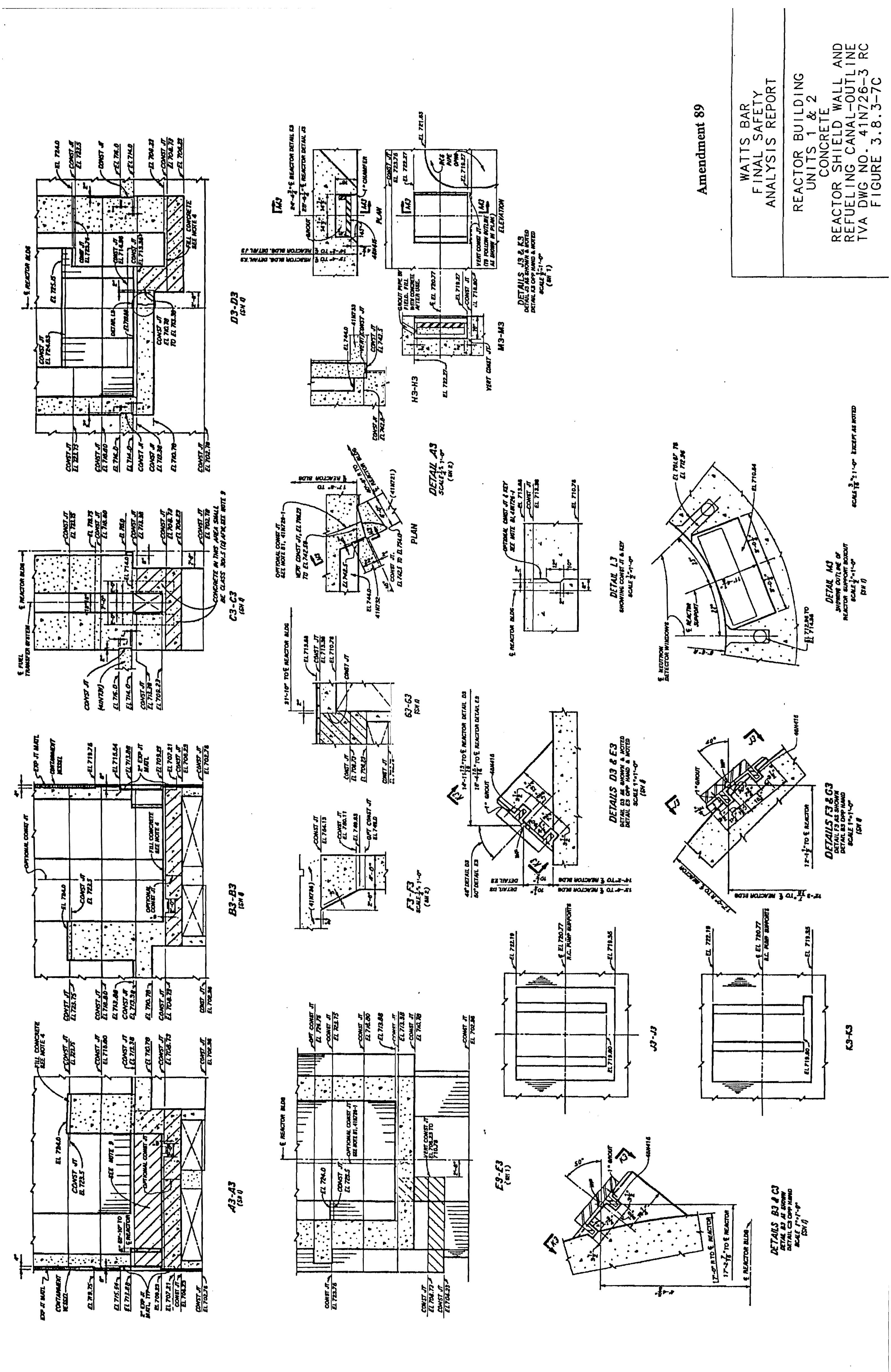


Figure 3.8.3-7c Reactor Building Units 1 & 2 Concrete Reactor Shield Wall and Refueling Canal - Outline

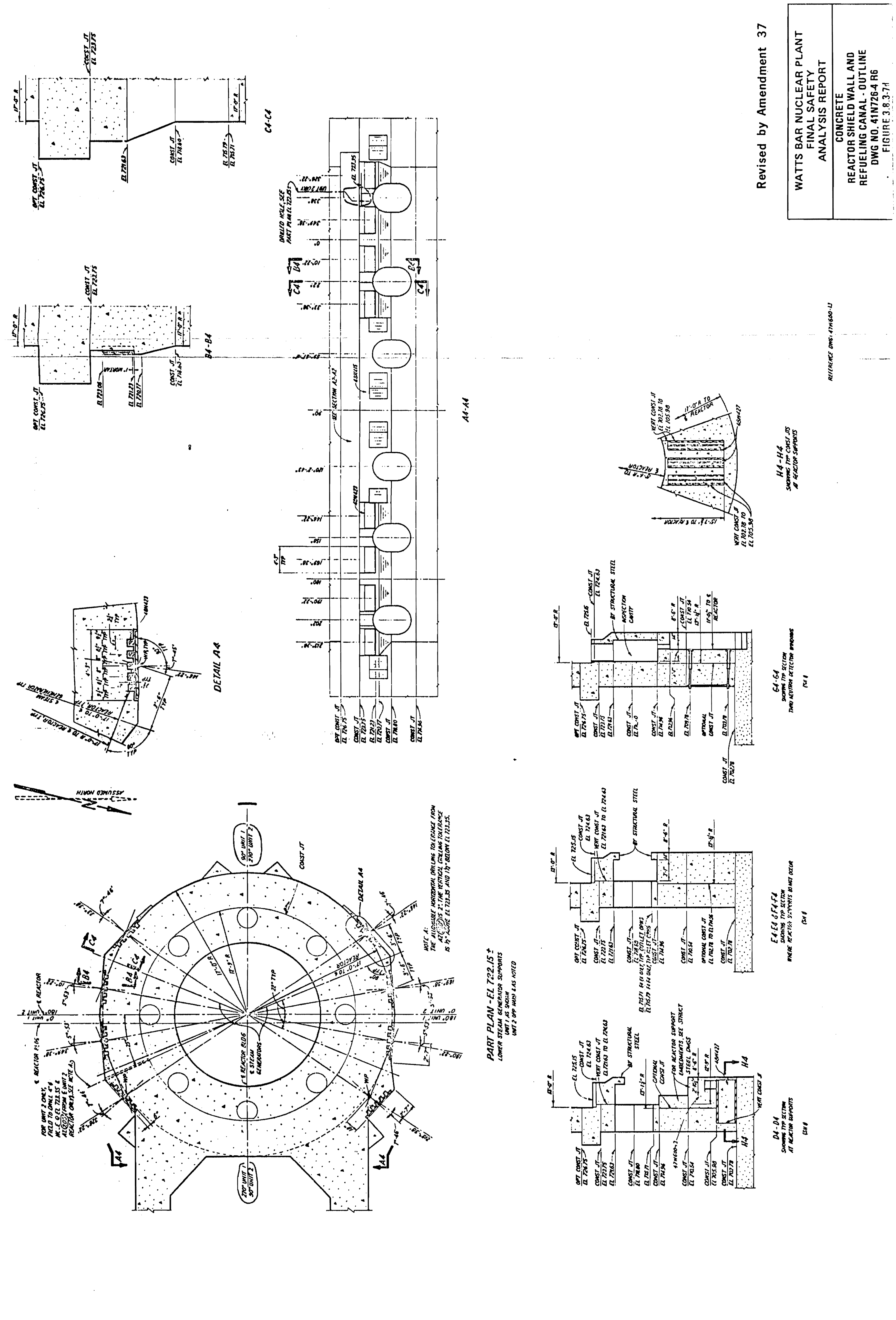


Figure 3.8.3-7d Concrete Reactor Shield Wall and Refueling Canal - Outline

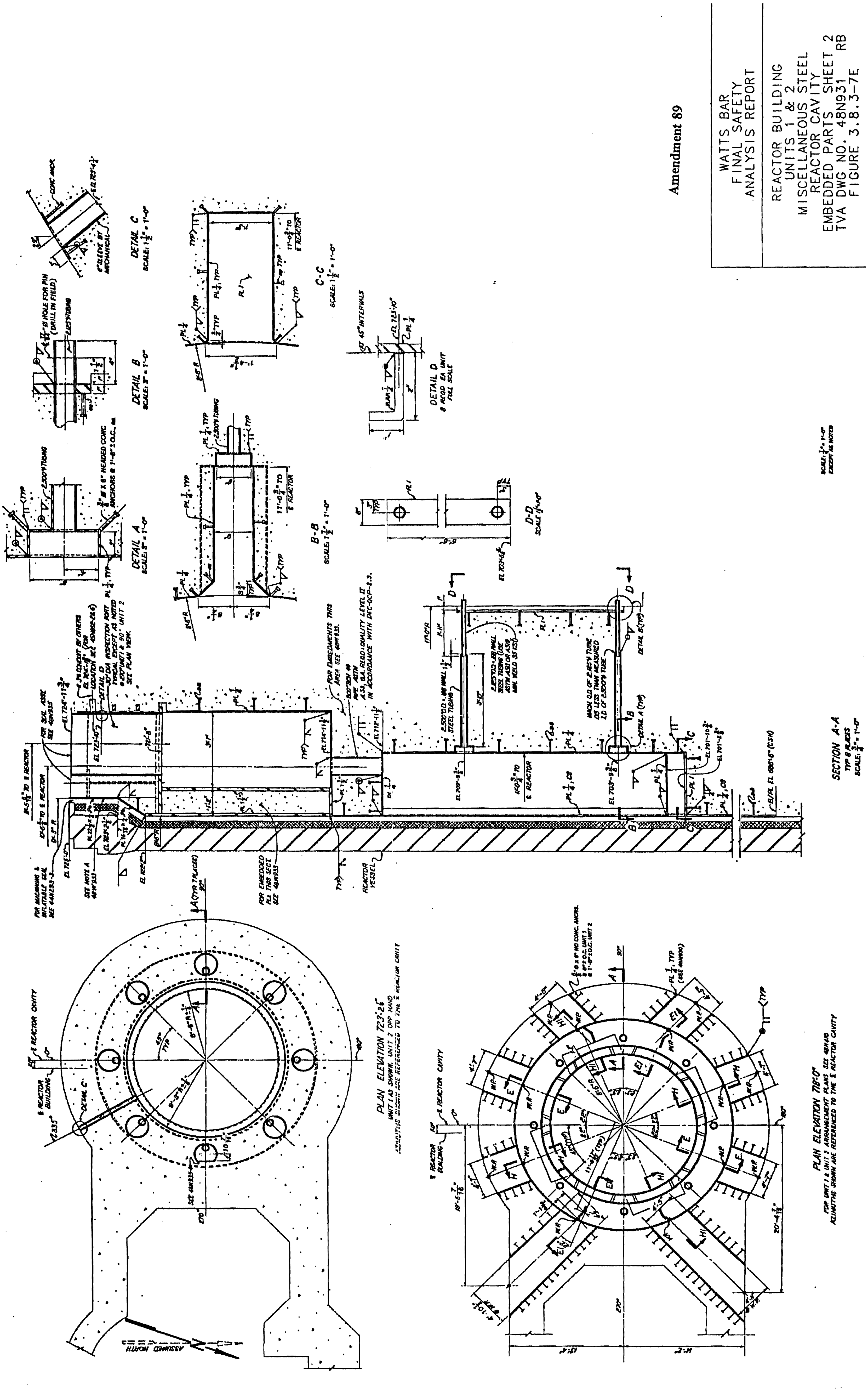


Figure 3.8.3-7e Reactor Building Units 1 & 2 Miscellaneous Steel Reactor Cavity Embedded Parts -Sheet 2

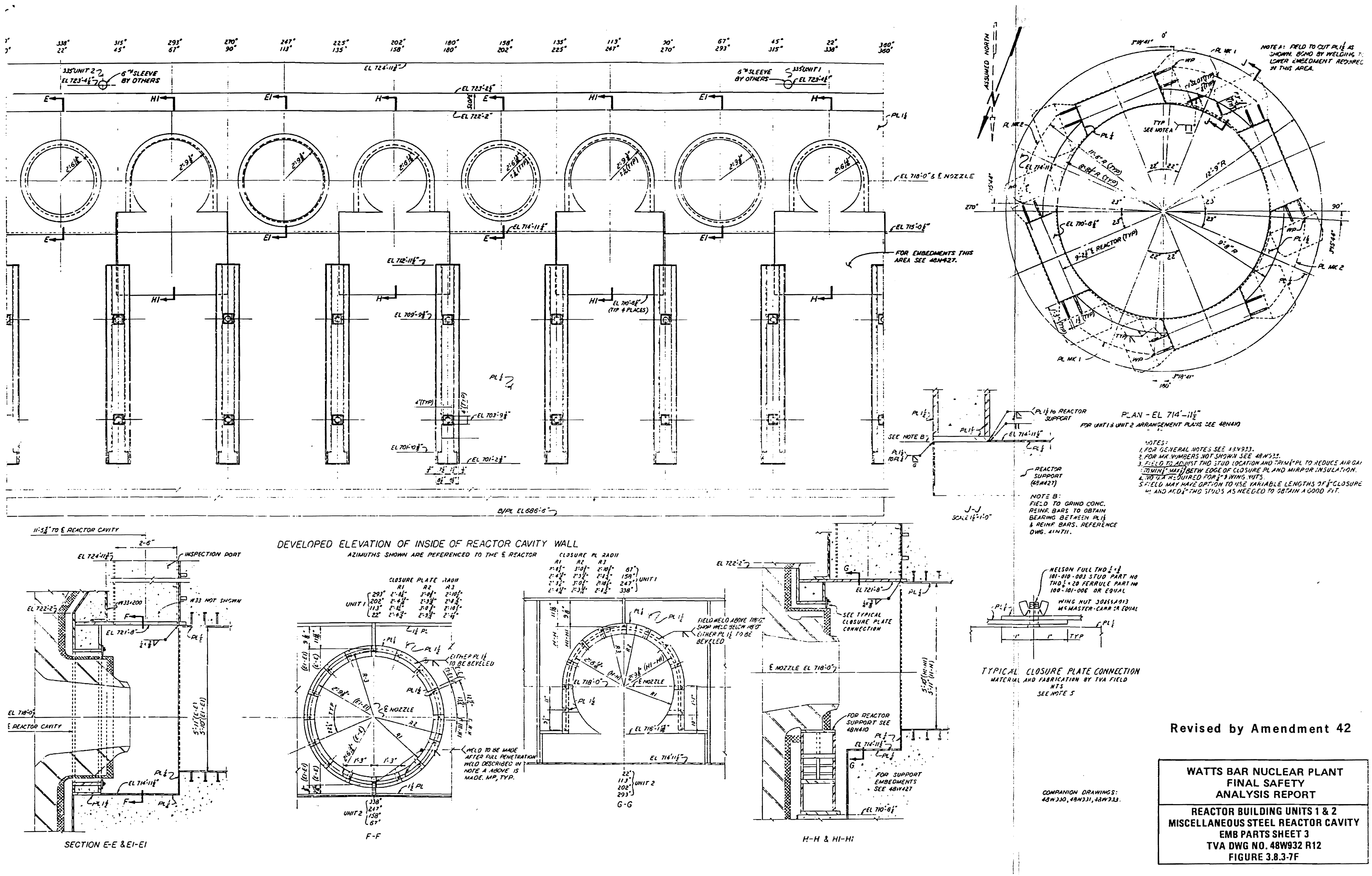
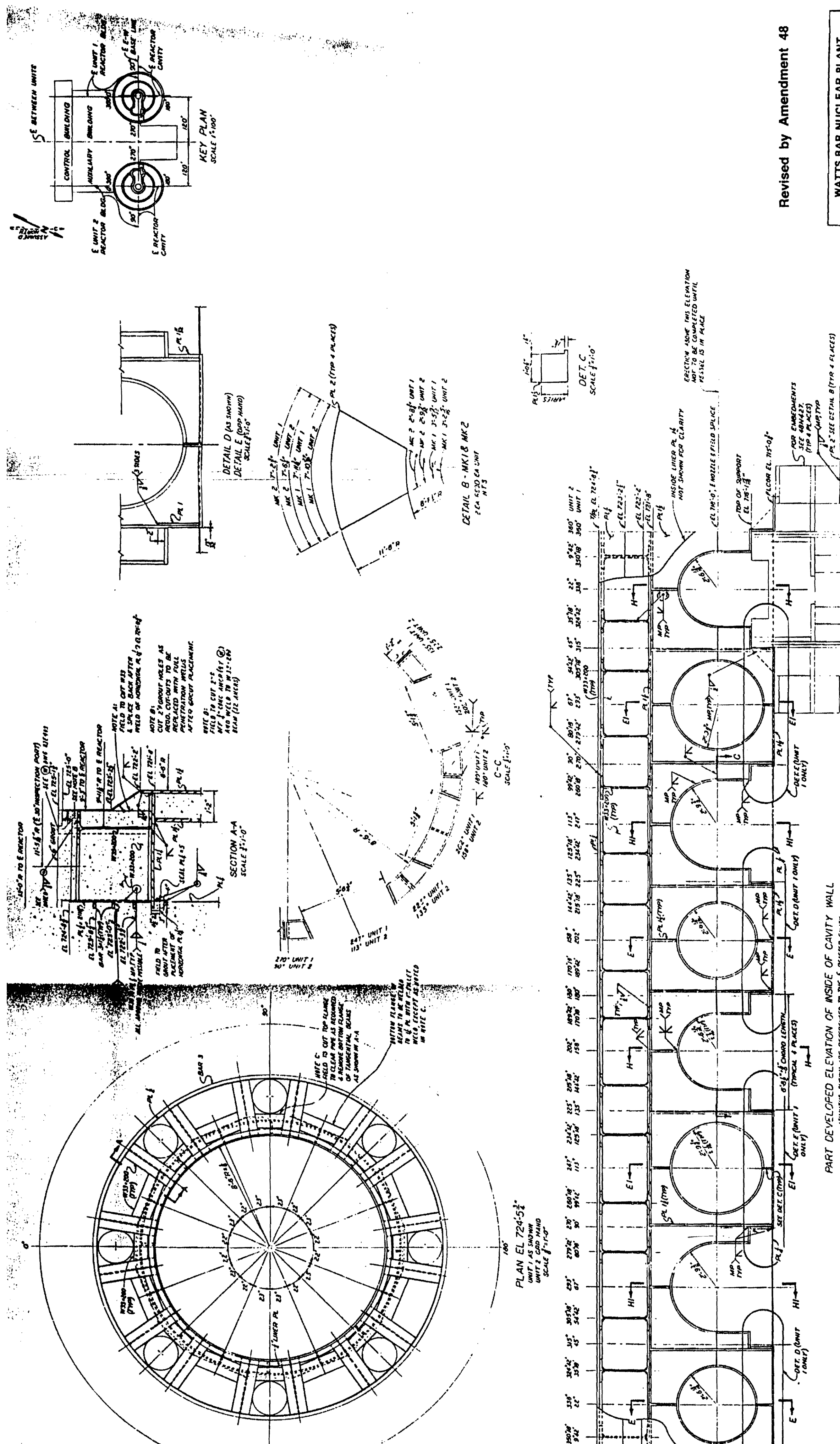


Figure 3.8.3-7f Reactor Building Units 1 & 2 Miscellaneous Steel Reactor Cavity Embedded Parts - Sheet 3



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WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

REACTOR BUILDING UNITS 1 & 2
MISCELLANEOUS STEEL REACTOR CAVITY
EMBEDDED PARTS – SHEET 4
TVA DWG NO. 48W933 R10
FIGURE 3.8.3-7G

Figure 3.8.3-7g Reactor Building Units 1 & 2 Miscellaneous Steel Reactor Cavity Embedded Parts - Sheet 4

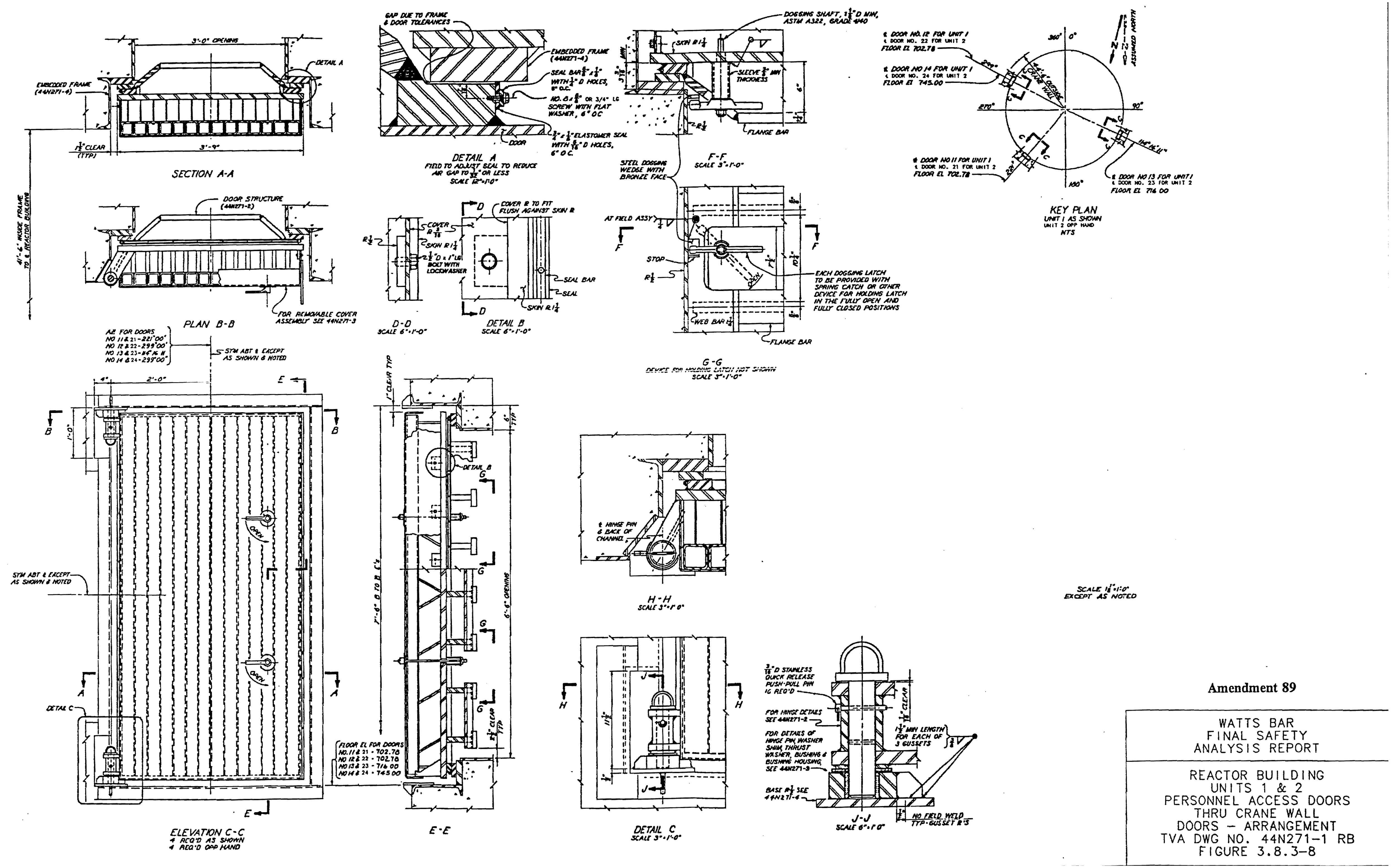

Figure 3.8.3-8 Reactor Building Units 1 & 2 Personnel Access Doors Thru Crane Wall Doors - Arrangement

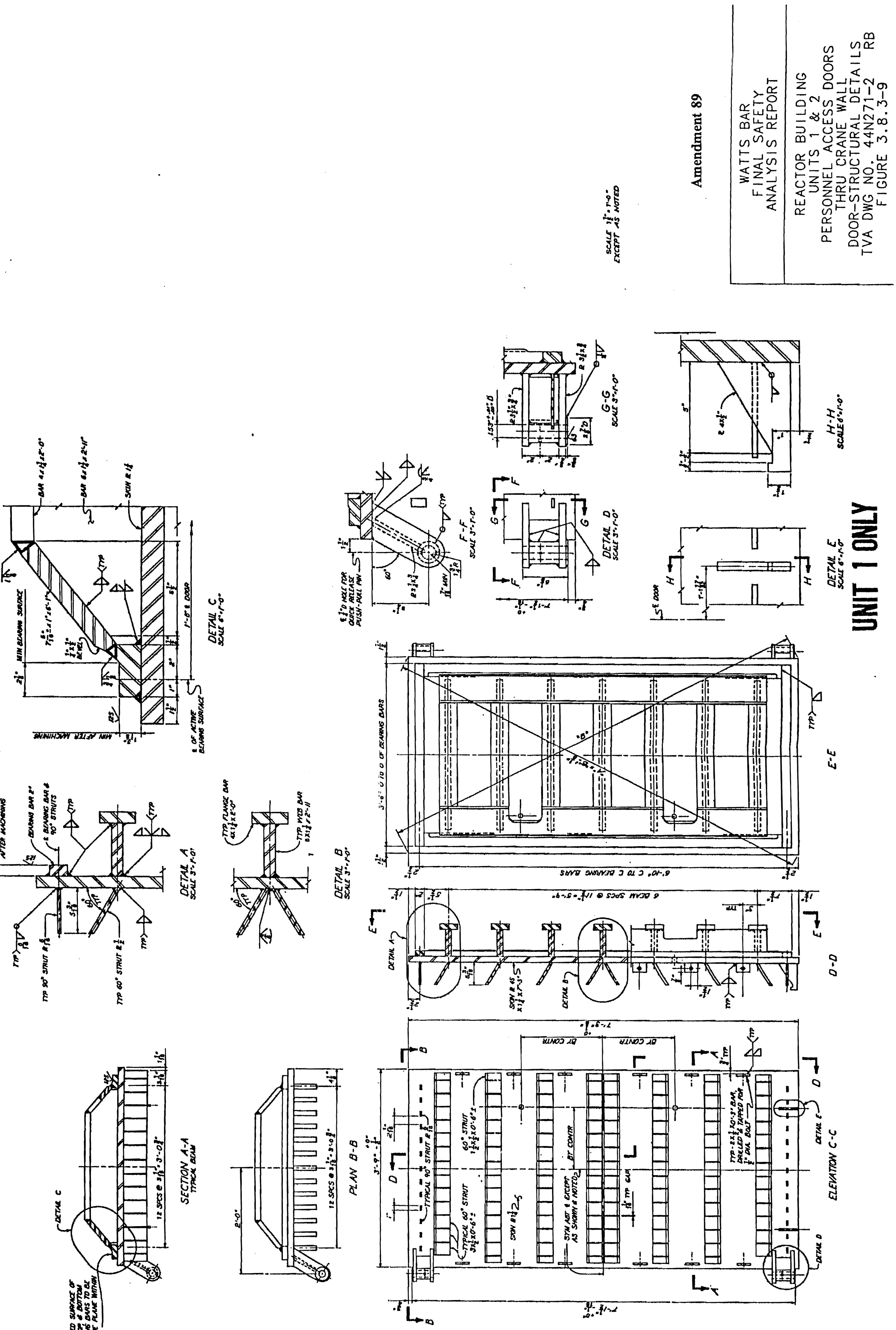

Figure 3.8.3-9 Reactor Building Units 1 & 2 Personnel Access Doors Thru Crane Wall Door - Structural Details

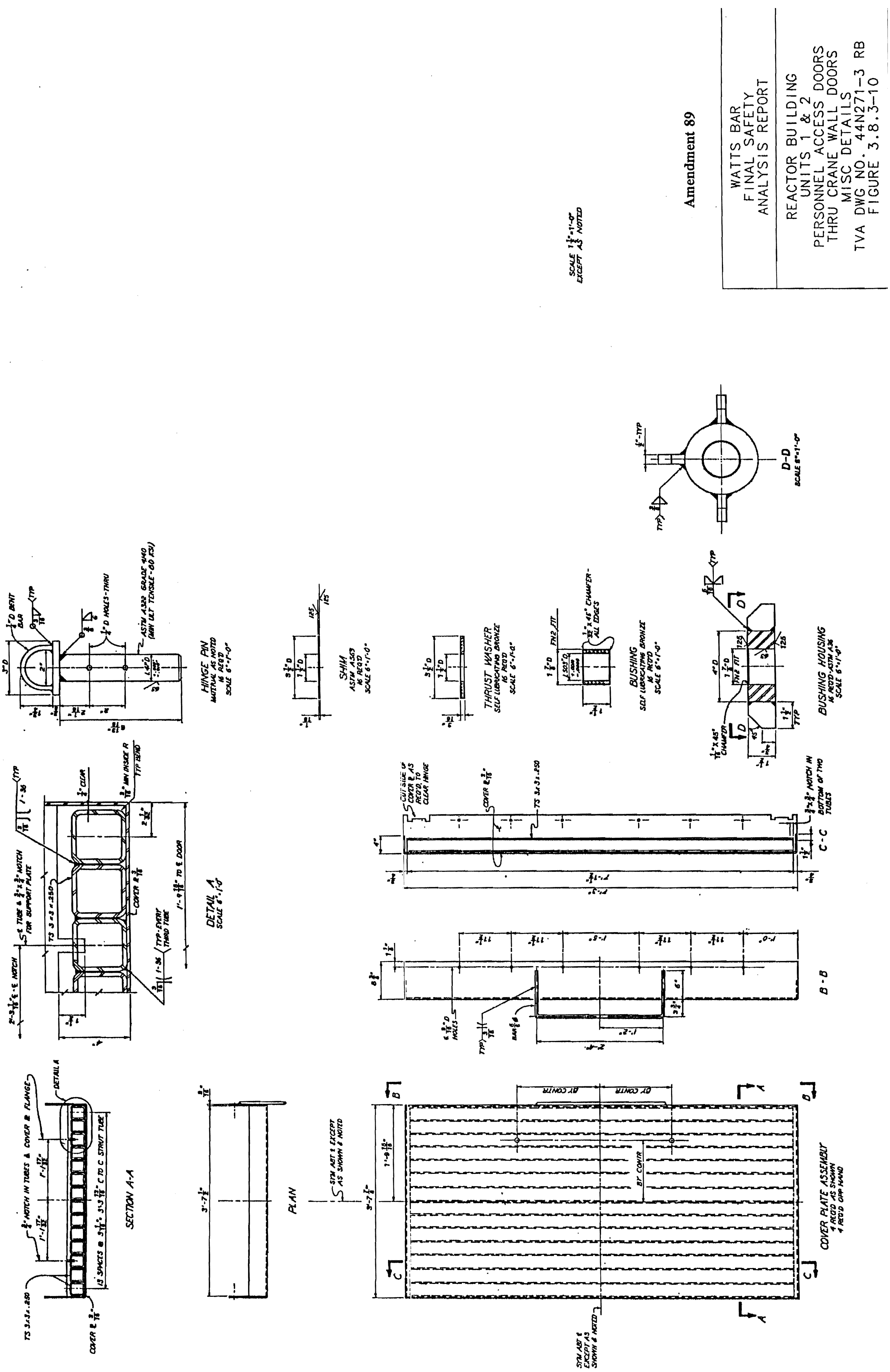

Figure 3.8.3-10 Reactor Building Units 1 & 2 Personnel Access Doors Thru Crane Wall Doors Misc. Details

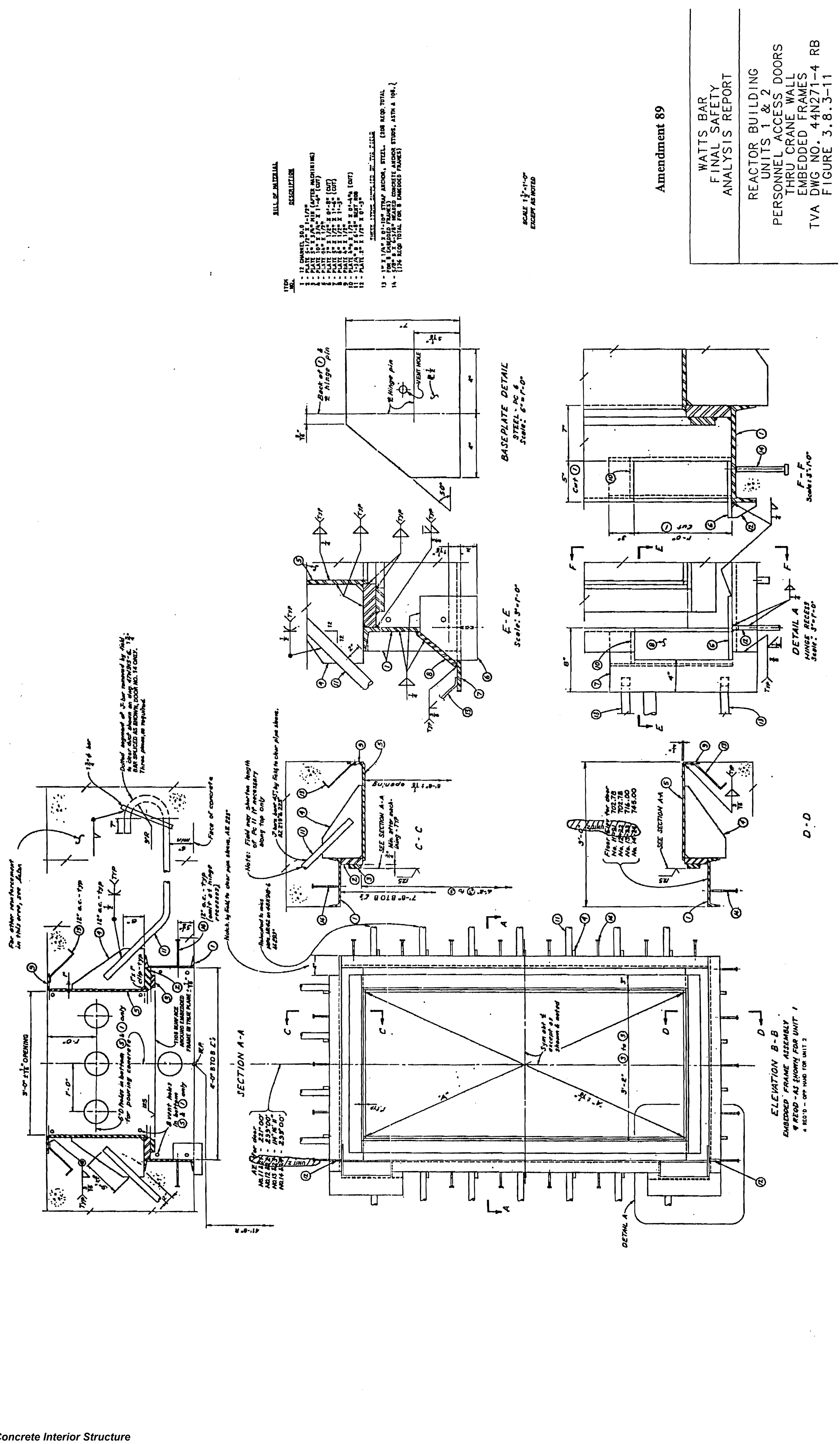

Figure 3.8.3-11 Reactor Building Units 1 & 2 Personnel Access Doors Thru Crane Wall Embedded Frame

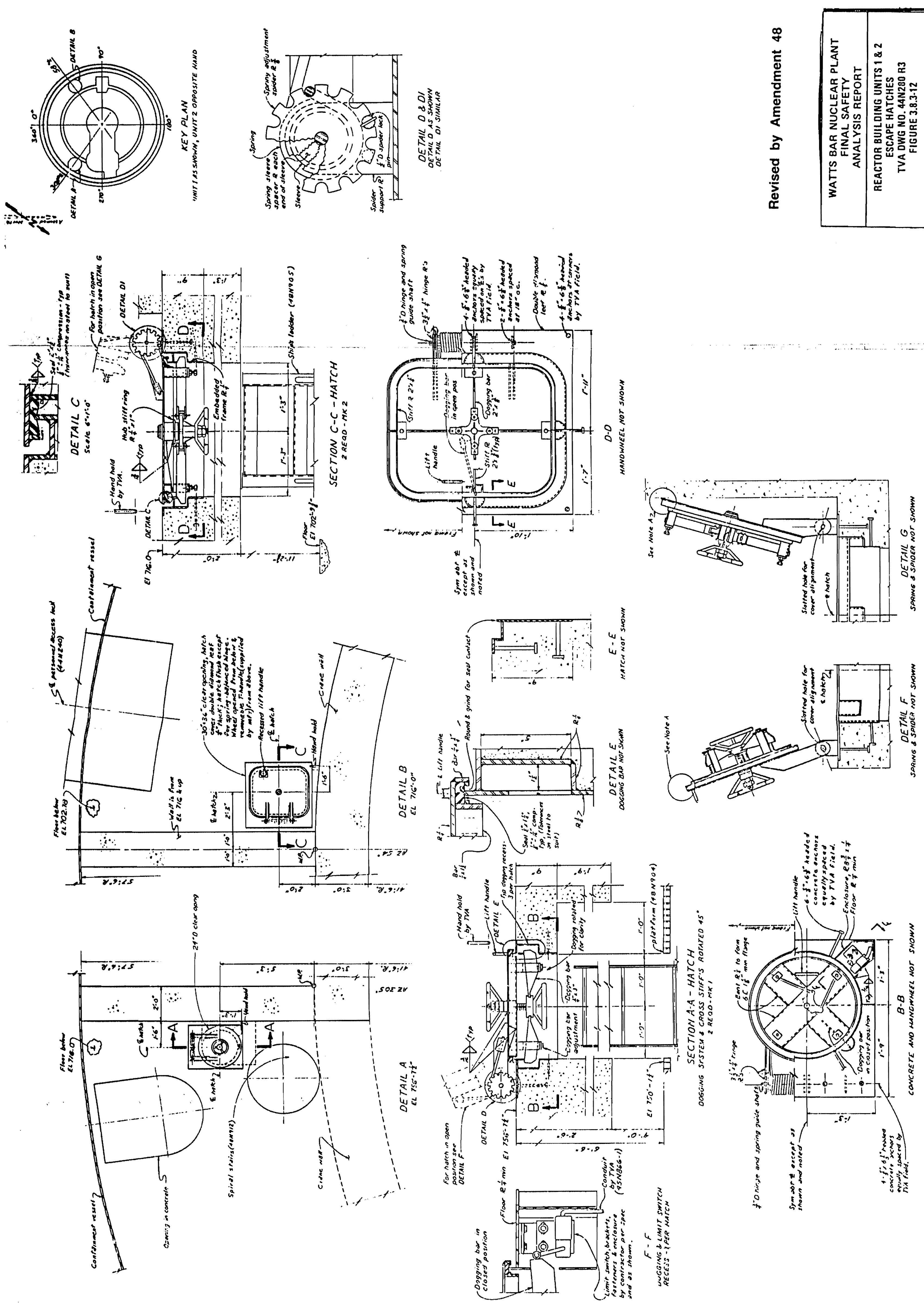

Figure 3.8.3-12 Reactor Building Units 1 & 2 Escape Hatches

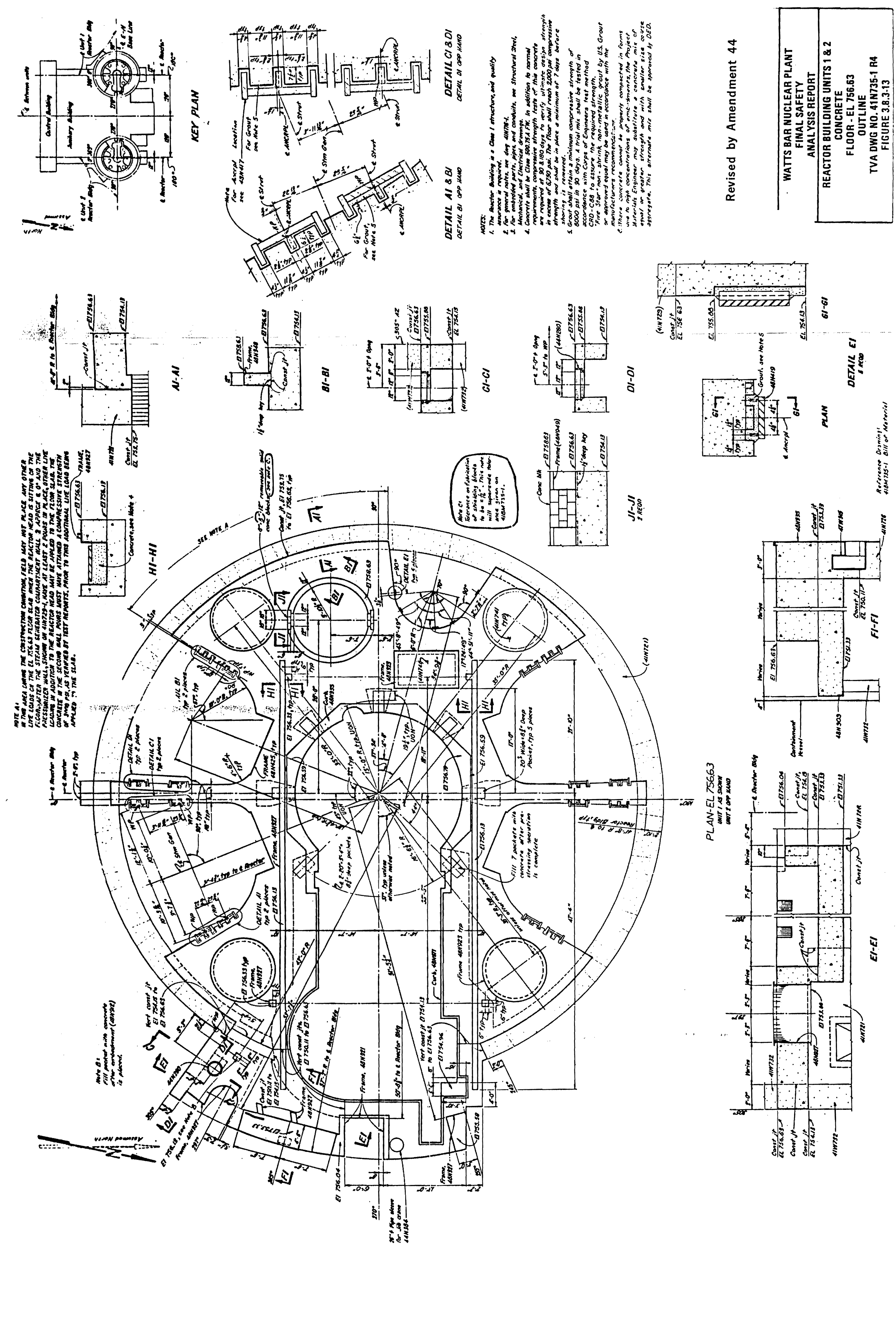

Figure 3.8.3-13 Reactor Building Units 1 & 2 Concrete Floor - El.756.63 Outline

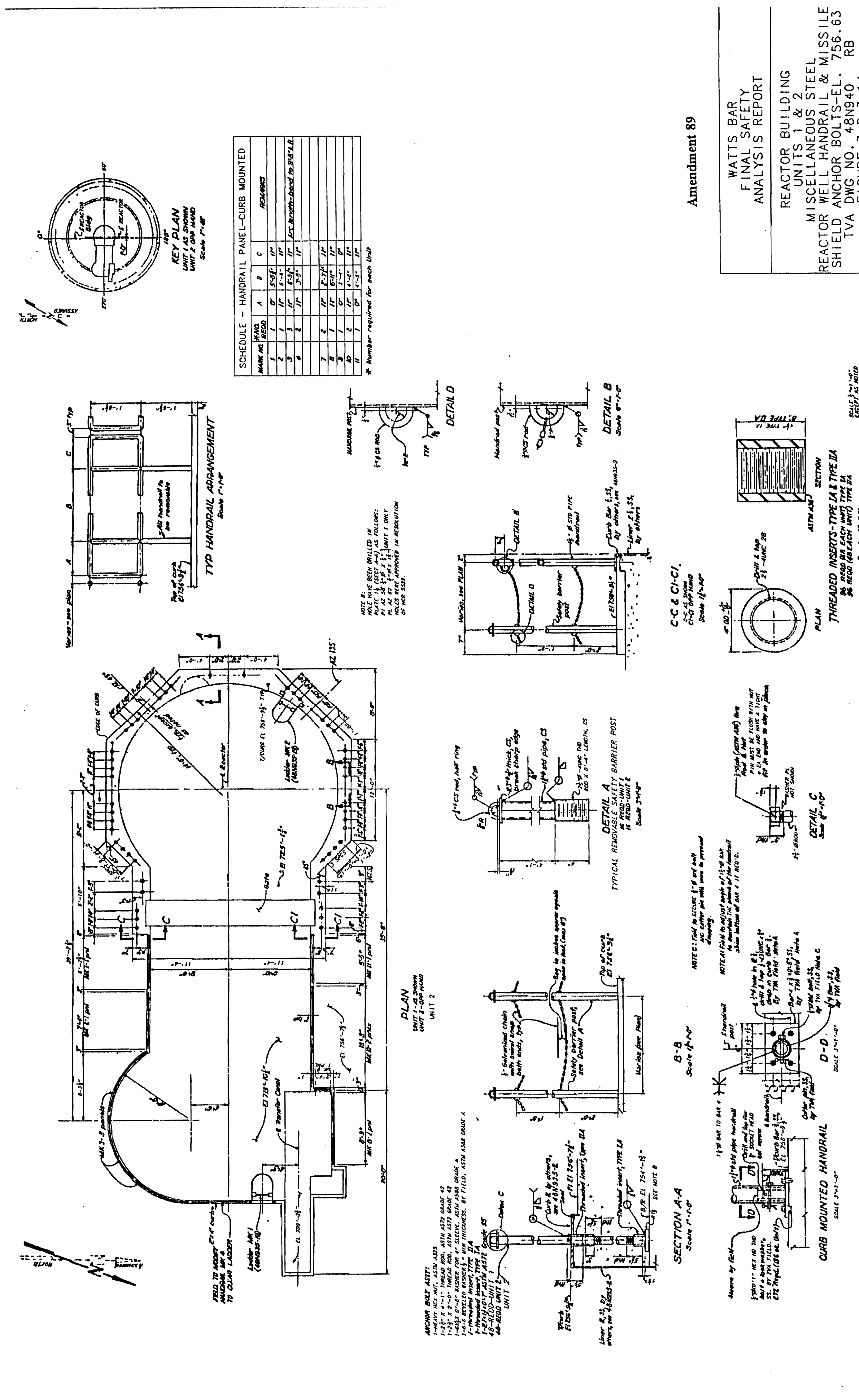

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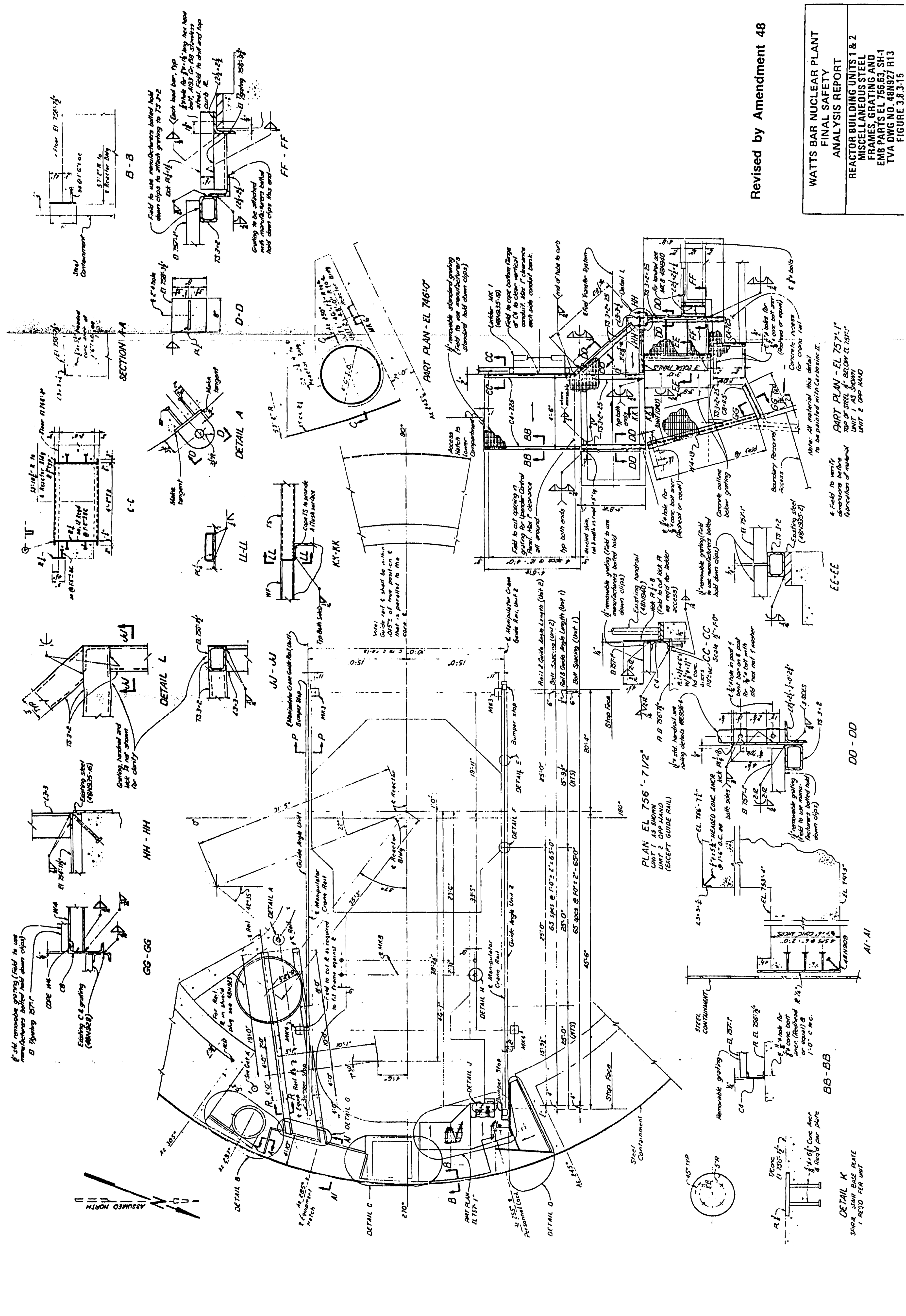
Figure 3.8.3-14 Reactor Building Units 1 & 2 Miscellaneous Steel Reactor Well Handrail & Missile Shield Anchor Bolts - EL 756.63



Revised by Amendment 48

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

REACTOR BUILDING UNITS 1 & 2
MISCELLANEOUS STEEL
FRAMES, GRATING AND
EMB PARTS EL 756.63, SH-1
TVA DWG NO. 48N927 R13
FIGURE 3.8.3-15

Figure 3.8.3-15 Reactor Building Units 1 & 2 Miscellaneous Steel Frames, Grating and EMB Parts EL. 756.63 - Sheet 1

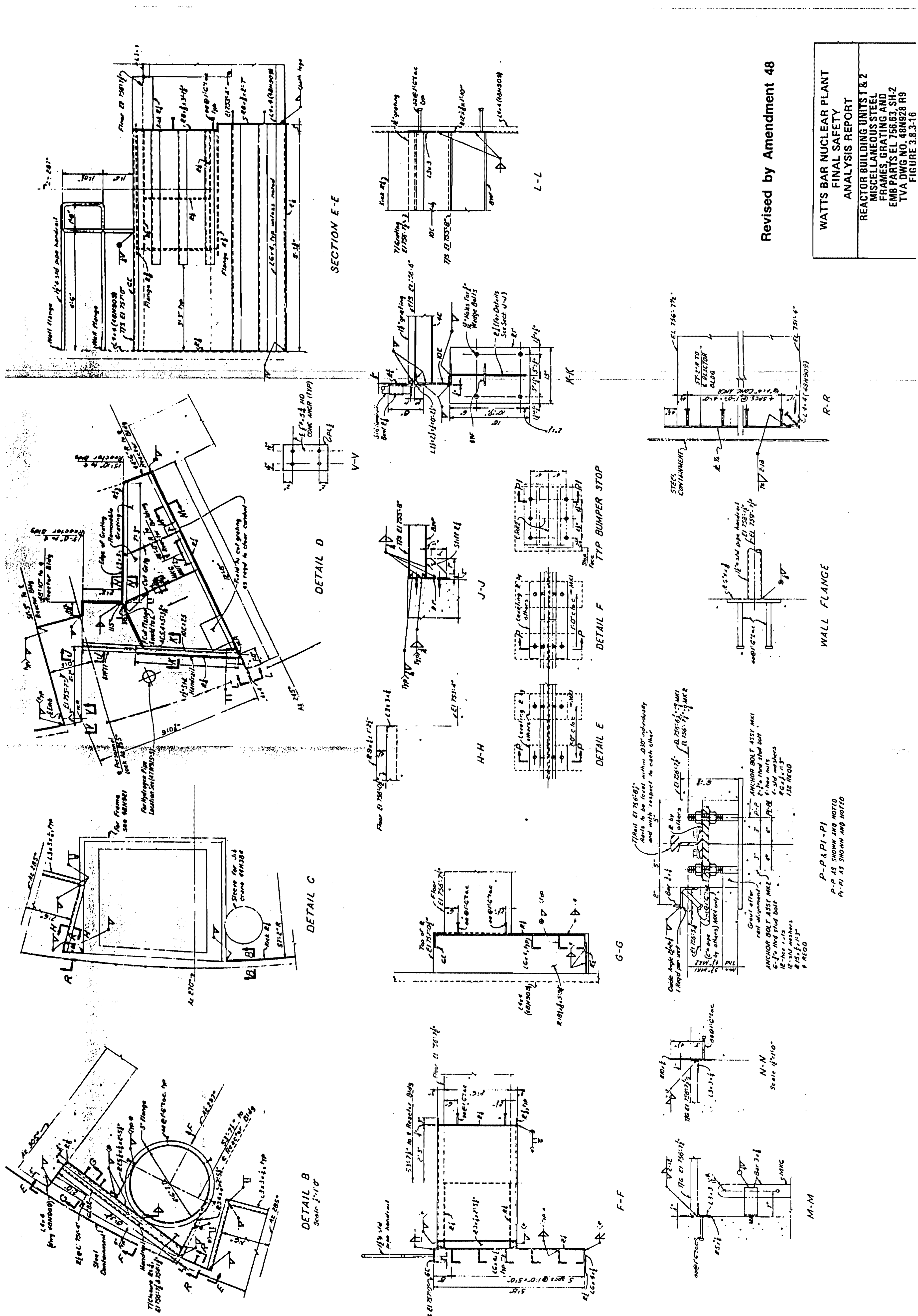


Figure 3.8.3-16 Reactor Building Units 1 & 2 Miscellaneous Steel Frames, Grating and EMB Parts EL. 756.63 - Sheet 2

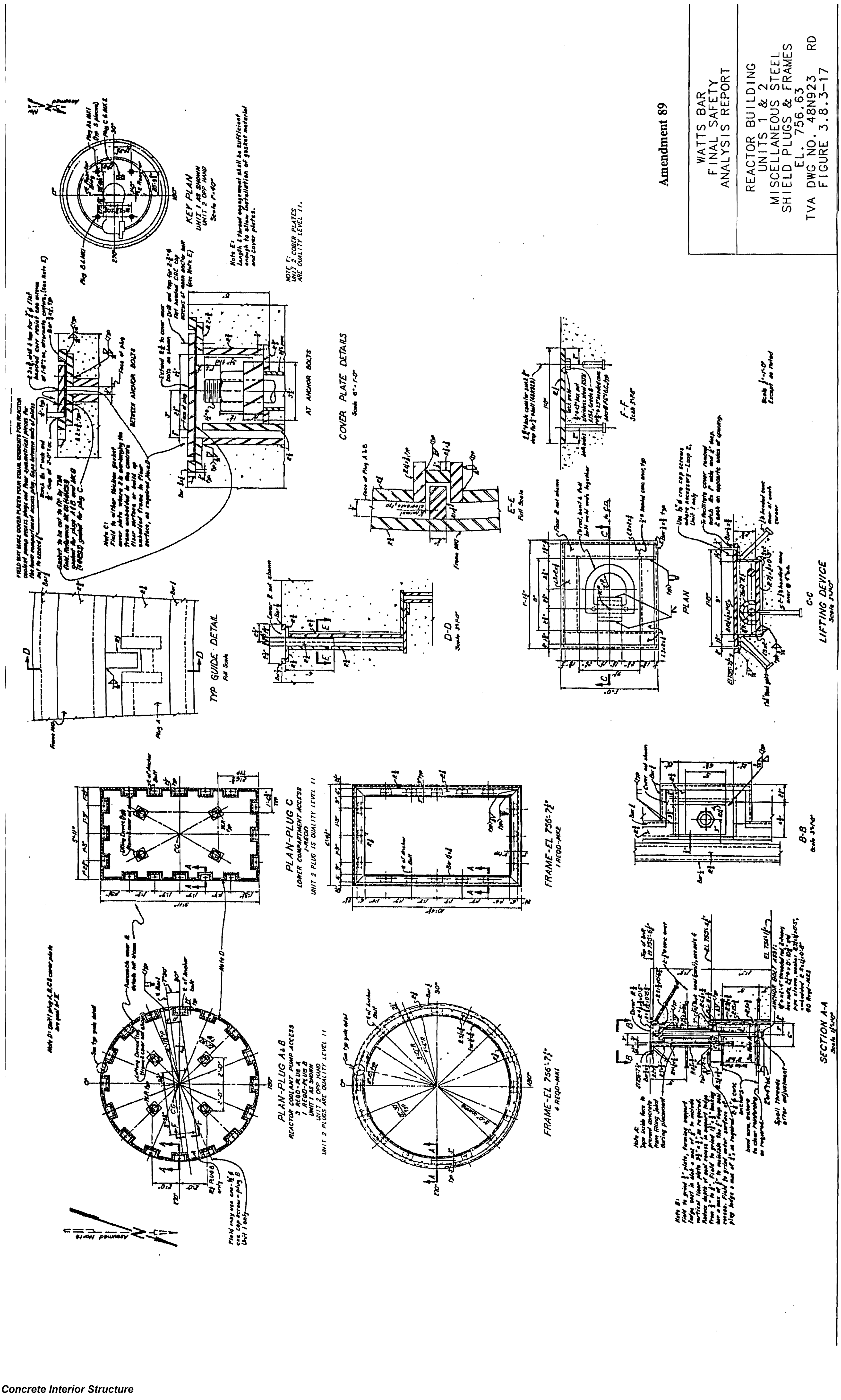


Figure 3.8.3-17 Reactor Building Units 1 & 2 Miscellaneous Steel Shield Plugs & Frames El. 756.63

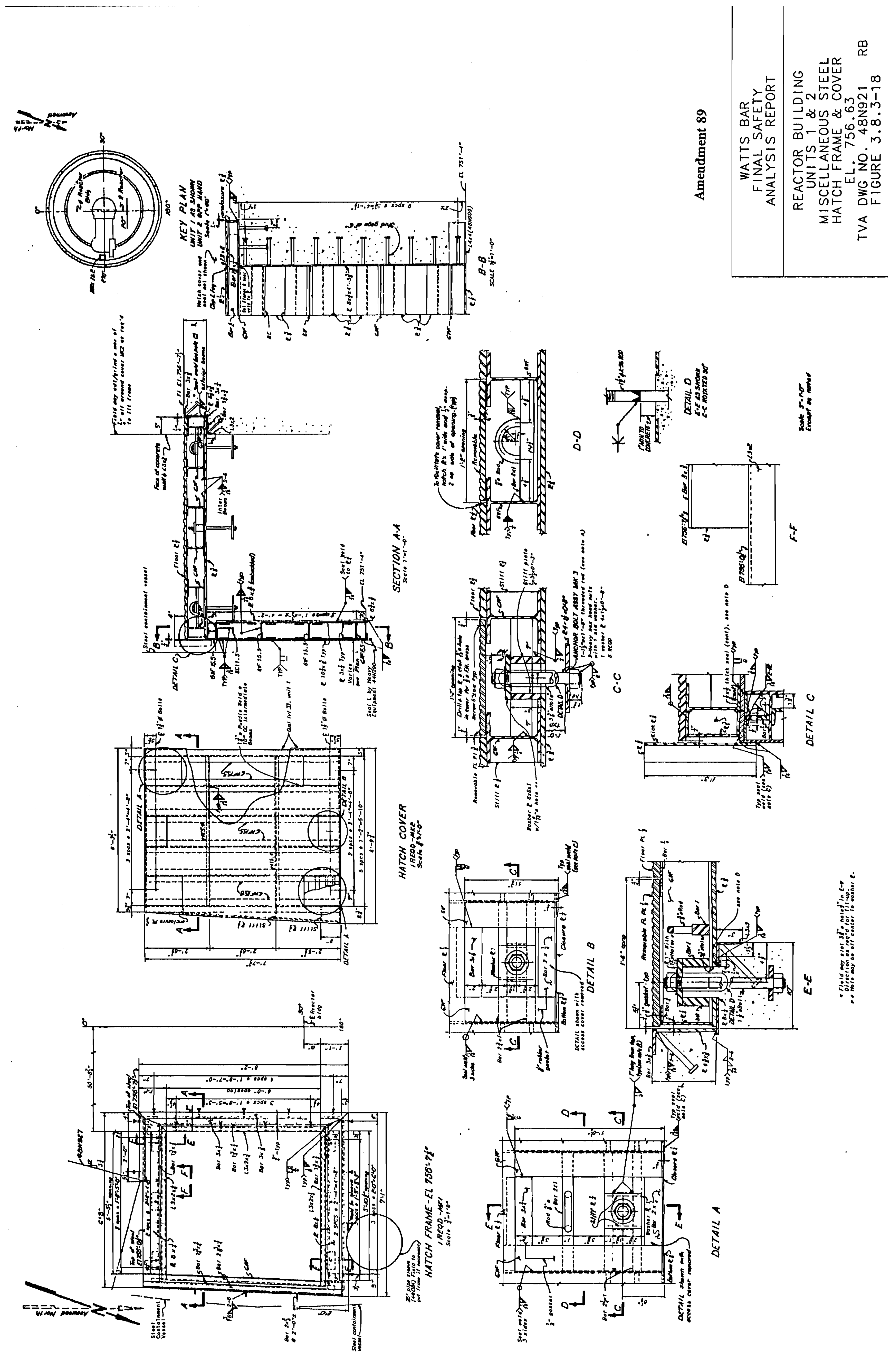
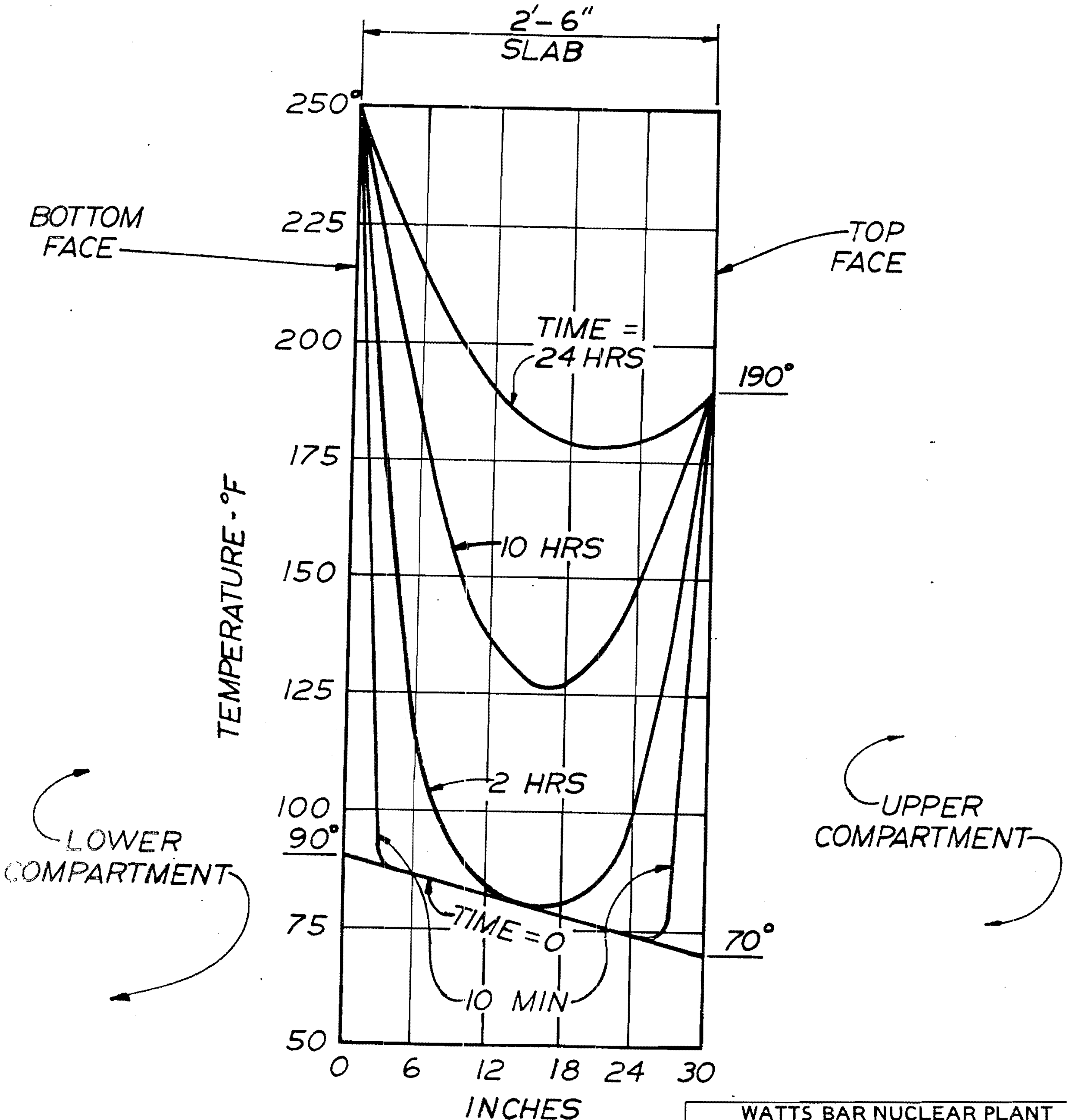


Figure 3.8.3-18 Reactor Building Units 1 & 2 Miscellaneous Steel Hatch Frame and Cover EL. 756.63



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TYPICAL DIVIDER FLOOR
LOCA TEMPERATURE GRADIENTS
Figure 3.8.3-19

Figure 3.8.3-19 Typical Divider Floor LOCA Temperature Gradients

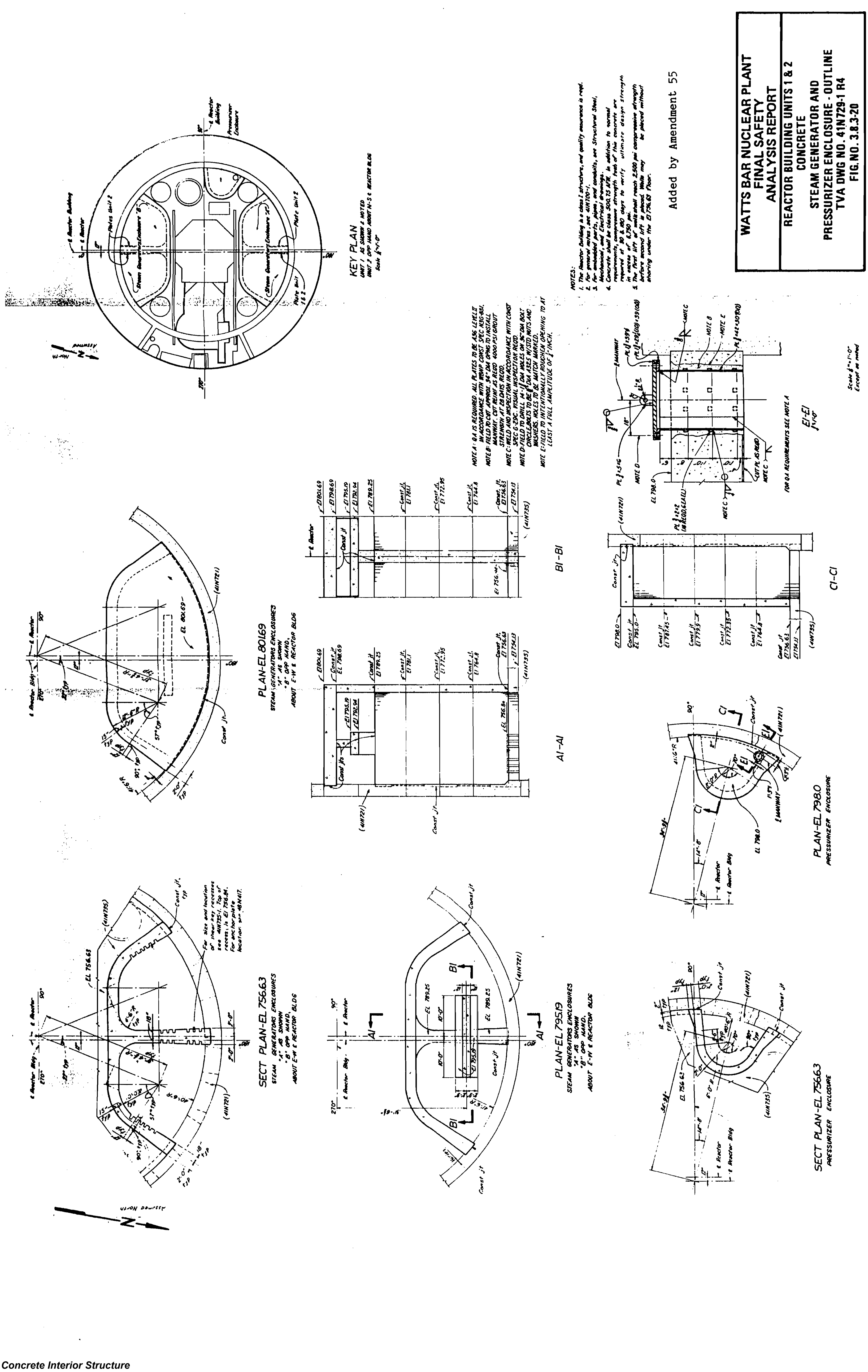

Figure 3.8.3-20 Reactor Building Units 1 & 2 Concrete Steam Generator and Pressurizer Enclosure - Outline

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3.8.4 Other Category I Structures

The Category I structures other than the primary containment, interior structures, and Shield Building are listed as follows:

(1) Auxiliary-Control Building and Associated Structures

(a) Control Bay Portion

(b) Auxiliary Building Portion

(c) Waste Packaging Structure

(d) Condensate Demineralizer Waste Evaporator Structure Portion

(e) Additional Equipment Building Portion

(2) Diesel Generator Building

(3) Category I Water Tanks and Pipe Tunnels

(4) Class 1E Electrical Systems Structures

(5) North Steam Valve Room

(6) Intake Pumping Station and Retaining Walls

(7) Miscellaneous ERCW Structures

(8) Additional Diesel Generator Building

(9) South Steam Valve Room

The structures are designed as described in Sections 3.8.4.1 through 3.8.4.7.

Evaluations and modifications of the reinforced concrete structures, are optionally done using ultimate strength design methods in accordance with the codes, load definitions and load combinations specified in Appendix 3.8E.

Evaluations and modifications of existing steel structures and miscellaneous steel and design of new steel members added after July 1979, are done in accordance with the codes, load definitions and load combinations specified in Appendix 3.8E.

3.8.4.1 Description of the Structures

3.8.4.1.1 Auxiliary-Control Building

This building and associate structures are multistory reinforced concrete structures which provide housing for the engineered safety feature systems, etc., which are necessary to the two reactor units. Certain floors in the control bay, the Condensate Demineralizer Waste Evaporator Structure, and the roof of the fuel handling bay are supported by structural steel framing. Refer to Figures 3.8.4-1 through 3.8.4-9 for the general layout and configuration of the structure.

Control Bay Portion

Structure

The control bay portion is a multistory reinforced concrete structure that is built integrally with the Auxiliary Building portion as shown in Figure 3.8.4-8. The structure is separated from the Turbine Building by a 2-inch expansion joint filled with fiberglass insulation which prevents interaction of the two buildings when subjected to seismic motion. The structure was built in one stage, in advance of the Auxiliary Building, and was used initially as a foundation for the construction gantry crane.

Structural steel framing in the control bay consists of four steel framed bays of 25.0 feet by 45.0 feet at elevation 729.0 and the entire floor at elevations 741.0 and 755.0, with the exception of the two exterior bays on both ends of the building.

At elevation 729.0 the two exterior bays on both ends of the building are pipe run areas and consequently the floor is 1-1/2-inch deep steel grating on steel beams. Reinforced concrete columns were used not only for structural requirements but also to reduce the beam size required.

The floor at elevation 741.0 is a 1-1/2-inch-deep steel grating on steel beams. The floor acts as both the structural support and the horizontal restraint for the cable tray support systems between elevation 729.0 and elevation 755.0. An extensive horizontal bracing system is used at elevation 741.0 to ensure that the floor maintains maximum horizontal restraint capacity with minimum member sizes.

The floor at elevation 755.0, except in the two exterior bays on both ends of the building, is an 8-inch concrete slab supported on steel beams with intermediate columns on the building longitudinal centerline. The main control boards and instrument racks are located on this floor and the cable trays are supported below it.

Control Room Shield Doors

Two doors located inside the Main Control Room at floor elevation 755.0 at doorways C-36 and C-54 provide radiation shielding for personnel inside the Main Control Room during a post-LOCA period. The doors normally remain open and are closed only in the event of a LOCA. The doors are manually operated from inside the control room and require no seals as they do not serve any pressure-confining function.

The two doors are identical except opposite hand and operate in opposite hand directions. Each door is a rectangular, structural steel frame with a skinplate on each side, thus forming a hollow box which is filled with lead shot to provide the required shielding. Each door is suspended from above by two monorail-type trolleys operating on a standard structural I beam. The trolley closest to the leading edge of each door is of the geared, hand chain-driven type for opening and closing the door.

Manually operated turnbuckle dogging linkages are provided at the top and bottom of each door for firmly securing the doors in either the open or closed positions.

Watertight Equipment Hatch Covers

There are two watertight equipment hatch covers provided at elevation 708.0. The covers will remain closed at all times during plant operation to ensure that essential safety equipment located below elevation 708.0 is protected from water resulting from a condenser circulating water system rupture in the Turbine Building. Each cover consists of an assembly of three welded structural steel sections held together by rows of steel screws with gaskets provided at each joint in the cover. In the closed or sealed position, the covers are secured by steel screws around the periphery. Sealing is accomplished by means of gaskets attached to the covers around the periphery which compress against an embedded structural steel frame when the covers are in place such that water leakage is prevented. In the event that the hatch covers are removed, supports are provided around the hatch opening for the installation of removable handrails.

Auxiliary Building Portion

Structure

The building is a multistory reinforced concrete structure that provides housing for the engineered safety feature systems, which are necessary to the two reactor units. The Auxiliary Building structure is attached to the Control Building and located between the Reactor Buildings as shown between column lines q and y, and A-1 and A-15, in Figure 3.8.4-8. In the final constructed state, the Control Building portion will act integrally with the Auxiliary Building portion. The Auxiliary Building is separated from the Reactor Buildings by a 1-inch expansion joint filled with fiberglass insulation that prevents interaction of the buildings when subjected to seismic motion. Seals are provided in the expansion joint to prevent the inleakage of either water or air since the Auxiliary Building, at times, serves as secondary containment. Below grade the seals, which consist of a polyvinyl chloride (PVC) material, are designed to withstand external water pressure, possible detrimental effects of the environment, the anticipated horizontal seismic movement of the buildings, and an assumed differential settlement of 1 inch between the buildings without loss of integrity.

The spent fuel pit and fuel transfer canal is housed within the Auxiliary Building. The massive reinforced concrete walls and slab are built integrally with the Auxiliary Building as illustrated by Figures 3.8.4-3 and 3.8.4-5. The spent fuel pit is equipped with two gates. The cask loading pit gate (UNID: 0-GATE-079-0006) is currently not utilized. The cask loading pit gate will be installed in its stored position only. The fuel transfer canal gate is installed or removed under balanced load. The spent fuel gates are illustrated by Figures 3.8.4-69 through 3.8.4-71.

Structural steel framing was used to support the Auxiliary Building roof over the area serviced by the main building crane because of the clear span requirements. This area is approximately 223.0 feet long and 80.0 feet wide. The roof is a reinforced concrete slab constructed on metal roof decking that is supported by steel purlins on welded steel trusses.

Railway Access Hatch Covers

Six hinged covers shown in Figures 3.8.4-10 and 3.8.4-11 combine to close the railroad access hatch opening in the floor of the Auxiliary Building at floor elevation 757.0 with the six covers in the raised position, a clear opening of approximately 16–feet, 6-inches by 68-feet, 3-inches is provided over the railroad tracks. Spent fuel casks, new fuel shipments, and major items of equipment entering or leaving the Auxiliary Building above the elevation 757.0 floor must go through this hatchway.

The hatch covers and their embedded frame provide a semi-airtight closure and operate in conjunction with the railroad access door to provide an airlock for the Auxiliary Building against a pressure differential of 1/4-inch of water.

An electrical interlock system is provided to interlock the operation of the access hatch covers with the railroad access door. Two limit switches, connected in series to provide redundancy, are provided with each hatch cover and arranged to trip when a hatch cover begins to open. The interlocking of these switches with switches on the door prevents the door from being opened when any hatch cover is open or partially open. In like manner, switches on the door prevent opening of any hatch cover when the door is open or partially open.

The hatch covers are required to maintain their integrity and Category I function only when closed. When closed, there is no load on the operating machinery and it has no function to perform. Therefore, the operating machinery is not considered as Category I.

Railroad Access Door

The railroad access door shown in Figures 3.8.4-12 through 3.8.4-15 for the Auxiliary Building provides closure for the access opening in the east wall at the railroad tracks which are at elevation 729.0. The door and its embedded frame provide a semi-airtight closure and operate in conjunction with the railroad access hatch covers to provide an airlock for the Auxiliary Building previously described. This door functions as an ABSCE airlock boundary - Door A112.

With the door fully opened, the clear opening in the wall is 16.0 feet wide and 20.0 feet high. All new or spent fuel shipments and major equipment entering or leaving the Auxiliary Building by truck or railroad passes through this door.

The door and door track are constructed of welded steel. The door, rectangular in cross section, is constructed of horizontal and vertical members with diagonal bracing as required for strength and rigidity. The exterior side of the door is covered with a steel skin plate. The embedded frame for the door is constructed of welded steel and is anchored to the concrete.

The door seals in the closed position with the side and top seals compressed against sealing surfaces on the embedded frame and the bottom seal compressed against an embedded sill plate. A sloped track guides the door rollers and positions the door so that the top and side seals contact the sealing surfaces only when the door is in or near the closed position.

An electric hoist unit opens and closes the door by lifting and lowering it vertically through a slot in the elevation 757.0 floor. The hoist unit is mounted on the inside wall above the door slot. The door passes through this slot, and extensions of the frame act as guides for the door in the raised position.

The area above the floor at elevation 757.0, occupied by the hoist and the door in its raised position, is enclosed with an airtight structural steel enclosure with gaskets provided on the access covers necessary for servicing the hoist unit and door.

The access door and its frame are required to maintain their integrity and Category I function only when closed. When closed there is no load on the hoist unit, and it has no function to perform. Also, the hoist unit has no function to perform relative to the airtightness of the steel enclosure at elevation 757.0. Therefore, the hoist unit is not considered as Category I.

Manways in RHR Sump Valve Room

Two 54-inch-diameter manways, shown in Figure 3.8.4-16, and located at elevation 698'-1" in the walls of the residual heat removal (RHR) sump valve room are provided for each reactor unit. The manways provide passageways through the walls of the sump valve room for workmen, tools, and equipment. The doors will normally be closed during plant operation, when reactor containment integrity is required, unless they are open to allow normal maintenance or monitoring activities. Although not required for containment integrity, the manways were designed to remain intact when the doors are open during an earthquake to prevent damage to other equipment in the vicinity of the manways.

Each manway consists of an embedded steel frame and a welded steel door. The door is secured in the closed position by bolts. The door is provided with slotted hinges to facilitate opening and closing and to allow for compression of the seals when the door is closed.

Pressure Confining Personnel Doors

This section covers the following pressure confining personnel access control doors located in the Auxiliary-Control Building. Door numbers listed for the doors are the designations used in Figures 3.8.4-17 through 3.8.4-20. The door details for specific heavy equipment type doors are shown in Figures 3.8.4-21 through 3.8.4-23. The door details for the remaining doors are shown on Reference [1].

(1) The doors for stairs 7 and 8 penthouses at Elevation 772.0, doors A184 and A191.

(2) The double doors to the personnel and equipment access rooms, Elevation 757.0 (one for each unit) doors A152* and A159*.

(3) The double doors at the ice condenser equipment room, Elevation 757.0, door A155.

(4) The double doors to the emergency gas treatment filter room, Elevation 757.0, door A158.

(5) The doors to the Reactor Building access room at Elevation 757.0 (one for each unit), doors A156 and A157.

(6) The doors for stairs 3 and 4 penthouses at Elevation 757.0, doors A154 and A173.

(7) The double doors to the elevator shaft at Elevation 757.0, door A153.

(8) The N-line control bay doors at Elevation 755.0 (two double doors with bidirectional pressure requirements), doors C36 and C54, and Elevation 729.0 (two double doors with bidirectional pressure requirements), doors C29 and C34.

(9) The N-line instrument rooms access door at Elevation 708.0 (single door with bidirectional pressure requirements), door C26.

(10) The double doors to the heating and ventilating spaces at Elevation 737.0 (one for each unit), doors A123* and A132*.

(11) The door separating the Additional Equipment Building and the airlock at Elevation 737.0 (one for each unit, bidirectional pressure requirements), doors A183*, A192*, A214*, and A215*.

(12) The door to the cask decontamination room, Elevation 729.0, door A115.

(13) The doors in the X-line wall of the cask loading area at Elevation 729.0 (one single door A113*, and one double door, A114*).

(14) Doors A161*, A162*, A64, A77, A216, A217, A94, A95, A96, A97, A98, A99, A164, A165, A166, and A167.

(15) The doors to the main steam and feedwater valve rooms at Elevation 729.0 (one for each unit), doors A101* and A105*.

(16) The double doors at main entrance from Service Building, Elevation 713.0, door A57*.

(17) Annulus access door A65* and door to the Reactor Building access rooms door A64 at Elevation 713.0.

(18) The airlock door to the radiochemical laboratory at Elevation 713.0, door A55*.

(19) The door in the C-3 line wall leading to the instrument room at Elevation 708.0, door C20 (water tight and pressure confining).

(20) The exterior double doors at the entrance to the Unit 1 Additional Equipment Building at Elevation 729.0, door A117.

(21) The Auxiliary Building door that separates stairwell no. 11 from the Unit 1 ventilation and purge air room on Elevation 737.0, door A125*.

(22) The Auxiliary Building door that separates stairwell no. 10 from the Unit 2 ventilation and purge air room on Elevation 737.0, door A130*.

(23) The Auxiliary Building door that separates shutdown board room A from the personnel & equipment access airlock (that leads to the refueling room floor) on Elevation 757.0, door A151.

(24) The Auxiliary Building door that separates shutdown board room B from the personnel & equipment access airlock (that leads to the refueling room floor) on Elevation 757.0, door A160.

(25) The Auxiliary Building doors that separate the mechanical equipment room from the HEPA filter plenum room at Elevation 772.0, doors A212* and A213*.

(26) The Auxiliary Building door that separates the upper portion of the refueling room from the airlock that leads to the Auxiliary Building roof at Elevation 786.0, door A206*.

(27) The Auxiliary Building exterior door that separates the Auxiliary Building roof from the airlock that leads to the upper portion of the refueling room at Elevation 786.0, door A207*.

(28) The Auxiliary Building door that separates the upper portion of the refueling room floor from the airlock that leads to the Auxiliary Building roof at Elevation 814.75, A208*.

(29) The Auxiliary Building exterior door that separates Auxiliary Building roof from the airlock that leads to the upper portion of the refueling room floor at Elevation 814.75, door A209*.

(30) The Control Building door that separates stairwell C1 from the corridor on the west side of the main control room at Elevation 755.0, door C37.

(31) The Control Building doors that separate the main control room from the Auxiliary Building at Elevation 755.0, doors C49 and C50.

(32) The Control Building door that separates stairwell C2 from the east side corridor at Elevation 755.0, door C-53.

(33) The Control Building door that separates stairwell C2 from the corridor leading to the Technical Support Center at Elevation 755.0, door C60.

(34) The exterior door at the entrance to the Condensate Demineralizer Waste Evaporator (CDWE) Building at Elevation 729.0, door DE1. This door functions as an ABSCE airlock boundary.

(35) The Auxiliary Building double doors at the entrance to the Service Building at Elevation 713.0, door A56*.

(36) The Auxiliary Building door that separates the Auxiliary Building from the airlock that leads to door A55 at Elevation 729.0, door A60*.

(37) The Auxiliary Building double doors that separate the heating and ventilating equipment rooms from the airlock that leads to door A123* at Elevation 737.0 door A-122*.

(38) The Auxiliary Building double doors that separate the heating and ventilating equipment rooms from the airlock that leads to door A132 at Elevation 737.0, door A133*.

(39) The interior CDWE Building doors that combine with door DE1 to establish the necessary airlock at the exterior entrance to the CDWE Building, doors DE4* and DE5*.

(40) The waste packaging room door that separates the waste packaging room from the railroad access room, double door A111*.

* These doors function as an ABSCE boundary for airlocks. See Table 3.8.4-7b.

The doors are hinged, manually operated type metal doors, complete with frames and closers. The frames are either welded to plates, bolted to the concrete walls, or welded to embedded plates. Both single and double doors are involved. Double doors consist of an active and inactive leaf, with the active leaf being used for normal traffic. Doors A65, A55, C20 and C26 have a single skin plate with horizontal stiffeners. Door A57 is a double skinned door with horizontal and vertical stiffeners. All other doors are the flush type. All doors except A55, A57, A65, C20, and C26 are secured for tornado, annulus pressure drop or flood by means of a normal latching mechanism. Doors A65 is secured by use of hand-operated dogs and doors A55, A57, C20, and C26 are secured by a dogging mechanism which is manually operated by a handwheel. All doors affected by tornadoes are secured during tornado watches and doors A65 is secured during flood warnings.

During normal operation, the doors provide personnel and equipment access. Doors A55, A56, A57, A60, A101,A105, A111, A112, A113, A114, A122, A123, A125, A130, A132, A133, A183, A192, A206, A207, A208, A209, A214, A215, DE1, DE4, and DE5 are also components of the building airlocks which serve to maintain a slight negative pressure in the Auxiliary and Reactor Buildings. These doors are equipped with electrical interlocks to assure that one of each pair of interlocked doors is always closed except when under administrative control. Doors A161, A162, DE1, DE4, and DE5 are also components of the building airlocks; however, they are not electrically interlocked.

Doors A55, A56, A57, A60, A101, A105, A111, A112, A113, A114, A122, A123, A125, A130, A132, A133, A151, A152, A159, A160, A161, A162, A183, A192, A206, A207, A208, A209, A212, A213, A214, A215, DE1, DE4, and DE5 are components of the Auxiliary Building Secondary Containment Enclosure (ABSCE) boundary. These doors will be subjected to the slight pressure differential (1/2" water gauge) needed to establish the ABSCE.

Doors C36, C37, C49, C50, C53, C54, and C60 are components of the Main Control Room Habitability Zone (MCRHZ) boundary. These doors will be subjected to the slight pressure differential (1/8" water gauge) needed to establish the MCRHZ.

Waste Packaging Structure

The waste packaging area is a one-story reinforced concrete structure supported on crushed stone backfill placed in four-inch layers and compacted to a minimum of 70% relative density and is located on the north end of the Auxiliary Building as shown in Figures 3.8.4-3 and 3.8.4-5. The roof of the structure slopes about 24° and consists of a series of precast beams topped by 4 inches of poured-in-place concrete. The structure is separated from the Auxiliary Building by a 2-inch expansion joint filled with fiberglass insulation which prevents interaction of the two buildings when subjected to seismic motion.

Condensate Demineralizer Waste Evaporator Structure

The Condensate Demineralizer Waste Evaporator Building portion is a two-story reinforced concrete structure that houses equipment necessary for processing condensate demineralizer wastes and for serving as a backup in processing floor drain wastes. The structure is supported on H-bearing piles and is located on the northeast side of the Auxiliary Building as shown in Figures 3.8.4-2 through 3.8.4-9. An access tunnel to the waste packaging area is separated from that structure by a 2-inch expansion joint filled with fiberglass material which prevents interaction of the buildings if subjected to seismic motion.

Additional Equipment Building Portion

The Additional Equipment Building portion consists of multi-story reinforced concrete structures, one for each unit, which accommodate accumulators for each unit and for the transfer of ice condenser equipment. The structures are located adjacent to the Reactor Buildings and near the north end of the main Auxiliary Building as shown in Figures 3.8.4-57 through 3.8.4-59. Each building is founded on sound rock and is separated from the Reactor Building by one inch of expansion joint material which prevents interaction of the building when subjected to seismic motion.

South Steam Valve Room

The South Steam Valve Room is an integral compartment of the Auxiliary Building portion of the Auxiliary-Control Building. The room is shown on Figures 3.8.4-3, 3.8.4–4, 3.8.4-8, and 3.8.4-49 through 3.8.4-49c. This compartment protects the isolation valves of the main steam lines, and other safety-related equipment, from the effects of tornados and earthquakes, as well as providing support for the main steam and feedwater pipes that exit from the Shield Building. The room is designed in accordance with the loads, load combinations, load factors, and allowable stresses given in Table 3.8.4-1.

Structural steel framing is used to support the roofing and roof decking of the valve room. The metal roof decking is designed to blow off to relieve pressure in the room.

Protection of the safety-related components within the room from horizontal tornado missiles is provided by the exterior walls of the Auxiliary-Control Building which includes one wall of the valve room. The other walls forming the room are interior to the building and are not subject to impact from horizontal tornado missiles.

Protection from vertical missiles is provided by the Reactor Building shield wall and by multiple levels of structural steel beams. The adjacent Reactor Building wall restricts the angles of possible missile entry. Since the roof of the steam valve room is more than 30 feet above plant grade, protection is required for the 1-inch diameter rod, missile A5 of Spectrum A (see Table 3.5-7 and Section 3.5.1.4). The multiple levels of structural steel beams partially screen safety-related components by further restricting possible missile entry angles. Small slender missiles such as the 1-inch diameter rod are known to be aerodynamically unstable and, therefore, tumble in flight. It is highly unlikely that a tumbling missile could strike safety-related equipment due to the limited pathways through the multiple levels of steel support structures. Therefore, adequate protection from vertical missiles is provided.

3.8.4.1.2 Diesel Generator Building

The building is a two-story rectangular reinforced concrete box-type structure that houses the diesel generators and associated auxiliary equipment. Interior walls of reinforced concrete separate the diesel generators into four compartments. The diesel fuel storage tanks are embedded in the base slab. The structure is supported on crushed stone backfill placed in four-inch layers and compacted to a minimum of 70% relative density. For general layout and configuration of the structure see Figures 3.8.4-24 through 3.8.4-29.

Diesel Generator Building Doors and Bulkheads

The four doors shown in Figures 3.8.4-30 through 3.8.4-32 at elevation 742.0 in the north wall of the Diesel Generator Building along with removable bulkheads above the doors provide closures for the 11' - 10" high by 8' - 8" wide access openings to the diesel generator units. They provide for passage of large tools and repair parts for the diesel generators. The doors are normally closed and latched. The bulkheads are bolted in position and are removed only for major repair of the diesel generators. The doors and bulkheads are covered on the outside of the Diesel Generator Building by precast concrete bulkheads as shown in Figures 3.8.4-27 and 3.8.4-33. Together they protect the generators from damage by tornadoes, missiles, wind, snow, ice, and rain and form part of the security to prevent entry into the Diesel Generator Building by unauthorized persons. See Section 3.8.4.5.5.

Each bulkhead above the door is a structural steel frame 4' - 5½" high by 9'- 5" wide. It is covered on both sides with a steel skin plate and provided with a crushable strip on the inner side along the top and sides. Turnbuckles support the bulkheads vertically, and they are held horizontally by bolted clamps at the sides and top.

Each door is 7' - 9½" high and consists of two leaves that are manually operated and hinged at the outer sides to an embedded steel frame. The two leaves bear against steel bars at the outer sides, against an embedded angle at the bottom, against each other at the center, and against a steel angle at the top. The bars are welded to the embedded frame and the angle to the bulkhead above the door.

Each door leaf is a structural steel frame covered on both sides with a steel skin plate and provided with a crushable strip around its periphery where it bears against lateral support. Both leaves are provided with latches that are operated from the inside only.

The steel doors and bulkheads were provided in the original design of the Diesel Generator Building to protect the diesel generators from missiles B1, B2, and B3 of missile spectrum B in Table 3.5-8. In a review of the tornado protection criteria by the NRC in 1975 a determination was made that the level of protection provided by the doors and bulkheads should be upgraded to resist three additional missiles (B4, B5, and B6). The existing steel doors and bulkheads were found to be inadequate for the additional missiles. Therefore, precast concrete bulkheads were placed in front of the door openings to provide the additional missile protection. The precast concrete bulkheads consist of several individual sections stacked into place and bolted in position to the concrete walls. The precast concrete bulkheads are required to be in place when the diesel generators are operable.

The precast concrete bulkheads are 14 inches thick which is adequate to prevent penetration from missiles B4, B5, and B6. The 14-inch thickness is not sufficient to prevent some scabbing. However, the steel doors prevent the scabbed particles from entering the generator compartments. In the event the steel doors are open the scabbed particles will not reach the diesel generators due to the separation of the generators and doors. This protective scheme of preventing penetration but not scabbing is necessary due to the desire to keep the weight of the precast sections low to facilitate removal by field personnel.

3.8.4.1.3 Category I Water Tanks and Pipe Tunnels

There is one refueling water storage tank for each unit at Watts Bar Nuclear Plant. (The functional requirements for these tanks are discussed in Chapter 6.) Pipes extending from these tanks to the Auxiliary Building are housed in reinforced concrete tunnels which vary in width and height.

Refueling Water Storage Tanks (RWST)

As noted in Tables 3.2-1 and 3.2-2a, the RWST foundation is classified as Category I. The RWST is a Seismic Category I structure. A storage basin is provided around the tank to retain sufficient borated water in the event the tank is ruptured by a tornado missile or other initiating event. Details of the storage basin and the technical basis for it are discussed below. The RWST has a minimum capacity of 370,000 gallons. The tank is a cylindrical vessel whose longitudinal axis is oriented in the vertical direction.

The end of the cylinder which forms the base or bottom of the anks is completely enclosed with a 5/16-inch thick flat plate. The base of the tanks sits on a support structure consisting of a concrete slab 57'-0" in diameter and 3'-6" thick to which the tanks are attached at 48 anchor points. The perimeter ring plate is grout supported by the slab foundation. The interior is filled with approximately 6 inches of sand on top of the concrete slab, which supports the entire surface of the tank base or bottom plate. The top of the cylindrical section of the tank is sealed at the sidewall roof intersection using conical-shaped roofs whose apexes coincide with the tank's longitudinal axis.

A barrier wall is located around the RWST to protect the bottom three feet of the tank. This provides storage for borated water after a postulated rupture of the tank. A steamline break in one of the lines outside of the Reactor Building in close proximity to the RWST is the most demanding event that could require suction from the RWST and also credibly be associated with a rupture of the RWST. For this event a maximum of 20,000 gallons of borated water was initially analyzed by the NSSS vendor to provide the negative reactivity to keep the reactor subcritical. A lesser amount has been analyzed to maintain departure from nucleate boiling ratio (DNBR). The barrier wall, however, is designed to retain a volume in excess of this amount while supplying adequate net positive suction head to all ESF pumps. Figure 3.8.4-36a shows the distance (greater than 3 feet) between the top of the wall and the suction intake elevation. The tank is equipped with an atmospheric vent located at the peak or cone apex of the roofs. The vent is designed to pass a volume flow rate of air that is at least equal to the maximum withdrawal rate from the tank. Necessary precautions have been taken in the design of the vent to assure birds, animals, and/or other foreign objects, including, rain cannot enter the tanks. Tank physical dimensions and other parameters are given in Section 9.2.7. The foundations are shown in Figures 3.8.4-35 through 3.8.4-36C. Load combinations for the foundations are shown in Table 3.8.4–20.

Pipe Tunnels

The pipe tunnels housing the piping extending from the primary and refueling water tanks to the Auxiliary Building are concrete box-type structures that vary in width and height. Protection against flooding of the Auxiliary Building in case of a tank or pipe rupture is provided by walls that separate the tanks from the main tunnel. The layout and configuration of the tunnels are shown in Figures 3.8.4-35 and 3.8.4-36.

3.8.4.1.4 Class 1E Electrical System Manholes and Duct Runs

The manholes and duct runs shown in Figures 3.8.4-37 through 3.8.4-46 house the electrical cables that must remain in operation when flood levels rise above the plant grade and emergency power is required for safe shutdown of the plant. The manholes and duct runs lie in soil overburden that varies in depth from 30 to 35 feet.

The Category 1E manholes are rectangular box-type structures of reinforced concrete built below plant grade with an access shaft projecting above the surrounding soil. Category 1 E manholes are equipped with watertight covers and sump pumps. A concrete cover is provided for protection from vertical missiles.

The duct runs connecting the manholes are continuous reinforced concrete beams with embedded electrical conduits. Duct runs are buried with a minimum of 18 inches of soil cover above them, except near the intake pumping station where they are exposed. Duct runs enter the manholes through sealed openings in manhole walls. A minimum of 6-1/2 inches of reinforced concrete is provided above the embedded conduits for protection from vertical missiles.

3.8.4.1.5 North Steam Valve Room

The structure, shown in Figures 3.8.4-47 through 3.8.4-49 is designed to protect the isolation valves of the main steam lines from the effects of tornadoes and earthquakes, as well as provide support for the valves and main steam pipes and feed water pipes that exit from the Shield Building. The structure consists principally of several high reinforced concrete walls anchored into a 7-foot-thick base slab that rests on a grillage of reinforced concrete foundation walls supported to rock. A 2-inch expansion joint separates the valve room from the Shield Building.

Structural steel framing is used to support roof decking of the valve room. The metal roof decking is designed to blow off when the internal pressure at the roof reaches 72 pounds per square foot.

Protection of the components from horizontal tornado missiles is provided by the walls of the steam valve room. Protection from vertical tornado missiles is provided by a 24-inch-thick concrete awning which covers approximately one-third of the roof, by the Reactor Building shield wall, and by multiple levels of structural beams. The concrete awning and the adjacent Reactor Building wall, which extends 89 feet above the decking, restrict the angles of possible missile entry. Since the roof of the steam valve room is more than 30 feet above plant grade, protection is required for small, slender missiles, such as the 1-inch-diameter roof missile A5 of Spectrum A (see Table 3.5-7 and Section 3.5.1.4). (See also Figures 3.8.4-49B through 3.8.4-49C).

The main steam safety and relief valves are located about 21 and 17 feet, respectively, below the decking. Four 30-inch-wide flange beams and numerous 8-inch steel channels serve to partially screen the safety and relief valves from tornado missiles by further restricting possible entry angles. The largest pathway through the wide flange beams and the channels is approximately 1.5 feet by 2.5 feet (two such pathways) in plan area. Small slender missiles such as the 1-inch-diameter rod are known to be aerodynamically unstable in flight and, therefore, tumble during flight. It is highly unlikely that a tumbling missile could follow the pathways discussed above without being deflected. Therefore, the main steam safety and relief valves are adequately protected from vertical tornado missiles. (See also Figures 3.8.4-49B through 3.8.4-49C).

The main steam, main feedwater, and feedwater bypass isolation valves are located below the safety and relief valves and are further protected from missile damage by five levels of wide flange beams (33-inch to 8-inch size) provided for pipe break restraint and support functions. There are no practical pathways by which tornado missiles could reach these valves. (See Figures 3.8.4-49B through 3.8.4-49C).

3.8.4.1.6 Intake Pumping Station and Retaining Walls

Pumping Station

The intake pumping station is a cellular box-type, reinforced concrete, waterfront structure founded on bedrock and partially backfilled on three sides. On the land side, retaining walls hold back the fill to elevation 710.0. Permanent openings are provided in the reservoir side of the pumping station to allow flooding of any unwatered pump wells when the reservoir level exceeds elevation 690.0. The essential raw cooling water (ERCW) pumps, fire protection pumps, and screen wash pumps are located on the upper deck at elevation 741.0 above the maximum possible flood and is covered by a roof. This deck is completely enclosed by a concrete wall 13 feet high. A wall also supports the structural steel grillage system, shown in Figure 3.8.4-68, which provides tornado missile protection to the equipment below. The raw cooling water pumps are located on the deck at elevation 728.0, which is below the maximum probable flood, but are not required for maintenance of plant safety. The mechanical and electrical equipment are located on the floors at elevation 722.0 and 711.0, respectively. A permanent pedestal crane is mounted above the upper deck at elevation 754.0 for handling of equipment. The structural outline is shown in Figures 3.8.4-50 and 3.8.4-51.

Traveling Water Screens

As shown in Figure 3.8.4-52, the screens are of the single or through flow, automatic cleaning type with a nominal basket width of 4.0 feet.

The capacity of each screen, with a head loss of 6 inches for a clean screen and minimum water depth, is approximately 25,000 gallons per minute at a water velocity of 2.0 feet per second. Basket travel speed is about 10 feet per minute. Removal of trash and refuse from the basket screens is by water sprays located in the head frame.

The drive motor for each screen is sized to start the screen with water at elevation 737.5 and a head loss of 2-feet, 6-inches. All drive components are rated for continuous duty and are suitable for outdoor service.

There are two watertight doors provided on Elevation 722.0 identified as W10A and W10B. These doors prevent water from the room containing one train of the ERCW System from entering the room containing the other train.

Timers provided in the control circuits for the screens function to operate the screens for 18 minutes every 60 hours to prevent "freezing" of the machinery parts from nonuse. This provides assurance that the screens are in an operable condition at all times.

The heads of the screens, including drive components, are located above the maximum possible flood level. The screens are designed to operate during any flood, including a maximum possible flood, with water to elevation 738.8 and a 5’-0” head loss.

The four screens are identical with two screens provided for each of the two supply trains at the intake station. Each of the two screens on each supply train has sufficient capacity to screen the total water required for one train. The capacity of one supply train is sufficient to supply all water required for the ERCW during a LOCA.

Starting of the screens by pressure switches on the spray water assures that adequate spray water for removal of trash is available when the screens are started. This greatly reduces the possibility of carrying trash over the screens and into the screened water.

Concrete Retaining Wall

The earthfill is hold back by two concrete retaining walls from the pumping station to a point 32 feet from the pumping station. The concrete walls are keyed into rock and are separated from the pumping station by expansion joints. For outline of walls, see Figure 3.8.4-53.

Sheet Pile Retaining Walls

The sheet pile walls are parallel and extend from each end of the back of the pumping station toward the main plant. These parallel walls contain earthfill to elevation 710.0 and project above the sloping grade a maximum of 29 feet at the pumping station. For layout of walls, see Figures 3.8.4-54 and 3.8.4-55.

3.8.4.1.7 Miscellaneous Essential Raw Cooling Water (ERCW) Structures

Slabs and Beams Supporting ERCW Pipes

At the Intake Pumping Station, the ERCW pipes are supported on a reinforced concrete slab. The slab is approximately 8 feet below grade and 50 feet above bedrock. The slab is supported by a bracket on the pumping station wall, bearing piles, and undisturbed earth. Structural separation from the pumping station is provided by 1/2-inch of expansion joint material. The slab is shown in Figure 3.8.4-56.

The ERCW pipes at the Diesel Generator Building are encased in concrete beams for support. The pipes are separated from the beams by insulation and the beams is separated from the Diesel Generator Building by expansion joint material. The beams are supported by brackets on the Diesel Generator Building and by Class A backfill. The beams are shown in Figure 3.8.4-56b.

Discharge Overflow Structure

The discharge overflow structure is a reinforced concrete box-type structure supported on granular fill material placed over basal gravel. The function of the discharge overflow structure is to provide for the normal flow rate discharge of the ERCW system without unacceptable back pressure if the downstream pipes are blocked and to permit flow to the holding pond under normal conditions. The structural outline is shown in Figure 3.8.4-46a.

Standpipe Structures

The two standpipe structures are mass reinforced concrete structures placed on firm granular material. The structures have backfill on four sides for the first 8 feet of height and extend 17 feet above grade. The function of these structures is to protect the standpipes from tornado-generated missiles. The structures are shown in Figure 3.8.4-56a.

Valve Covers

These structures consist of reinforced concrete slabs covering the valves in the ERCW pipes. The slabs are located at grade above the pipes and are supported by either the missile protection slab and/or backfill. The slabs have small openings with precast concrete covers above each valve stem. The openings in the missile protecting valve covers provide immediate access to the valves in an emergency. The structures are shown in Figure 3.8.4-56c.

3.8.4.1.8 Additional Diesel Generator Building

The Additional Diesel Generator Building is located 349.25 feet north of the centerline of the Reactor Buildings and 54.5 feet west of the centerline of the Unit 1 Reactor Building. It is a two-story rectangular, reinforced concrete, box-type structure which houses the additional diesel generator unit and its auxiliary equipment. The building is 96 feet long by 53 feet wide and is supported entirely on end bearing structural steel H-Piles as shown in Figure 3.8.4-72. The base slab is 12 feet thick with the finished floor at elevation 742.0. The diesel fuel storage tanks are embedded in the base slab. For general layout and configuration of the building see Figures 3.8.4-73 through 3.8.4-80.

Additional Diesel Generator Building Doors and Bulkheads

The two large door openings, shown in Figures 3.8.4-74 and 3.8.4-75, in the north and east exterior walls of the building at elevation 742.0, provide for passage of large tools and repair parts for the additional diesel generator unit and its auxiliary equipment. Removable missile barriers of precast, stackable concrete sections are installed and bolted into position in front of these doorways to protect safety-related equipment from tornado wind and missiles. These missile barriers also form part of the security system by preventing unauthorized entry into the building through these doors. Due to the presence of the precast concrete missile barriers in front of the doorways, the equipment doors do not need to function as missile barriers and therefore standard double doors are used. The precast concrete missile barriers will be removed only for major repair of the diesel generator.

3.8.4.2 Applicable Codes, Standards, and Specifications

Unless otherwise indicated in the FSAR, the design and construction of the Category I structures other than the primary containment and interior structures are based upon the appropriate sections of the following codes, standards, and specifications. Modifications to these codes, standards, and specifications are made where necessary to meet the specific requirements of the structures. These modifications are noted in Sections 3.8.3.2, 3.8.4.3, 3.8.4.4, and 3.8.4.6. Where date of edition, copyright, or addendum is specified, earlier versions of the listed documents were not used. In some instances, later revisions of the listed documents were used where design safety was not compromised.

3.8.4.2.1 List of Documents

(1) American Concrete Institute (ACI)

ACI 214-77	Recommended Practice for Evaluation of Strength Results of Concrete
ACI 318-63	Building Code Requirements for Reinforced Concrete. (See Section 3.8.4.2.2 for basis for use of this section.)
ACI 318-71	Building Code Requirements for Reinforced Concrete
ACI 347-68	Recommended Practice for Concrete Formwork
ACI 305-72	Recommended Practice for Hot Weather Concreting
ACI 211.1-70	Recommended Practice for Selecting Proportions for Normal Weight Concrete
ACI 304-73	Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete
ACI 349-76	Code Requirements for Nuclear Safety Related Concrete Structures, Appendix C only
ACI 531-79	Building Code Requirements for Concrete Masonry Structures

(2) American Institute of Steel Construction (AISC), "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," adopted February 12, 1969, as amended through June 12, 1974, except welded construction is in accordance with Item 5 below.

(3) Steel Structures Painting Council, Surface Preparation Specification No. 2, 'Hand Tool Cleaning.'

(4) American Society for Testing and Materials (ASTM), 1971 Annual Book of ASTM Annual Standards.

(5) American Welding Society (AWS)

'Structural Welding Code,' AWS D1.1-72, with revisions 1-73 and 2-74 except later editions may be used for prequalified joint details, base materials, and qualification of welding procedures and welders.

Visual inspection of structural welds will meet the minimum requirements of Nuclear Construction Issues Group documents NCIG-01 and NCIG-02 as specified on the design drawings or other engineering design output. See Item 18 below.

'Recommended Practice for Welding Reinforcing Steel, Metal Inserts, and Connections in Reinforced Concrete Connections,' AWS D12.1-61.

(6) American Gear Manufacturers Association. Standards for Helical and Herringbone Gears.

(7) Uniform Building Code, International Conference of Building Officials, Los Angeles, 1970 Edition.

(8) Southern Standard Building Code, 1969 Edition, 1971 Rev.

(9) 'Nuclear Reactors and Earthquakes,' USAEC Report TID-7024, August 1963.

(10) American Society of Civil Engineering Transactions,

Vol. 126, Part II, Paper No. 3269, 'Wind Forces on Structures,' 1961.

(11) Code of Federal Regulations, Title 29, Chapter XVII, "Occupational Safety and Health Administration, Dept. of Labor", Part 1910, 'Occupational Safety and Health Standards.'

(12) Regulatory Guides (RG)

RG 1.10	Mechanical (Cadweld) Splices in Reinforcing Bars of Category I Concrete Structures
RG 1.13	Fuel Storage Facility Design
RG 1.15	Testing of Reinforcing Bars of Category I Concrete Structures
RG 1.31	Control of Stainless Steel Welding
RG 1.55	Concrete Placement in Category I Structures

(13) Section deleted by Amendment 89

(14) TVA Reports

TVA-TR-1 Topical Report TVA-TR1, Protection Against Pipe Whip Resulting From Piping Ruptures, 1973.

TVA-TR-78-4 Design of Structures to Resist Missile Impact, 1978.

TVA-CEB-86-12 Study of Long Term Concrete Strength at Sequoyah and Watts Bar Nuclear Plants

TVA-CEB-86-19-C Concrete Quality Evaluation

(15) National Electrical Manufacturers Association, Motor and Generator Standards MG-1, 1970 Edition.

(16) Structural Engineers Association of California, "Recommended Lateral Force Requirements and Commentary," 1968 Edition.

(17) National Fire Protection Association(NFPA) 30.

(18) Nuclear Construction Issues Group (NCIG)

NCIG-01, Revision 2 - Visual Weld Acceptance Criteria (VWAC) for Structural Welding at Nuclear Power Plants

NCIG-02, Revision 0 - Sampling Plan for Visual Reinspection of Welds

The referenced NCIG documents may be used after June 26, 1985, for weldments that were designed and fabricated to the requirements of AISC/AWS.

NCIG-02, Revision 0, was used as the original basis for the Department of Energy (DOE) Weld Evaluation Project (WEP) EG&G Idaho, Incorporated, statistical assessment of TVA performed welding at WBNP. Any further sampling reinspections of structural welds subsequent to issuance of NCIG–02, Revision 2, are performed in accordance with NCIG-02, Revision 2 requirements.

The applicability of the NCIG documents is specified in controlled design output documents such as drawings and construction specifications. Inspectors performing visual weld examination to the criteria of NCIG-01 are trained in the subject criteria.

3.8.4.2.2 Basis for Use of the 1963 Edition of ACI 318

The reason for using the 1963 edition of the ACI 318 Code was that much of the Watts Bar Plant was a duplicate of the Sequoyah Plant, for which structures were designed using the 1963 edition. On that basis, design computations for the Sequoyah Plant were the initial design computations for the Watts Bar Plant.

In some instances, structures could not be duplicated and new design computations were prepared for these structures with the designs in accordance with the ACI 318-71 Code. Within duplicate structures, where loading changes required investigation of the Sequoyah design for an element of the structure, and the result was a change in member size or reinforcement requirement, the redesign for the member was in accordance with the ACI 318-71 Code.

The duplicate structures are the Auxiliary-Control Building and the Diesel Generator Building.

The differences between the two code editions for working stress design were examined and the conclusion was that none of these differences significantly affect the safety of the plant.

The following are comparisons of the important parts of the code which affect safety

Flexure	**ACI 318-63 W. S. Design**	**ACI 318-71 Alt. Sect 8.10**
Concrete Stress	$0.45\ f'_C$	$0.45\ f'_C$
Steel Stress	24,000 psi	24,000 psi
Tied Columns		
Maximum P	$0.212\ f'_C Ag$	$0.268\ f'_C Ag$
	$+20{,}400\ As$	$+18{,}900\ As$
Balanced P	$\sim 0.15\ f'_C Ag$	$\sim 0.16\ f'_C Ag$
Concrete Shear Stress		
Beam V_C	$1.1\sqrt{f'_c}$ 1	$1.1\sqrt{f'_c}$ 2
Slab V_C	$2\sqrt{f'_c}$ 3	$2\sqrt{f'_c}$ 4
Bearing Stress		
Concrete	0.25 to $0.375\ f'_C$	0.3 to $0.6\ f'_C$

One of the apparent major differences between the two codes is the method by which rebar contact splice lengths are calculated. The 1971 code does require longer splice lengths where bars are spaced closer than 6 inches, but it is TVA practice to specify bar spacing of 6 inches or greater for bars which must be lapped by a contact splice. Listed below is a comparison considered to be typical in TVA practice. The results of this comparison show that the use of the 318-63 Code does not significantly affect the safety of the plant.

Contact Splice Lengths

f'c = 3,000 psi

fy = 60,000

No. 11 bar in tension, top bar

Bar spacing, 6 inches to 17 inches

	318-63 **W. S. Design**	**318-71** **Alt. Sect.8.10**
A_s Provided Equals That <u>Required by Computation</u>		
Less than 1/2 bars spliced at a given section	102 in.	99 in.
More than 1/2 bars spliced at a given section	Not specified TVA practice 1.2x102=122 in.	129 in.
A_s Provided Equals Twice <u>That Required by Computation</u>		
Less than 3/4 bars spliced at a given section	102 in.	76 in.
More than 3/4 bars spliced at a given section	102 in.	99 in.

3.8.4.3 Loads and Loading Combinations

3.8.4.3.1 Description of Loads

See Tables 3.8.4-1 through 3.8.4-13, Tables 3.8.4-15 through 3.8.4-23, and Appendix 3.8E for the loads for other Category I structures. Other Category I structures are subject to the same natural phenomena and basic dead, live, and earth pressure loading as described for the Shield Building in Section 3.8.1.3. In addition to active earth pressure loading described in Section 3.8.1.3, the other Category I structures are designed for at rest and passive earth pressures where applicable.

Construction loads differ for the Auxiliary Building because of the multistory effect of shoring from one floor to the next and the construction crane loading on the Control Building portion.

The maximum temperature gradient for walls above grade with exterior exposure is the same as the normal operating temperature gradient of the Shield Building. The spent fuel pit and fuel transfer canal require additional temperature considerations. Under accident conditions the water was assumed to reach 212°F in 8 hours with the inside building temperature initially at 60°F. The normal temperature of the water in the fuel pit and canal is 120°F.

Hydrostatic pressure loads in the fuel pit and canal vary with water levels in the pit, cask loading area, and canal. The water in the cask loading area is normally maintained at approximately the same level as the water in the spent fuel pool. The canal may be emptied.

The wind and tornado loading are described in Section 3.3. Blowout panels are necessary to restrict the maximum internal pressure to 1.25 psi above the elevation 757.0 floor in the Auxiliary Building as shown between column lines t and y, and A-5 and A-11 in Figure 3.8.4-4 and 3.8.4-5.

The load associated with supports for cable trays, piping systems, and other fastenings to interior reinforced masonry walls was restricted to a maximum of 20 psf over one face of the wall (i.e., 10 p.s.f. on each face).

A 1730 psf surcharge loading was applied to the A-1 and A-15 line walls as a construction loading in the Auxiliary Building.

3.8.4.3.2 Load Combinations and Allowable Stresses

See Tables 3.8.4-1 through 3.8.4-13, Tables 3.8.4-15 through 3.8.4-23 and Appendix 3.8E for the loading combinations and allowable stresses for concrete, miscellaneous steel, and structural steel.

The normal allowable stresses of ACI 318-63 and ACI 318-71 were used for the basic loading combinations of dead, live, earth pressure, hydrostatic ground water to elevation 710.0 (or full pool water levels in the spent fuel pit) and effects of normal temperature gradients.

For additional loads such as induced moments or shears resulting from operating basis earthquake (OBE), accident pressure loading caused by a LOCA or steam pipe rupture and thermal effects corresponding to the accident condition, a 25% increase in steel stress was allowed with concrete stresses restricted to normal allowables.

For construction loading instead of normal live loading or for hydrostatic pressure to elevation 724.4, a 35% increase in both steel and concrete stresses was allowed.

For the combination of the basic loads with safe shutdown earthquake (SSE) effects, or tornado wind loads and associated missiles, or maximum possible flood loads, or impact loadings from jet impingement or jet loading on pipe restraints in conjunction with accident pressures a 67% percent increase in concrete stresses was allowed with steel stresses allowed to reach 0.9 of yield.

The maximum lateral forces generated by the SSE are transmitted to the base through shear walls which are designed in accordance with Section 2631 (c) of the "Recommended Lateral Force Requirements and Commentary", of the Seismology Committed, Structural Engineers Association of California, 1968.

3.8.4.4 Design and Analysis Procedures

3.8.4.4.1 Auxiliary-Control Building

Control Bay Portion

This concrete structure was designed in accordance with the ACI Building Code 318-63 using the elastic working stress theory. The loads, loading combinations, and allowable stresses used are as given in Section 3.8.4.3.2.

The control bay was designed as an independent structure. A standard frame analysis was performed on the building in the design of the main structural walls and a separate analysis was performed for each loading combination. The stage that construction of the building's component walls, slabs, and columns would have progressed by the time of the application of a particular loading, was taken into account and reflected accordingly in the model frame.

The floor slabs at elevations 708.0 and 729.0 were designed by ICES STRUDL-II, Volume I program as flat slabs restrained at the exterior structural walls and supported on concrete columns. At elevation 755.0 the two exterior bays on both ends of the building were designed by ICES STRUDL-II, Volume II program to resist a break in the main steam lines below. The roof slab was designed as a one-way slab spanning between the walls at column lines n and q, as shown in Figure 3.8.4-8. These walls act as shear walls in the event of east-west seismic motion or any other east-west lateral force, with the walls along column lines C1, C3, C11, and C13, as shown in Figure 3.8.4-3 acting as shear walls for north-south lateral forces. The roof slab and floor slabs act as diaphragms. The columns and main structural walls transmit vertical load to the base slab.

The floors and walls of the Auxiliary Building are continuous with the control bay north wall. Dowels and shear keys were provided in the wall in order to provide for this structural continuity.

Procedures used to design the structural steel framing were based on simple beam and column construction as covered in AISC 'Specification for the Design, Fabrication and Erection of Structural Steel for Buildings,' Part 1, with type 2 framing connections. The beams at elevation 755.0, between column lines C3 and C5, and between column lines C9 and C11, were designed to function compositely with the concrete slab through the use of headed concrete anchor studs welded through steel decking to the top beam flanges. The beam-to-beam and beam-to-column connections were typical AISC double angle connections as required by the beam reactions, using either rivets or high strength bolts. Between column lines C5 and C9 the beams were not designed to function compositely. For column line references, see Figures 3.8.4-3 and 3.8.4-4. In these areas horizontal bracing is used to resist the horizontal forces for the support of components such as cable trays, conduit, and pipe supports. At elevation 741.0, there were special connections required that were either bolted or welded in accordance with the codes, standards, and specifications identified in Section 3.8.4.2.1. Transfer of loads into the concrete structure was through bearing plates.

Reinforced concrete partition walls are shown in key plan on Figures 3.8.4-60 through 3.8.4-65. These walls were analyzed as free at the top, fixed at the base, and were designed to resist seismic stresses. A minimum steel percentage of .1 was provided horizontally for each face. A 2-inch space was left between the top of the walls and the bottom of the slab or beam above, in order to ensure that the walls do not act as structural components of the building frame. Each side of this space was filled with a minimum of 2-inch-wide grout.

All reinforced masonry walls are designed in accordance with ACI 531-79 and NUREG-0800, Section 3.8.4, Appendix A.

Control Room Shield Doors

The doors were designed assuming that the entire dead load is carried by the two vertical members in the door directly under the trolleys with the load from the lead shot acting as a fluid pressure load.

The end panels were designed as a fixed beam with uniform load, while the skinplate was designed as a square flat plate stayed at the four corners. The top and bottom members of the door were considered as simple beams.

Earthquakes are the only natural environmental condition which applies to the doors. Being inside the control room, the doors are protected from outside elements.

For design, the earthquake loads for the various parts consisted of the loads produced by an OBE or an SSE. Accelerations due to a SSE are greater than those due to an OBE by a factor of 2. The doors were designed to maintain their structural integrity based on loading conditions resulting from an OBE or an SSE.

Earthquake loads used in design of door and dogs were the greater loads produced by OBE or SSE having peak accelerations at floor elevation 755.0 in the Control Building.

These accelerations were used as static loads for determining component and member sizes. After establishing the component and member sizes, a dynamic analysis, using appropriate response spectra, was made of the door and dogs. This analysis indicated that additional stiffness was required in order to limit the loads on the dogging linkages such that allowable stress would not be exceeded. After adding diagonal stiffeners, another dynamic analysis was made and it was determined that the allowable stresses had not been exceeded. The door assembly was qualified to the Set "B" response spectrum.

Watertight Equipment Hatch Covers

The covers were designed to resist a downward uniform pressure created by a 3-foot head of water caused by a condenser circulating water system (CCWS) rupture in the Turbine Building in addition to the dead load of the structural steel components. The covers were also investigated for a uniform pressure differential of 3.0 psi upward caused by the rapid depressurization during the occurrence of a tornado.

The covers were designed as a structure supported around its periphery. The structural steel members were designed as simply supported beams with uniformly distributed loads. Loads from the covers are transmitted to embedded frames which are continuously supported by concrete. The embedded structural steel frames are solidly anchored in the concrete by headed steel studs welded to the frames.

Auxiliary Building Portion

This concrete structure is designed according to the ACI Building Code 318-63 and the stresses are determined by the working stress method for the principal design cases as shown in Section 3.8.4.3.2. Stresses resulting from the static analysis are combined by the method of superposition with stresses resulting from moments, shears, deflections, and accelerations determined by the dynamic earthquake analysis described in Section 3.7.2. The exterior concrete walls above grade Elevation 728± are designed to resist the tornado-generated missiles as described in Section 3.3.

The condition of rapid depressurization during a tornado is provided for in the following manner. The exterior part of the building is designed for an internal positive pressure of 3 psi occurring in 3 seconds with the following exceptions:

(1) The area above the refueling floor at Elevation 757.0 as illustrated by Figure 3.8.4-3, is designed with blowout panels which open at 0.25 psi. The roof and exterior walls of the spent fuel pool room and cask loading area were evaluated for the effective tornado-generated pressure differential and were found to be within allowable stress limits.

(2) The area below the Elevation 786.0 roof is vented from openings in the roof as illustrated in Figure 3.8.4-8. The roof and walls housing this area are nevertheless designed for 3 psi. The floor at elevation 772.0 below this roof is also designed for an uplift of 3 psi recognizing the venting of the area above this floor.

(3) The heating and ventilating rooms at Elevation 737.0 (see Figure 3.8.4-3) are vented by the air intakes on the exterior walls. This results in the floor, roof, and interior walls of these rooms being designed as exterior member for 3 psi pressure.

The exterior walls below grade Elevation 728$\pm$ are designed for earth pressures. The exterior walls on the east and west ends of the Auxiliary Building are designed as cantilevered retaining walls from Elevation 690 to Elevation 711$\pm$. These walls are built early before any adjacent walls and slabs to allow the construction field force to backfill and have early access to the area at Elevation 711$\pm$. The lateral earth pressure are calculated using Coulomb theory and values are given in Section 3.8.1.3.

The exterior walls north of the Shield Buildings with the buttress walls framing into them, as shown In Figure 3.8.4-2, are designed as cantilevered retaining walls from Elevation 690 to Elevation 727$\pm$ to allow for earth backfill and placement of the Elevation 729 slab on grade.

Horizontal seismic forces are resisted by shear walls with the floor slabs and roof acting as diaphragms. Only those walls parallel to the seismic motion are assumed to resist that motion. The total shear at any level is proportioned among the shear walls in accordance with the method in Reference [3].

For the Safe Shutdown Earthquake, an allowable ultimate shear stress of $5.4\phi\sqrt{f'_c}$ is used. This is the value specified in the SEAOC Code in Section 2631 (c) for walls with a height to width ratio less than one, as is the case for this structure. For the operating Basis Earthquake, an allowable value of one-half of the above is used.

Main steam and feedwater water pipes penetrate the exterior walls of the valve rooms on the south sides of the Shield Building. These penetrations furnish pipe restraints through flued heads embedded in the walls. The flued heads restrict concrete surface temperatures to a maximum of 150° F.

The primary structural support system is designed as a flat slab floor system with concrete columns. Large openings that required separate design are framed with beams. The thickness for many slab sections throughout the building is determined by shielding requirements. The general thickness and live load requirements for different slab areas are shown on Figure 3.8.4-9.

The major portions of the building slabs are designed by the ICES STRUDL-II, Volumes I and II. Moments and shears from small frames, beams, and one-way slabs are designed by the moment distribution method. Where slabs act as two-way slabs due to walls or beams below, moments and shears are determined by use of method 2 of Appendix A in ACI Code 318-63. The effects of the relative deflections of the supports and the effects of column shortening were taken into account in the design of all slabs.

The minimum percentage of reinforcing in the slabs is 0.15% in the top face and 0.18% in bottom face.

The roof slab at elevation 786.0 is designed for 3 psi uplift pressure as a flat slab using the ICES STRUDL-II, Volume I computer program.

The roof at elevation 801.0 is also designed for 3 psi uplift pressure using the ICES STRUDL-II, Volume II Finite Element Method.

In the interior of the building there are many areas around equipment that require shielding which is provided by poured-in-place concrete walls. To permit equipment installation the construction of shielding walls is delayed until the building frame and floor construction is completed and equipment is installed. These shield walls contain minimum steel percentages in the horizontal and vertical directions as specified by the TVA Temperature and Shrinkage Standards and the ACI Code 318-63, Section 2202 (f). These walls were checked for stresses resulting from seismic loading; however, seismic stresses did not control.

Reinforced concrete partition walls are shown in plan on Figures 3.8.4-60 through 3.8.4-65. These walls were analyzed as free at the top, fixed at the base, and were designed to resist seismic stresses. Minimum steel percentages provided were the same as those described for the shield walls. A 2-inch space was left between the top of the walls and the bottom of the slab or beam above in order to ensure that the walls do not act as structural components of the building frame. Each side of this space was filled with a minimum of 2-inch-wide grout.

The thick concrete walls of the spent fuel pit and transfer canal are required for shielding. They are shown in Figure 3.8.4-3. The walls are supported by a concrete base slab, which is approximately 27 feet thick. The walls and base slab are built integrally with the slabs and walls of the Auxiliary Building. A structural wall separates the cask loading area from the spent fuel storage area preventing any loads from being lifted over the spent fuel storage area. The design of the pool walls take into account hydrodynamic effects of the water caused by earthquake and temperature effects caused by failure of the spent fuel cooling system. This structure was designed by moment distribution methods. The stresses in the walls between the spent fuel pit and fuel transfer canal and those between the spent fuel pit and cask loading area were checked by the ICES STRUDL-II, Volume II computer program to determine the effect of the slot in the walls.

Procedures used to design the structural steel framing were based on simple beam and column construction as covered in AISC 'Specification for the Design, Fabrication and Erection of Structural Steel for Buildings,' Part 1, with type 2 framing connections.

Railroad Access Hatch Covers

Structural members for the covers were designed as simple beams. Members of the embedded frame were considered as being rigidly supported by concrete. Loads from the embedded frame are transferred to the concrete by embedded anchors.

The earthquake forces, specified as follows for design, were determined by dynamic analysis including amplification through the supporting structure.

Accelerations for the SSE were used as static loads for determining component and member sizes. After establishing the component and member sizes, a dynamic analysis, using appropriate response spectrum was made of the covers to determine that allowable stresses had not been exceeded.

Railroad Access Door

The horizontal structural members of the door were designed as simple beams with uniformly distributed loads. The end reactions from these members were then transferred to the door end posts as concentrated loads located between rollers. As a conservative design, it was assumed that one roller was not in contact with the track and that the loading from the two horizontal members with the highest reactions was carried by the two adjacent rollers.

The skin plate for the door was designed, without regard to support of the plate from diagonal stiffeners, for the largest open rectangle within the structure. The plate was assumed to be a rectangular diaphragm with fixed edges.

The embedded door frame is rigidly supported by concrete. The portions of the frame which form the door track were designed as cantilever members with loading as applied by the door rollers.

The structural members of the steel enclosure above the door were designed as simple beams and the hoist supports as cantilevers from the Auxiliary Building wall.

Earthquake loads used in design of the door, frame, and track were the loads produced by a SSE having peak accelerations at ground level elevation 729.0, which is the bottom of the door.

Earthquake loads used in design of the hoist supports and enclosure were the loads due to accelerations at the hoist platform, elevation 773.0, produced by a SSE. These accelerations were determined by dynamic analysis of the Auxiliary Building structure. These accelerations were used as static loads for determining component and member sizes. After establishing the component and member sizes, a dynamic analysis, using appropriate response spectra, was made of the door, embedded frame, door track, and hoisting unit enclosure to determine that allowable stresses had not been exceeded.

Manways in the (RHR) Sump Valve Room

In the closed position, each door was considered as a structure supported around the periphery. In the open position, each door was considered as a cantilevered structure with the hinges and hinge anchorages being designed for their loading from the door in the open position. Each embedded frame was considered as being rigidly supported by concrete. Loads from the embedded frame are transferred to the concrete by embedded anchors.

Earthquake loads used in designing the manways were the forces due to accelerations determined for the sump valve room walls at the center of the manways by dynamic analysis of the Auxiliary Buildings for an OBE or SSE. These forces were used as static loads since the manways are rigid and firmly secured to the walls when closed.

Pressure Confining Personnel Doors

Structural members for the doors, in the closed position, were designed as simple beams with end reactions carried by the outside members to the frames which were considered as being rigidly supported by concrete. Loads are transferred to the concrete through embedded anchors or bolt anchors.

In the open position, the doors were designed as cantilever structures with resultant concentrated loads being used for design of the hinge members. For design, the earthquake loads for the various doors consisted of the loads produced by an OBE or SSE.

Earthquake forces due to building accelerations at the elevation of the center of gravity of the various doors were used as static loads for determining door component and member sizes. The building accelerations were determined by dynamic analysis including amplification through the supporting structures. After establishing the component and member sizes, a dynamic analysis, using appropriate response spectra, was made of the doors to determine that allowable stresses had not been exceeded.

Fuel Pool Gates

The gates are designed for a waterhead load of 25.0 feet imposed from the fuel pool side as measured from the centerline of the horizontal bottom seal to the normal pool level at elevation 749.13. The gates are constructed of welded corrosion resistant steel. When dewatering the fuel transfer canal or handling the fuel cask over the fuel cask pit, inflatable elastomer seals provide a near watertight seal between the skin plate and the pool wall liner face.

The gates have been analyzed for the effects of the OBE and SSE for both the operating and stored position. The gates are designed to maintain their sealing and structural integrity during and after an OBE or SSE. Earthquake loading considers simultaneous vertical and horizontal dynamic forces that act on the gates when there is water either on both sides or on the fuel pool side only. The gates are restrained by guides at the top, mid-height, and bottom. When in the storage position, the gates are horizontally restrained by top and bottom guides and vertically supported by hanger brackets.

Waste Packaging Structure

The Waste Packaging Structure Building is a shear wall structure. The structure is basically a box-like structure. The sloping roof of the structure is a series of 19 pre-cast 20-inch thick panels which span one-way from the south to north walls of the structure. The entire roof structure is covered by a 4-inch thick topping slab which is bonded to the pre-cast panels. The response of the structure varies with the direction of loading. Load cases which have lateral motion to the north result in shear wall and roof diaphragm action with a triangular soil distribution developing beneath the structure.

Loading which produces motion southward brings the Waste Packaging Structure Building against the fibrous glass filler material and the Auxiliary Building.

Condensate Demineralizer Waste Evaporator Structure

This two story structure is designed using the loads, loading combinations, and allowable stresses as given in Tables 3.8.4-1 and 3.8.4-2. The concrete portion is designed in accordance with the ACI 318-71 Building Code and the structural steel portion in accordance with AISC 'Manual of Steel Construction,' Seventh Edition. The building is designed to be supported by a bearing pile foundation, with the piles founded on sound rock. The intermediate floor and roof are supported by interior bearing walls and metal decking spanning between steel beams.

3.8.4.4.2 Diesel Generator Building

The structure is designed in accordance with the ACI 318-63 Building Code and is analyzed as a box-type structure assuming all walls fixed at the base slab, elevation 742.0. The frame is analyzed by the moment distribution methods. Floor elevation 760.5 and the roof elevation 773.5 are one-way slabs continuous across interior walls and restrained at exterior walls. All horizontal forces are transmitted through the floor and roof slabs as diaphragms to parallel shear walls and then to the foundation base slab as discussed in Section 3.8.4.4.1 for the Auxiliary Building.

The 9-foot 9-inch base slab distributes superstructure loads uniformly to the supporting crushed stone fill and was analyzed as a flat slab.

The exterior walls and roof of the building are designed to resist the tornado-generated missiles of Spectrum A in Table 3.5-7. Due to the openings in the exterior walls and floor slab at elevation 760.5, the building is assumed to depressurize. In the hallway and stairway the glass is assumed to break in the event of a tornado, thereby preventing pressure buildup.

A reinforced concrete curb is provided to protect the diesel exhaust stacks from closure due to the impact of tornado-generated missiles.

The exhaust stacks extend 24 inches above roof level. The concrete curb is 18 inches thick and extends 12 inches above the exhaust stacks. The fuel oil storage tank vent lines on the roof are encased in concrete to prevent closure due to missile impact. Details of the curbs, exhaust stacks, and fuel oil storage tank vent line encasements are shown on Figures 3.8.4-26 and 3.8.4-33. Missile entry through the air-intake opening in the ceiling over each electrical board room is prevented by the use of steel canopy with barrier protection to intercept missiles. The items discussed above are also listed in Table 3.5-14.

Concrete block walls are shown on Figures 3.8.4-24 and 3.8.4-25. All reinforced masonry walls are designed in accordance with ACI 531-79 and NUREG-0800, Section 3.8.4, Appendix A.

Diesel Generator Building Doors and Bulkheads

Structural members for the doors and bulkheads were designed as simple beams. The skin plates were designed as square or rectangular diaphragms with all edges fixed.

Earthquake loads used in designing the doors and bulkheads were the accelerations determined for ground level elevation 742.0, which is the bottom of the doors, for an SSE. These accelerations were used as static loads for determining component and member sizes. After establishing the component and member sizes, a dynamic analysis was made of the doors and bulkheads.

The precast concrete bulkheads (see Figure 3.8.4-33) covering the doors were analyzed for the missile impact loads discussed in Section 3.8.4.1.2.

3.8.4.4.3 Category I Water Tanks and Pipe Tunnels

See Section 9.2.7 for a description of the refueling water storage tanks.

Pipe Tunnels

The pipe tunnels were analyzed using the ICES STRUDL-II Volume I computer program frame analysis and designed in accordance with the provision of the ACI 318-71 Building Code.

3.8.4.4.4 Class 1E Electrical System Manholes

The manholes were analyzed using a continuous frame or a series of flat plates, depending upon the boundary conditions created by the duct run openings. The frames were modeled using the computer program STRESS. The concept of joint continuity was utilized with the plate analysis, i.e., joints were designed for the larger moment from adjacent plates.

The design of the duct runs assumes bending moments due to earthquake loading are caused by direct imposition of the soil curvature on the duct bank.

3.8.4.4.5 North Steam Valve Room

The concrete structure is analyzed as a three-sided open box structure. The 7-foot-thick base slab is designed to span between the foundation walls. The slab is subjected to a pressure loading due to a main steam pipe rupture as well as anchorage loads from restraints located in the slab. The slab was designed using the SAP IV Finite Element computer program. No support from the soil and crushed stone beneath was assumed in the design of the slab. The main steam and feedwater lines exit from the 4-foot-thick west wall where restraints for these lines are anchored. Pipe restraints are also located in the 5-foot thick interior wall in the east end as well as in the 7-foot by 10-foot-deep beam portion of the north wall. The 5-foot interior wall at the east end stiffens the 3-foot-thick east exterior wall. The 2-foot-thick north wall spans horizontally between the stiff complex of end walls and vertically from the base slab to the 7-foot-thick beam portion. The walls are investigated using the SAP IV Finite Element computer programs.

Design procedures for the roof steel were based on simple beam construction as covered in AISC "Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings," Part 1 with Type 2 framing connections. The metal decking was attached to a cold formed steel frame which in turn is attached to structural steel with the appropriate number of pressure relief fasteners designed to fail allowing the deck/frame to blow off when the internal pressure at the roof reaches 72 pounds per square foot. The deck/frame is restrained from becoming a missile by using wire rope and clamps which are attached to the main concrete structure.

3.8.4.4.6 Intake Pumping Station and Retaining Walls

The box-type structure is analyzed using conventional structural analysis methods. In accordance with ACI 318-71 Code and subsequent addenda, the alternate design method is used in the design of the structure.

The base slab was analyzed as a flat plate fixed on four sides for areas within the walls. The overhanging areas of the base slab were analyzed as a cantilever or flat plate fixed on three sides. The other floors were analyzed as flat plates with either three or four sides fixed. The walls were analyzed as one-way span vertically for the first 20 feet, above that the walls were analyzed as flat plates fixed on four sides. The missile barrier walls around the top deck were analyzed as cantilevers. The structure is investigated as a whole to ensure continuity of design. The structure has also been investigated for stability against overturning, floating, and sliding. In addition, the structure is designed to resist the pressure differential during a tornado and to maintain its stability under all credible environmental conditions. The structural adequacy is also checked for missile penetration.

Concrete Retaining Walls

The concrete retaining walls were analyzed as cantilevers. The walls have also been investigated for stability against overturning and sliding.

Sheet Pile Retaining Walls

The sheet pile walls are connected by ties to a common concrete "dead man" placed midway between the walls in the earthfill. The ties are steel cables and are anchored in the "dead man" at one end and the sheet piling at the other end. The steel walls are on the inside of the sheet piling and bolted to each pile to transfer the reaction of the pile to the wall. The wall was divided into sections and analyzed as a multibraced wall or cantilever wall depending upon the depth of backfill on the wall.

3.8.4.4.7 Miscellaneous ERCW Structures

Slabs and Beams Supporting ERCW Pipes

The slab supporting the ERCW pipes was analyzed by the use of McDonnel-Douglas' ICES STRUDL computer program. Support was assumed to be furnished entirely by the bearing piles and the piles were designed for the reaction from the computer analysis. Missile protection is provided by roller compacted concrete above the pipes.

The beam encasing the ERCW pipes are analyzed as simple beams with no support from the soil. The encased pipes are in the tension zone of the beam; therefore, the design is for a rectangular beam with no special consideration given to the embedded pipe for flexure or shear. The concrete encasement is designed for missile penetration.

Discharge Overflow Structure

The discharge overflow structure was analyzed assuming it as a series of flat plates. The concept of joint continuity was utilized with the plate analysis by designing the joints for the larger moment from adjacent plates.

Standpipe Structures

The standpipe structures consists of a free standing cantilever supported on a flat slab base on in-situ soil. Generally, the structures were considered solid mass concrete and the design was controlled by structural response for missile impact utilizing an elastic analysis.

Valve Covers

The function of these structures is solely to protect the ERCW valves from tornado missiles; therefore, the design was for missile penetration only.

Missile Protection Slabs and Backfill

See Section 3.8.4.1.7.

3.8.4.4.8 Additional Diesel Generator Building

The building is a 96 feet long by 53 feet wide by 32 feet tall (measured from top of base slab) reinforced concrete structure, consisting of a base slab supported by end bearing H-piles, interior floor, roof, and interior and exterior walls. The structure was analyzed as a box-type structure assuming all walls are fixed at the base slab. The building span in the short direction is analyzed using a STRUDL frame program and is designed to withstand all loading conditions assuming a one-way span. In the short direction the interior walls are not considered effective shear walls, but the exterior walls are. Therefore, shear wall and diaphragm deflections are considered in the short direction frame analysis. The building span in the long direction is designed using standard plate theory assuming the interior and exterior walls effectively prevent side sway. The building base slab is a 96 foot long by 53 foot wide by 12 foot thick reinforced concrete slab supported by 154 end-bearing steel H-piles. See Section 3.8.5.5 for additional information on the piles and base slab.

The load definitions, load combinations and allowable stresses are as specified in Section 3.8.4.3.2.

Base Slab Design

The base slab is pile supported. The slab was designed for a uniform live load except where equipment weights dictated a higher value. Equipment loads due to vibration or earthquake acceleration that were transmitted to the slab from anchor bolts were also taken into consideration. In addition, the slab was designed for hydrostatic pressures.

The base slab is a rectangular, cast-in-place, reinforced concrete structure with embedded diesel fuel storage tanks and is supported by piles bearing on rock.

Roof Slab Design

The roof slab was designed for live, seismic, and tornado loads.

Floor Slab

The floor slab is a poured-in-place reinforced concrete slab designed to carry and transmit the floor loads to the building walls. The slab was designed for a uniform live load.

Exterior Walls

The building was designed for tornado venting. However, the exterior walls were designed for tornado, wind and seismic loads.

Fuel Oil Storage Tanks

The steel liner serves no other function except to maintain leak tightness and, therefore, was designed in accordance with ASME Boiler and Pressure Vessel Code, Section VIII, Division I. In addition, the liner was designed to prevent buckling of the steel shell due to the following external loads:

(a) Hydrostatic pressure from underground water.

(b) Shrinkage of the concrete encasement during construction.

(c) Expansion or contraction due to temperature differentials.

For flammable liquids storage requirements, the fuel oil storage tanks meet the requirements of the National Fire Protection Association (NFPA) Code 30, which applies to fuel oil storage tanks supplying underground storage of a Class II liquid (diesel fuel).

Equipment Door

The equipment door is composed of a structural steel frame and covered on both sides with a steel-skin plate.

The removable precast concrete missile barrier bulkheads are placed in front of the equipment doors to provide protection from tornado-generated missiles, which are discussed in Section 3.8.4.1.8. In establishing the required concrete thickness for these missile barriers, no consideration was given to the equipment door. Therefore, these barriers are designed to absorb the full missile impact.

Allowable Settlement

The building was designed to accommodate a settlement of 2 inches, with a differential settlement of 1-inch over a 96-foot structure length.

End-Bearing Steel H-Piles

The piles were designed to withstand and transmit to rock the effects of the design loads and conditions.

Seismic Analysis

The structure was analyzed for the effects of the OBE and the SSE as described in Section 3.7.2.1.1.

3.8.4.5 Structural Acceptance Criteria

3.8.4.5.1 Concrete

The Category I structures were proportioned to maintain elastic behavior and stresses within stress allowables when subject to the loading combinations of Section 3.8.4.3.

Most Category I structures are essentially low profile box structures with height to base ratios less than 1 and a high factor of safety against sliding or overturning under the most severe loading conditions. Those structures with height to base ratios greater than 1 are designed with adequate factors of safety applied to stability. In addition, all structures are designed to flood or have sufficient weight to prevent flotation under maximum flood conditions. For consideration of sliding, overturning, and floatation of the Additional Diesel Generator Building, see the loading combinations and minimum factor of safety in Table 3.8.4-22.

3.8.4.5.2 Structural and Miscellaneous Steel

Structural and miscellaneous steel (including inside containment) and welds are designed in accordance with AISC "Manual of Steel Construction," Seventh Edition, for Case I loading condition so that the stress in the members and connections do not exceed the allowable stress criteria as set forth in the February 1969 AISC "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," as amended through June 12, 1974. For the factor of safety of these allowable stresses with respect to specified minimum yield point of the material used, see Section 1.5 of "Commentary on the Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings." Both specifications and commentary are included in the AISC "Manual of Steel Construction."

For Case II loading condition the actual stresses do not exceed the allowable stresses as set forth in Table 3.8.4-2. The allowable stresses for Case II loading have a minimum factor of safety of 1.11 based on the specified minimum yield point of the material used.

TVA has generally installed, and will continue to install fillet welds to meet the minimum weld size specifications of Table 1.17.5 of AISC Manual of Steel Construction. Where

TVA drawings have specified fillet welds below the minimum sizes specified by AISC, these welds do meet the allowable stress requirements identified above. Weld qualification testing has demonstrated the adequacy of all fillet welds that were installed below minimum AISC specifications.

The Additional Diesel Generator Building structural steel was proportioned to meet the applicable codes discussed in Appendix 3.8E and load combinations in Section 3.8.4.3.

Structural steel and miscellaneous steel, which is highly restrained and is located in a high temperature environment, is evaluated for effects of thermal loads.

3.8.4.5.3 Miscellaneous Components of the Auxiliary Building

Control Room Shield Doors

Allowable stresses for all load combinations used for the various parts of the door and dogs are given in Table 3.8.4-3. For normal load conditions the allowable stresses provide a safety factor of 2 to 1 on yield for structural parts and 5 to 1 on ultimate for mechanical parts. For the limiting condition of SSE, stresses do not exceed 0.9 yield.

Watertight Equipment Hatch Covers

Allowable stresses for all load combinations used for the various parts are given in Table 3.8.4-23. Allowable stresses for normal loading combinations are based on the AISC specification (see Section 3.8.4.2 and Table 3.8.4-23). For limiting conditions, such as SSE, tornado, and flood, stresses do not exceed 0.9 yield.

Railway Access Hatch Covers

Allowable stresses for all load combinations used for the various parts are given in Table 3.8.4-4. For normal load conditions, the allowable stresses provide safety factors of 2 to 1 on yield for structural parts and 5 to 1 on ultimate for mechanical parts. For limiting conditions, such as an OBE or SSE, stresses do not exceed 0.9 yield.

Railroad Access Door

Allowable stresses for all load combinations used for the various parts of the door, embedded frame, and hoist enclosure are given in Table 3.8.4-5. For normal load conditions the allowable stresses provide a safety factor of 2 to 1 on yield for structural parts and 5 to 1 on ultimate for mechanical parts. For limiting conditions such as an OBE or SSE and hoist stall, stresses do not exceed 0.9 yield.

Manways in RHR Sump Valve Room

Allowable stresses for all load combinations used for the various parts are given in Table 3.8.4-6. For limiting conditions, such as a SSE, stresses do not exceed 0.9 yield.

Pressure Confining Personnel Doors

Allowable stresses for all load combinations used for the various parts are given in Table 3.8.4-7. For normal load conditions, the allowable stresses provide safety factors of 2 to 1 on yield on structural parts and 5 to 1 on ultimate for mechanical parts. For limiting conditions, such as an SSE, flood, and tornado loadings, stresses do not exceed 0.9 yield.

Fuel Pool Gates

Allowable stresses for all load combinations used for the gates are given in Table 3.8.4-21. For normal load conditions the allowable stresses do not exceed 0.6 of yield. For limiting conditions, such as the SSE, the stresses do not exceed 0.90 of yield.

3.8.4.5.4 Intake Pumping Station Traveling Water Screens

Allowable stresses for all load combinations used for the various parts are given in Table 3.8.4-11. For normal load conditions, the allowable stresses provide safety factors of 1.79 (F_y/0.56 F_y) to 1 on yield for structural parts and 5 to 1 on ultimate for mechanical parts. For limiting conditions, such as a safe shutdown earthquake, stresses do not exceed 0.9 yield.

3.8.4.5.5 Diesel Generator Building Doors and Bulkheads

Load combinations and allowable stresses for all combinations are given in Table 3.8.4-13. For missile impact, yield point of material will be exceeded and the member practically deform. For normal load condition, the allowable stresses provide safety factors of 1.67 (F_y/0.6 F_y) to 1 on yield for structural parts and 5 to 1 on ultimate for mechanical parts. For limiting conditions, except for missile impact, stresses do not exceed 0.9 yield.

3.8.4.5.6 Additional Diesel Generator Building Missile Barriers

Design of missile barriers for the Additional Diesel Generator Building is discussed in Section 3.5.3.1.

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

General

See Section 3.8.1.6.

3.8.4.6.1 Materials

See Section 3.8.1.6.1.

For the Additional Diesel Generator Building the following materials were used:

Structural Steel

Rolled shapes, plates, and bars meet Specification ASTM A 36. Fabricated high-strength steel meets Specification ASTM A 572 and bolting meets Specification ASTM A 325 or A 490. Anchor bolts meet ASTM A 307 or A 36 steel.

Reinforcing Steel

The yield strength of reinforcing steel used in the building is 60,000 psi (ASTM A 615, grade 60) or greater.

Concrete

The compressive strength of concrete is 3000 psi or greater.

3.8.4.6.2 Quality Control

Concrete production and testing are discussed in Section 3.8.1.6.2.

In addition to the 4,000-psi-at-28-days mix discussed in Section 3.8.1.6.2, a 3,000-psi-at-28-days mix, a 3,000-psi-at-90-days mix, a 5,000-psi-at-28-days mix, and a 4,000-psi-at-90-days mix were used. Some concrete did not meet specification requirements. This was evaluated and documented in the report CEB-86-19-C "Concrete Quality Evaluation."[2]. Results have been documented in affected calculation packages and drawings.

Testing of reinforcing steel and cadweld splices is discussed in Section 3.8.1.6.2.

The control room shield doors, watertight equipment hatch covers, railway access hatch covers, railroad access doors, equipment hatch doors and sleeves, manways in the RHR sump valve room, and the pressure confining personnel doors were designed and erected by TVA in accordance with TVA's quality assurance program. Design and fabrication by the contractor were in accordance with the contractor's quality assurance program which was reviewed and approved by TVA's design engineers. The contractor's quality assurance program covers the criteria in Appendix B of 10 CFR 50.

Fabrication procedures such as welding and nondestructive testing were included in appendices to the contractor's quality assurance program.

ASTM standards were used for all material specifications and certified mill test reports were provided by the contractor for materials used for load-carrying members.

The fuel pool gates were designed and procured before quality assurance requirements were imposed. An evaluation was conducted to verify that the gates were equivalent to gates that would have been designed and fabricated under a quality assurance program.

3.8.4.6.3 Special Construction Techniques

No special construction techniques were used, except for the fuel oil storage tanks in the Additional Diesel Generator Building. For these tanks joint welding procedures used in fabrication of the steel liner were qualified in accordance with ASME Boiler and Pressure Vessel Code, Section IX, prior to use by TVA or the fabricator.

3.8.4.7 Testing and Inservice Surveillance Requirements

Testing for the steel liners in the fuel oil storage tanks for the Additional Diesel Generator Building was accomplished by subjecting them to a standard hydrostatic test in accordance with ASME, Section VIII.

3.8.4.7.1 Concrete and Structural Steel Portions of Structures

A program to monitor the settlement of other Category I structures is as shown in Figures 3.8.4-66 and 3.8.4-67.

3.8.4.7.2 Miscellaneous Components of Auxiliary-Control Building

Control Room Shield Doors

After erection and adjustment the doors were inspected for proper operation of the dogs and free movement on the trolleys.

After the initial inspection, periodic visual inspections of the doors are to be made. Parts inspected during these visual inspections are to include connections to trolleys, structural members for paint deterioration, and dogs.

Watertight Equipment Hatch Covers

After initial inspection, periodic visual inspections of the covers are to be made. A visual inspection is made of all screws to see that they are securely tightened and that none are missing. The painted inscriptions on the covers are inspected for any deterioration. In the event that the hatch covers are removed, an inspection is made of the gaskets to ensure that they are clean and free of any damage or deterioration which would prevent their forming a proper seal. The embedded frames are inspected to ensure that the mating surfaces are clean and free of foreign material before the covers are reinstalled.

Railway Access Hatch Covers

After the initial inspection, periodic visual inspections of the covers are made. Parts inspected during the visual inspection are to include all bolted connections, structural members for paint deterioration, limit switches, and rubber seals. The seals are carefully, inspected for cracks, blemishes, or any other indications of deterioration of the rubber and for properly seating at the sealing surfaces.

Railway Access Door

Prior to shipment of the door from the contractor's plant, the splice welds in the skin plate of the door and welds among the periphery of the skin plate and structural members were magnetic particle tested.

After completion of the initial tests and inspection, periodic visual inspections of the door and its parts are made. Parts inspected are to include all bolted connections, limit switches, door tracks, and rollers. Painting is to be inspected for evidence of deterioration, and the seals are carefully inspected for cracks, blemishes, or any other indications of deterioration of the rubber.

Pressure Confining Personnel Doors

After the initial inspection, periodic visual inspections of the doors are made. Parts inspected during these visual inspections include all bolted connections, structural members for paint deterioration, latching or dogging mechanisms and limit switches for physical condition, and the seals. The seals are carefully inspected for cracks, blemishes, or any other indications or deterioration and for proper seating at the sealing surfaces.

Fuel Pool Gates

After initial inspection, periodic visual inspection of the gates are made. The seals are carefully inspected for cracks, blemishes, or any other indications of deterioration.

3.8.4.7.3 Deleted by Amendment 79

3.8.4.7.4 Miscellaneous Components of the Intake Pumping Station

Traveling Water Screens

After the initial inspection and testing, the screens are inspected at periodic intervals. Parts inspected include drive components, carrier chains, baskets including the wire panels, spray pipes, spray nozzles, main frames, lights, and lubricating devices.

Water Tight Doors

After the initial inspection, periodic visual inspections of the doors are made. Parts inspected during these visual inspections include the bolted connections, structural members for paint deterioration, latching or dogging mechanisms and limit switches for physical condition, and the seals. The seals are carefully inspected for cracks, blemishes, or any other indications or deterioration and for proper seating at the sealing surfaces.

References

(1) TVA drawing series 46W454 "Architectural Door and Hardware Schedule".

(2) TVA Civil Engineering Branch Report Number CEB-86-19-C, "Concrete Quality Evaluation"

(3) Portland Cement Association publication, T18-4, "Analysis of Small Reinforced Concrete Buildings for Earthquake Forces," pp. 30-32.

Table 3.8.4-1 Auxiliary-Control Building Concrete Structure Loads, Loading Combinations And Allowable Stresses (Page 1 of 4)

I. Loads

The following terms are used in the load combination equations.

C	=	Construction condition.
C'	=	Crane load, including wind on crane.
D	=	Dead load of structure and equipment plus any other permanent load contributing stress, such as soil pressure. Hydrostatic pressure from ground water Elevation 710, exterior walls; Elevation 726, uplift.
D'	=	D + hydrostatic pressure from ground water Elevation 724.4.
E	=	Operating Basis Earthquake
E'	=	Safe Shutdown Earthquake
H	=	Spent fuel pit hydrostatic pressure. Worst condition of the following except as noted:
		1. Normal water level in pit, cask loading area and canal.
		2. Canal empty of water. Normal water level in other areas.
		3. Cask loading area empty of water. Normal water level in other areas. Considered for Case I load combinations only.
L	=	Live load. For live load on slabs, see Figure 3.8.4-9.
P	=	Accidental drop of fuel cask on walls of cask loading area.
T_a	=	Accidental increase in temperature of water in pit to 212°F in 8 hours. Temperature inside building 60°F.
T_N	=	Normal temperature of water in fuel pit and canal 120°F. Temperature inside building 60°F.
W	=	Wind load, see Section 3.3.
Wt	=	Tornado, see Section 3.3.
Pa	=	Pressure from main steam break.
Ra	=	Pipe reaction from thermal effects of main steam break.
Ta	=	Thermal effects from main steam break.
Yr	=	Pipe anchor force due to jet from pipe break.
Yj	=	Jet force from pipe break.
Ym	=	Missile impact force from pipe whip.

Table 3.8.4-1 Auxiliary-Control Building Concrete Structure Loads, Loading Combinations And Allowable Stresses (Page 2 of 4)

II. Load Combinations and Allowable Stresses

Auxiliary-Control Building

Load combinations	Allowable WSD Stresses
Case I = D+L	Normal (ACI 318-63 or 318-71)
Case Ia = D'+L	1.35 x normal
Case Ib = D+L+W+C	1.33 x normal
Case II = D+L+E	fc = normal (ACI 318-63 or 318-71) fs = 0.50 fy
Case III = D+L+E'	*fc = 0.75 f'c fs = 0.90 fy
Case IV = D+L+W_t	*fc = 0.75 f'c fs = 0.90 fy

Where main steam lines pass through the Auxiliary-Control Building at the south main steam valve room, the following factored load combinations were considered in addition to those listed above:

Load Combinations	Allowable WSD Stresses	USD Load Factors
Case VI = D+L+P_a	*f_c = .75f'_c f_s = .9f_y	1.0D+1.0L+1.5P_a
Case VII = D+L+P_a+1.0 (Y_r+Y_j+Y_m)+1.0E	*f_c = .75f'_c f_s = .9f_y	1.0D+1.0L+1.25P_a+ 1.0(Y_r+Y_j+Y_m)+1.25E
Case VIII = D+L+1.0P_a+1.0 (Y_r+Y_j+Y_m)+1.0E'	*f_c = .75f'_c f_s = .9fy	1.0(D+L+P_a+Y_r+Y_j+Y_m +E')

* Concrete stresses other than flexure = 1.67 x normal

Where the main steam lines pass under the elevation 755.0 floor slab of the Conrol Building, vital structural elements in that area weredesigned for cases I through IV and the case IX listed below.

	Allowable WSD Stresses
Case IX = D+E'+J*	f_c = 1.67 (Normal Concrete) f_s = 0.9 f_y

* J is a jet load of 360 kips spread over 50 ft^2

Table 3.8.4-1 Auxiliary-Control Building Concrete Structure Loads, Loading Combinations And Allowable Stresses (Page 3 of 4)

Material Properties	
Concrete	
Slabs and walls	f'_c = 3000 or 4000 psi
Columns	f'_c = 4000 psi
Concrete weight	w = 145 pcf
Reinforcing steel	f_y = 60,000 psi (ASTM A615. Grade 60)
Auxiliary Building Spent Fuel Pit	
Load Combinations	Allowable WSD Stresses
Case I = D+H	Normal (ACI 318-63)
= D+H+T_N	Normal (ACI 318-63)
Case II = D+H+E	f_c= normal (ACI 318-63) f_s= 0.50 f_y
= D+H+E+T_N	f_c= normal (ACI 318-63) f_s= 0.50 f_y
Case III = D+H+E'	*f_c= 0.75 f'_c f_s= 0.90 f_y
= D+H+E'+T_N	*f_c= 0.75 f'_c f_s= 0.90 f_y
Case IV = D+H+T_a	*f_c= 0.75 f'_c f_s= 0.90 f_y
Case IVa = D+H+P	*f_c= 0.75 f'_c f_s= 0.90 f_y
*Concrete stresses other than flexure = 1.67 x normal.	
Material Properties (see above)	

Table 3.8.4-1 Auxiliary-Control Building Concrete Structure Loads, Loading Combinations And Allowable Stresses (Page 4 of 4)

Auxiliary Building Concrete Structure Earth Values	
Angle of internal friction	$\Phi = 32^\circ$
Angle of friction between fill and structure	$\Phi_F = 16^\circ$
Unit weight of fill	
Dry	w = 120 psf
Saturated	w = 65 psf
Surcharge	
A1 and A15 line walls	1730 psf
Others	200 psf

Table 3.8.4-2 Auxiliary-Control Building Structural Steel Loads, Loading Conditions And Allowable Stresses (Page 1 of 4)

Control Building Portion

1. Live Loads (LL)
   a. Elevation 755.0 - 400 psf (to include cable trays, ducts, walls, and electrical boards)
   b. Elevation 741.0 - 100 psf plus equipment loads when seismic loads (E and E') are not present 10 psf; when seismic loads are present
   c. Elevation 729.0 - 100 psf
2. Dead Loads (DL)
   a. 8-inch concrete brick wall - 100 psf
   b. 1-1/2-inch steel grating - 12 psf
   c. Concrete - 12.5 psf per inch thickness
   d. Steel framing - 15 psf
   e. Piping - varies
3. Tornado (TOR)
   a Elevation 729.0 - 3.0 psi (between column lines C-1 to C-3 and C-11 to C-13).

Auxiliary Building Portion

1. Live Loads (LL) - The following loads shall be used unless shown otherwise on Figure 3.8.4-9, "Concrete Floor Design Data."
   a. Construction load - 20 psf
   b. Miscellaneous live load - 30 psf
2. Dead Loads (DL)
   a. Concrete - 12.5 psf per inch thickness
   b. Steel roof decking - 4 psf
   c. Steel roof framing - 30 psf
   d. Steel floor framing - 15 psf
3. Tornado (TOR)
   a. Velocity - 360 mph

Table 3.8.4-2 Auxiliary-Control Building Structural Steel Loads, Loading Conditions And Allowable Stresses (Page 2 of 4)

Auxiliary-Control Building

Seismic Loads

a. Operating Basis Earthquake (OBE) maximum ground acceleration
Horizontal 0.09g
Vertical 0.06g

b. Safe Shutdown Earthquake (SSE) maximum ground acceleration
Horizontal 0.18g
Vertical 0.12g

Loading Condition	Tension on Net Section	Shear on Gross Section	Compression on Gross Section	Bending	Concrete Bearing
Case I DL + LL + OBE	0.60 FY	0.40 FY	See Note 1	0.66 FY to -0.60 FY	0.25 f'c
Case II DL + LL + SSE	0.90 FY	$\frac{0.9\ FY}{\sqrt{3}}$	See Note 2	0.90 FY	.595 f'c See Note 3.
Case III DL + LL + TOR	0.90 FY	$\frac{0.9\ FY}{\sqrt{3}}$	See Note 2	0.90 FY	.595 f'c See Note 3.

Note 1 - Varies with slenderness ratio, see AISC "Manual of Steel Construction," 7th Edition, Table 1-36, Page 5-84.

Note 2 - Varies with slenderness ratio:

Main and secondary members, where KL/r $\leq$ Cc: (A)

$$F_a = 0.9F_Y\left(1 - \frac{(KL/r)^2}{2C_c^2}\right)$$

Table 3.8.4-2 Auxiliary-Control Building Structural Steel Loads, Loading Conditions And Allowable Stresses (Page 3 of 4)

Main members, where C_c < KL/r < 200:

$$F_a = \frac{0.9\pi^2 E}{(KL/r)^2} \qquad \text{(B)}$$

Secondary members, where 120 < L/r $\leq$ 200:

$$F_{as} = \frac{F_a \text{ [by Formula (A) or (B)]}}{1.6 - L/200r}$$

Where:

$$C_c = \sqrt{\frac{2\pi^2 E}{F_Y}}$$

E = Modulus of elasticity of steel (29,000 kips per square inch)

F_a = Axial compressive stress permitted in the absence of bending moment (kips per square inch)

F_{as} = Axial compressive stress, permitted in the absence of bending moment, for bracing and other secondary members (kips per square inch)

F_Y = Specified minimum yield stress of material (kips per square inch)

f'_c = Compressive strength of concrete

K = Effective length factor

L = Actual unbraced length (inches)

r = Governing radius of gyration (inches)

Material Properties

Steel Properties

Cc = 126.1

E = 29,000,000 psi

F_Y = 36,000 psi

Table 3.8.4-2 Auxiliary-Control Building Structural Steel Loads, Loading Conditions And Allowable Stresses (Page 4 of 4)

Note 3 -When the supporting surface is wider on all sides than the loaded area, the permissible bearing stress on the loaded area

may be multiplied by $\sqrt{A_2 / A_1}$, but not more than 2.

Where: A1 = Loaded area

A2 = Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area.

Table 3.8.4.-3 Control Room Shield Doors Loads, Loading Combinations, And Allowable Stresses

Door and Jamb Shield Assemblies Structural Parts

		Allowable Stresses (psi)		
No.	Load Combinations	Tension	Compression(2)	Shear
	Doors Open or Closed			
I	Dead	$0.50F_y$	$0.47F_y$	$0.33F_y$
II	Dead + OBE(1)	$0.60F_y$	$0.60F_y$	$0.40F_y$
III	Dead + SSE(1)	$0.90F_y$	$0.90F_y$	$0.60F_y$

Mechanical Parts

		Allowable Stresses (psi)		
No.	Load Combinations	Tension	Compression(2)	Shear
	Doors Open or Closed			
I	Dead	$\frac{\text{Ultimate}}{5}$	$\frac{\text{Ultimate}}{5}$	$\frac{\text{Ultimate}}{7.5}$
II	Dead + OBE(1)	$0.6F_y$	$0.6F_y$	$0.4F_y$
III	Dead + SSE(1)	$0.9F_y$	$0.9F_y$	$0.6F_y$

Notes:

(1) Acts in any one horizontal direction only at any given time and acts in vertical and horizontal directions simultaneously.

(2) The value given for allowable compression stress is the maximum value permitted when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_y when buckling controls.

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] \text{ when } \frac{Kl}{r} \le C_c \qquad 1$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \frac{Kl}{r} > C_c \qquad 2$$

Table 3.8.4-4 Auxiliary Building Railroad Access Hatch Covers Loads, Loading Combinations, And Allowable Stresses (Page 1 of 2)

Cover Structure and Embedded Frame

No.	Load Combinations	Allowable Stresses (psi) Tension	Compression[1]	Shear
	Covers Closed			
I	Dead load plus live load at 100 lb/ft^2	$0.50F_y$	$0.47F_y$	$0.33F_y$
II	Dead load plus live load at 100 lb/ft^2 plus OBE	$0.60F_y$	$0.60F_y$	$0.40F_y$
III	Dead load plus live load at 100 lb/ft$_2$ plus SSE	$0.90F_y$	$0.90F_y$	$0.60F_y$
	Covers Open			
IV	Dead load plus hoist pull	$0.50F_y$	$0.47F_y$	$0.33F_y$
V	Dead load plus hoist pull plus OBE	$0.60F_y$	$0.60F_y$	$0.40F_y$
VI	Dead load plus hoist pull plus SSE	$0.90F_y$	$0.90F_y$	$0.60F_y$

Mechanical Parts on Covers and Frame

No.	Load Combinations	Allowable Stresses (psi) Tension and Compression[1]	Shear
	Covers Closed		
I	Dead load plus live load at 100 lb/ft^2	$\frac{\text{Ult}}{5}$	$\frac{2 \times \text{Ult}}{15}$
II	Dead load plus live load of 100 lb/ft^2 plus OBE	$0.6F_y$	$0.4F_y$
III	Dead load plus live load at 100 lb/ft^2 plus SSE	$0.9F_y$	$0.6F_y$
	Covers Open		
IV	Dead load plus hoist pull	$\frac{\text{Ult}}{5}$	$\frac{2 \times \text{Ult}}{15}$
V	Dead plus live load of 100 lb/ft^2 plus OBE	$0.6F_y$	$0.4F_y$
VI	Dead load plus hoist pull plus SSE	$0.9F_y$	$0.6F_y$

Table 3.8.4-4 Auxiliary Building Railroad Access Hatch Covers Loads, Loading Combinations, And Allowable Stresses (Page 2 of 2)

Hoist Unit Supports				
		Allowable Stresses (psi)		
No.	Load Combinations	Tension	Compression(1)	Shear
	Hatch Opening			
I	Dead load Hoist pull	18,000	17,000	12,000
II	Dead load Stall	32,400	32,400	21,600

Other Mechanical Parts			
		Allowable Stresses (psi)	
No.	Load Combinations	Tension and Compression(1)	Shear
	Covers Open		
I	Dead load Hoist pull	$\frac{\text{Ult}}{5}$	$\frac{2 \times \text{Ult}}{15}$
II	Dead load Stall	$0.9F_y$	2/3 x $0.9F_y$

(1) The value given for allowable compression stress is the maximum value, F_{cr}, permitted when buckling does not control. The critical buckling stress shall be used in place of F_y when buckling controls.

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] \text{ when } \frac{Kl}{r} \le C_c \quad 1$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \left(\frac{Kl}{r} > C_c\right) \quad 2$$

Table 3.8.4-5 Railroad Access Door Loads, Loading Combinations, and Allowable Stresses Door, Embedded Frame and Door Track (Page 1 of 2)

		Allowable Stresses (psi)		
No.	Load Combinations	Tension	Compression[2]	Shear
	Door Closed			
I	Dead load plus windload at 10 lb/ft^2	$0.50F_y$	$0.47F_y$	$0.33F_y$
II	Dead load plus windload at 30 lb/ft^2	$0.90F_y$	$0.90F_y$	$0.60F_y$
III	Dead load plus windload at 10 lb/ft^2 plus OBE	$0.60F_y$	$0.60F_y$	$0.40F_y$
IV	Dead load plus windload at 10 lb/ft^2 plus SSE	$0.90F_y$	$0.90F_y$	$0.60F_y$
	Door Open			
V	Dead load plus hoist pull	$0.50F_y$	$0.47F_y$	$0.33F_y$
VI	Dead load plus hoist pull plus OBE	$0.60F_y$	$0.60F_y$	$0.40F_y$
VII	Dead load plus hoist pull plus SSE	$0.90F_y$	$0.90F_y$	$0.60F_y$

Hoist Unit & Enclosure

		Allowable Stresses (psi)		
No.	Load Combinations	Tension	Compression[2]	Shear
I	Dead load plus hoist pull	$0.50F_y$	$0.47F_y$	$0.33F_y$
II	Dead load plus stall	$0.90F_y$	$0.90F_y$	$0.60F_y$
III	Dead load plus hoist stall plus OBE	$0.60F_y$	$0.60F_y$	$0.40F_y$
IV	Dead load plus hoist pull plus SSE	$0.90F_y$	$0.90F_y$	$0.60F_y$

Mechanical Parts on Door

		Allowable Stresses (psi)	
No.	Load Combinations	Tension and Compression(2)	Shear
	Door Open		
I	Dead load plus windload at 10 lb/ft^2	$\frac{Ult}{5}$	$\frac{2x\ Ult}{15}$
II	Dead load plus windload a 10 lb/ft^2 plus OBE	$0.6F_y$	$0.4F_y$
III	Dead load plus windload at 10 lb/ft^2 plus SSE(1)	$0.9F_y$	$0.6F_y$

Table 3.8.4-5 Railroad Access Door Loads, Loading Combinations, and Allowable Stresses Door, Embedded Frame and Door Track (Page 2 of 2)

Other Mechanical Parts			
		Allowable Stresses (psi)	
No.	Load Combinations	Tension and Compression[(2)]	Shear
	Door Open		
I	Dead load Hoist pull	$\frac{\text{Ult}}{5}$	$\frac{2 \times \text{Ult}}{15}$
II	Dead load Stall	$0.9F_y$	$0.6F_y$

Note:

(1) Acts in one horizontal direction only at any given time and acts in the horizontal and vertical directions simultaneously.

(2) The value indicated for the allowable compression stresses is the maximum value permitted when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_y when buckling controls.

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] \text{ when } \frac{Kl}{r} \le C_c \qquad 1$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \frac{Kl}{r} > C_c \qquad 2$$

Table 3.8.4-6 Manways In RHR Sump Valve Room Loads, Loading Combinations, and Allowable Stresses (Page 1 of 2)

Structural Parts			
		Allowable Stresses (psi)	
No.	Load Combinations	Tension and Compression(2)	Shear
	Manway Closed		
I	Dead load plus OBE(1)	$0.6F_Y$	$0.4F_Y$
II	Dead load plus SSE(1)	$0.9F_Y$	$0.6F_Y$
III	Dead load plus 19 psi from outside	$0.9F_Y$	$0.6F_Y$
	Manway Open		
IV	Dead load plus OBE(1)	$0.6F_Y$	$0.4F_Y$
V	Dead load plus SSE(1)	$0.9F_Y$	$0.6F_Y$
Mechanical Parts			
		Allowable Stresses (psi)	
No.	Load Combinations	Tension and Compression(2)	Shear
	Manway Closed		
I	Dead load plus OBE(1)	$0.6F_Y$	$0.4F_Y$
II	Dead load plus SSE(1)	$0.9F_Y$	$0.6F_Y$
III	Dead load plus 19 psi from outside	$0.9F_Y$	$0.6F_Y$
	Manway Open		
IV	Dead load plus OBE(1)	$0.6F_Y$	$0.4F_Y$
V	Dead load plus SSE(1)	$0.9F_Y$	$0.6F_Y$

Notes:

(1) Acts in one horizontal direction only at any given time and acts in vertical and horizontal directions simultaneously.

(2) The values given for allowable compression stress is the maximum value permitted when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_Y when buckling controls.

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] \text{ when } \frac{Kl}{r} \le C_c \qquad 1$$

or

Table 3.8.4-6 Manways In RHR Sump Valve Room Loads, Loading Combinations, and Allowable Stresses (Page 2 of 2)

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \left(\frac{Kl}{r} > C_c\right) \qquad 2$$

Table 3.8.4-7 Pressure Confining Personnel Doors Loads, Loading Combinations, and Allowable Stresses[1] (All Doors as shown in Table 3.8.4-7a except A55, A57, C20, C26, A101, A105, A216, and A217)
(Page 1 of 7)

Structural Parts				
			Allowable Stresses (psi)	
No.	Load Combinations	Tension	Compression[2]	Shear
	Doors Open or Closed			
I	DL + Load from Door Closers	0.50 F_y	0.47 F_y	0.33 F_y
II	DL + OBE + Load from Door Closers	0.60 F_y	0.60 F_y	0.40 F_y
	DL + SSE + Load from Door Closers	0.90 F_y	0.90 F_y	0.60 F_y
	Doors Closed			
III[3]	DL + 3-psi pressure (bidirectional where applicable)	0.90 F_y	0.90 F_y	0.60 F_y
IV[4]	DL + OBE + 2-psi toward annulus	0.60 F_y	0.60 F_y	0.40 F_y
	DL + SSE + 2-psi toward annulus	0.90 F_y	0.90 F_y	0.60 F_y
V[5]	DL + 3 inches of water pressure on either side of door	0.50 F_y	0.47 F_y	0.33 F_y
VI[6]	DL + Flood to elevation 738.8	0.90 F_y	0.90 F_y	0.60 F_y

Table 3.8.4-7 Pressure Confining Personnel Doors Loads, Loading Combinations, and Allowable Stresses[1] (All Doors as shown in Table 3.8.4-7a except A55, A57, C20, C26, A101, A105, A216, and A217)
(Page 2 of 7)

1. Thermal load effects are insignificant and hence need not be considered in the design of doors.

2. The values indicated for the allowable compression stresses are the maximum values permitted, when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_y when buckling controls.

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] \text{ when } \frac{Kl}{r} \le C_c \qquad 1$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \frac{Kl}{r} > C_c \qquad 2$$

3. Applies to all doors except A64, A65, A77, A78, A56, A60, A111, A117, A118, A122, A125, A130, A133, A151, A160, A162, A183, A192, A206, A207, A208, A209, A212, A213, C37, C49, C50, C53, C60, DE1, DE4 and DE5.

4. Applies to doors A64 and A65 only.

5. For doors A56, A60, A65, A78, A94, A99, A111, A122, A123, A125, A130, A132, A133, A151, A152, A159, A160, A161, A162, A183, A192, A206, A207, A208, A209, A212, A213, A214, A215, DE1, DE4, and DE5, the load combination is:

   DL + 1/2" water pressure on either side of door.

   For doors C36, C37, C49, C50, C53, C54 and C60, the load combination is:

   DL + 1/8" water pressure on either side of door.

6. Applies to door A65 and A78 only.

Table 3.8.4-7 Pressure Confining Personnel Doors Loads, Loading Combinations, and Allowable Stresses[1] (All Doors as shown in Table 3.8.4-7a except A55, A57, C20, C26, A101, A105, A216, and A217)
(Page 3 of 7)

Mechanical Parts

No.	Load Combinations	Allowable Stresses (psi) Tension and Compression[2]	Shear
	<u>Doors Open or Closed</u>		
I	DL + Load from door closers	$F_u/5$	$2\ F_u/15$
II	DL + OBE + Load from door closers	$0.60\ F_y$	$0.40\ F_y$
	DL + SSE + Load from door closers	$0.90\ F_y$	$0.60\ F_y$
	<u>Doors Closed</u>		
III[3]	DL + 3-psi pressure (bidirectional where applicable)	$0.90\ F_y$	$0.60\ F_y$
IV[4]	DL + OBE + 2-psi toward annulus	$0.60\ F_y$	$0.40\ F_y$
	DL + SSE + 2-psi toward annulus	$0.90\ F_y$	$0.60\ F_y$
V[5]	DL + 3 inches of water pressure on either side of door	$F_u/5$	$2\ F_u/15$
VI[6]	DL + Flood to elevation 738.8	$0.90\ F_y$	$0.60\ F_y$

Table 3.8.4-7 Pressure Confining Personnel Doors Loads, Loading Combinations, and Allowable Stresses[1] (All Doors as shown in Table 3.8.4-7a except A55, A57, C20, C26, A101, A105, A216, and A217)
(Page 4 of 7)

1. Thermal load effects are insignificant and hence need not be considered in the design of doors.
2. The values indicated for the allowable compression stresses are the maximum values permitted, when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_y when buckling controls.

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] \text{ when } \frac{Kl}{r} \le C_c \qquad 3$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \frac{Kl}{r} > C_c \qquad 4$$

3. Applies to all doors except A64, A65, A77, A78, A56, A60, A111, A117, A118, A122, A125, A130, A133, A151, A160, A162, A183, A192, A206, A207, A208, A209, A212, A213, C37, C49, C50, C53, C60, DE1, DE4 and DE5.

4. Applies to doors A64 and A65 only.

5. For doors A56, A60, A65, A94, A111, A113, A114, A122, A123, A125, A130, A132, A133, A151, A152, A159, A160, A161, A162, A183, A192, A206, A207, A208, A209, A212, A213, A214, A215, DE1, DE4, and DE5, the load combination is:

   DL + 1/2" water pressure on either side of door.

   For doors C36, C37, C49, C50, C53, C54, and C60, the load combination is:

   DL + 1/8" water pressure on either side of door.

6. Applies to door A65 and A78 only.

Table 3.8.4-7 Pressure Confining Personnel Doors Loads, Loading Combinations, and Allowable Stresses[1] (All Doors as shown in Table 3.8.4-7a except A55, A57, C20, C26, A101, A105, A216, and A217)
(Page 5 of 7)

(Doors A55, A57, C20, C26, A101, and A105)

Structural Parts

No.	Load Combinations	Allowable Stresses (psi) Tension	Compression[2]	Shear
	Doors Open			
I	DL + Load from Door Closers	$0.50\ F_y$	$0.47\ F_y$	$0.33\ F_y$
II	DL + OBE + Load from Door Closers	$0.60\ F_y$	$0.60\ F_y$	$0.40\ F_y$
	DL + SSE + Load from Door Closers	$0.90\ F_y$	$0.90\ F_y$	$0.60\ F_y$
	Doors Closed			
III[3]	DL + CCWS Flood + OBE + 3-psi pressure (bidirectional where applicable)	$0.60\ F_y$	$0.60\ F_y$	$0.40\ F_y$
IV[3]	DL + CCWS flood + SSE + 3-psi pressure (bidirectional where applicable)	$0.90\ F_y$	$0.90\ F_y$	$0.60\ F_y$
V[4]	DL + OBE + Pressure from valve rooms	$0.60\ F_y$	$0.60\ F_y$	$0.40\ F_y$
	DL + SSE + Pressure from valve rooms	$0.90\ F_y$	$0.90\ F_y$	$0.60\ F_y$

1. Thermal load effects are insignificant and hence need not be considered in the design of doors.

2. The values indicated for the allowable compression stresses are the maximum values permitted, when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_y when buckling controls.

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] \text{ when } \frac{Kl}{r} \le C_c \qquad 5$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \frac{Kl}{r} > C_c \qquad 6$$

3. The CCWS flood condition does not apply to doors A101 and A105, and differential pressure load due to tornado need not be considered simultaneously with seismic load.

4. Applies to doors A101 and A105 only.

Table 3.8.4-7 Pressure Confining Personnel Doors Loads, Loading Combinations, and Allowable Stresses[1] (All Doors as shown in Table 3.8.4-7a except A55, A57, C20, C26, A101, A105, A216, and A217)
(Page 6 of 7)

(Doors A55, A57, C20, C26, A101, and A105)

Mechanical Parts

		Allowable Stresses (psi)	
No.	Load Combinations	Tension and Compression[2]	Shear
	<u>Doors Open</u>		
I	DL + Load from door closers	$F_u/5$	$2\ F_u/15$
II	DL + OBE + Load from door closers	$0.60\ F_y$	$0.40\ F_y$
	DL + SSE + Load from door closers	$0.90\ F_y$	$0.60\ F_y$
	<u>Doors Closed</u>		
III[3]	DL + CCWS flood + 3-psi pressure (bidirectional where applicable)	$0.90\ F_y$	$0.60\ F_y$
IV[4]	DL + OBE + Pressure from valve room	$0.60\ F_y$	$0.40\ F_y$
	DL + SSE + Pressure from valve room	$0.90\ F_y$	$0.60\ F_y$

1. Thermal Load effects are insignificant and hence need not be considered in the design of doors.
2. The values indicated for the allowable compression stresses is the maximum value permitted, when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_y when buckling controls.

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] \text{ when } \frac{Kl}{r} \le C_c \qquad 7$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \frac{Kl}{r} > C_c \qquad 8$$

3. The CCWS flood condition does not apply to doors A101 and A105, and differential pressure load due to tornado need not be considered simultaneously with seismic load.

4 Applies to doors A101 and A105 only.

Table 3.8.4-7 Pressure Confining Personnel Doors Loads, Loading Combinations, and Allowable Stresses[1] (All Doors as shown in Table 3.8.4-7a except A55, A57, C20, C26, A101, A105, A216, and A217)
(Page 7 of 7)

(Doors A216 and A217)

Structural Parts

		Allowable Stresses (psi)		
No.	Load Combinations	Tension	Compression[2]	Shear
I	DL + P	$0.50\ F_y$	$0.47\ F_y$	$0.33\ F_y$
II	DL + P + OBE	$0.60\ F_y$	$0.60\ F_y$	$0.40\ F_y$
III	DL + P + SSE	$0.90\ F_y$	$0.90\ F_y$	$0.60\ F_y$

Mechanical Parts

		Allowable Stresses (psi)	
No.	Load Combinations	Tension and Compression[2]	Shear
I	DL + P	$F_u/5$	$2\ F_u/15$
II	DL + P + OBE	$0.60\ F_y$	$0.40\ F_y$
III	DL + P + SSE	$0.90\ F_y$	$0.60\ F_y$

DL - Stresses generated by dead loads and door closer loads.

P - Stresses generated by a pressure differential of 1/2 inch of water acting to open doors.

1. Thermal Load effects are insignificant and hence need not be considered in the design of doors.

2. The values indicated for the allowable compression stresses is the maximum value permitted, when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_y when buckling controls.

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] \text{ when } \frac{Kl}{r} \le C_c \qquad 9$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \frac{Kl}{r} > C_c \qquad 10$$

Table 3.8.4-7a LIST OF PERSONNEL ACCESS DOORS IN AUXILIARY / CONTROL BUILDING (Page 1 of 1)

A55, A57, A64, A65, A78, A94, A95, A96, A101, A105, A113, A114, A115, A123, A132, A152, A153, A154, A155, A156, A157, A158, A159, A161, A164, A165, A173, A184, A191, A214, A215, A216, A217

C20, C26, C29, C34, C36, C54

A111, A117, A125, A130, A151, A160, A162, A183, A192, A206, A207, A208, A209, A212, A213

C37, C49, C50, C53, C60

DE1, DE4, DE5

A56, A60, A122, A133

Table 3.8.4-7b ABSCE AIR BOUNDARY DOORS (Page 1 of 1)

The following pairs of doors function as ABSCE air lock boundary doors. These doors become part of the ABSCE boundary when the associated airlock boundary door (or damper) is open.

Primary ABSCE Boundary Door	Associated (Extended) Airlock Boundary Door
A55	A60
A57	A56
A65	A64 (See Note 1.)
A112 (Railroad access door)	A111, A113, A114
A123	A122
A125	A101
DE1	DE4, DE5 (See Note 2.)
A130	A105
A132	A133
A206	A207
A208	A209
A214	A192
A215	A183

Notes:

1. Doors A64 andA6 are intelocked to protect the EGTS (Annulus) pressure boundary. A64 is not an ABSCE boundary component.

2. These doors are not required to be interlocked because of the infrequent egress/ingress through them during an ABI.

Table 3.8.4-8 Intake Pumping Station Loading Cases, Allowable Stresses, Factors, Factors of Safety, And Material Properties (Page 1 of 2)

Case	Description	WSD Allowable Stresses	Calculated Factors of Safety	
			Overturning	Sliding
I	Reservoir level at El. 690.0, operating loads, earthfill, one pump well unwatered (1/2 of structure)	Normal per ACI	2.27	3.52
II	Reservoir level at E1. 713.0, operating loads, earthfill, Operating Bases Earthquake (OBE)	Normal Concrete fs = 0.5 fy per ACI	1.33	1.39
IIa	Reservoir level at E1. 732.1 (assuming upstream dam failure), operating loads, earthfill, OBE, both pump wells full	1.35 x Normal per ACI	1.36	2.18
III	Reservoir level at E1. 724.4, operating loads, earthfill, both pump wells full	1.35 x Normal per ACI	N/A	N/A
IV	Reservoir level at E1. 675.0, operating loads, earthfill, both pump wells full, tornado	1.67 x Normal per ACI fs = 0.9 fy	2.84	2.84
V	Reservoir level at E1. 695.0 (25-year flood), operating loads, earthfill, both pump wells full, safe shutdown earthquake (SSE)	1.67 x Normal per ACI fs = 0.9 fy	1.23	1.10
Case	Description	WSD Allowable Stresses	Factors of Safety	
			Overturning	Sliding
Va	Reservoir level at El. 732.8 operating loads earthfill, both pump wells full, SSE	1.67 x Normal per ACI $f_s = 0.9\ F_y$	1.11	1.10
VI	Construction condition-dead load of structure, no equipment, earthfill, no ground water	1.33 x Normal per ACI	14.5	4.0
VII	Reservoir level at El. 742.2 (738.8 PMF + 3.4 ft wave run up) operating loads, both pump wells full. (The electrical equipment room begins to flood when the reservoir level exceeds El. 728)	1.67 x Normal per ACI $f_s = 0.9\ f_y$	1.519	4.378

ACI = Allowable stresses per ACI 318-71 Edition (working stress design)

Table 3.8.4-8 Intake Pumping Station Loading Cases, Allowable Stresses, Factors, Factors of Safety, And Material Properties (Page 2 of 2)

Material Properties Concrete: $f'c$ = 3000 psi w = 145 pcf Reinforcing Steel: f_y = 60,000 psi (ASTM A615, grade 60)

Table 3.8.4-9 Concrete Retaining Walls Loading Cases Allowable Stresses, Factors of Safety, and Material Properties

Case	Description-Reservoir	WSD Allowable Stresses
I	Normal Operating condition - level at El. 675.0	Normal per ACI 318-71
IA	Same as (I) + Operating Basis	fc = .45 f'c fs = .5 fy
IB	Same as (I) + Safe Shutdown Earthquake	fc = .75 f'c fs = .67 fy
II	Construction condition - earth pressure, 200 psf surcharge	fc = .5 f'c fs = .5 fy
Material Properties		
Concrete: f'c = 3000 psi w = 145 pcf		
Reinforcing Steel: fy = 60,000 psi (ASTM A615, grade 60)		

Table 3.8.4-10 Sheet Pile Retaining Wall Design Loadings, Allowable Stresses, Material Properties

Case	Description	WSD Allowable Stresses	Allowable Stresses ASTM A36	Allowable Stresses ASTM A328
I	Earth pressure plus 200 psf surcharge	Normal per ACI 318-71	0.8* (AISC allowable)	18,000 psi
II	Same as I plus Operating Basis Earthquake	Normal per ACI fs = 0.5 f_y	0.8* (AISC allowable)	18,000 psi
III	Same as I plus Safe Shutdown Earthquake	1.67 x Normal per ACI fs = 0.9 f_y	0.8* x F_y	28,000 psi

*Reduced allowable stresses are used to provide corrosion allowance.

Material Properties

ASTM A36 Steel: F_y = 36,000 psi

Table 3.8.4-11 Traveling Water Screens (Intake Pumping Station) Load Combinations And Allowable Stresses (Page 1 of 2)

Structural Parts			
		Allowable Stresses (psi)	
No.	Load Combinations	Tension and Compression(2)	Shear
I	Dead Live with water at El. 683.0 and 2' 6" head loss Impact from live load	0.56 F_y	0.38 F_y
II	For headframe only Dead Live with water at El. 683.0 and 2' 6" head loss Impact from live load Snow and ice	0.56 F_y	0.38 F_y
III	Dead Live with water at El. 713.0 and 2' 6" head loss OBE(1)	0.56 F_y	0.38 F_y
IV	Dead Live with water at El. 695.0 and 2' 6" head loss SSE(1)	0.9 F_y	0.6 F_y
V	Dead Live with water at El. 736.9 and 5' 0" head loss Impact	0.9 F_y	0.6 F_y
VI	Dead Stall at 300% capacity	0.9 F_y	0.6 F_y
Other Parts			
I	Dead Live with water at El. 683.0 and 2' 6" head loss Impact from live load	$\frac{\text{Ult}}{5}$	$\frac{2 \times \text{Ult}}{15}$
II	For headframe only Dead Live with water at El. 683.0 and 2' 6" head loss Impact from live load Snow and ice	$\frac{\text{Ult}}{5}$	$\frac{2 \times \text{Ult}}{15}$

Table 3.8.4-11 Traveling Water Screens (Intake Pumping Station) Load Combinations And Allowable Stresses (Page 2 of 2)

III	Dead Live with water at El. 713.0 and 2' 6" head loss OBE(1)	Ult 5	2 x Ult 15
IV	Dead Live with water at El. 695.0 and 2' 6" head loss SSE(1)	0.9 F_y	2/3 x 0.9 F_y
V	Dead Live with water at El. 736.9 and 5' 0" head loss Impact	0.9 F_y	2/3 x 0.9 F_y
VI	Dead Stall at 300% capacity	0.9 F_y	2/3 x 0.9 F_y

Notes:

(1) Acts in one horizontal direction only at any given time and acts in vertical and horizontal directions simultaneously.

(2) The value given for allowable compression stress is the maximum value permitted when buckling does not control. The critical buckling stress. F_{cr}, shall be used in place of F_y when buckling controls.

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] \text{ when } \frac{Kl}{r} \le C_c \qquad 1$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \frac{Kl}{r} > C_c \qquad 2$$

Table 3.8.4-12 Diesel Generator Building Loads, Loading Combinations, Allowable Stresses, And Material Properties

Loads

D = Dead load of structure including the weight of the diesel generators

L = Live load -200 psf or equipment load in mechanical areas
-300 psf in electrical areas
- 20 psf on roof

L_c = Construction live load (50 psf on roof)

E = Operational basis earthquake (OBE)

E' = Safe shutdown earthquake (SSE)

WT = Tornado-generated missiles[(1)]

Load Combinations

Case	Description[(3)]	Allowable Stresses
I	D+L	Normal stresses[(2)]
II	$D+L_c$	Normal stresses[(2)] + 33%
III	D+L+E	$f_c = 0.45\ f'_c$ $f_s = 0.50\ f_y$
IV	D+L+E'	$f_c = 0.75\ f'_c$ $f_s = 0.90\ f_y$
V	$D+L+W_t$	$f_c = 0.75\ f'_c$ $f_s = 0.90\ f_y$

Material Porperties

Concrete: f'c = 3000 psi
w = 145 pcf

Reinforcing Steel: f_y = 60,000 psi (ASTM A615, grade 60)

Notes:

(1) The exterior walls and roof are designed to resist missile spectrum A of Table 3.5-7. The precast concrete bulkheads placed in front of the equipment doors are designed to withstand tornado-generated missiles of missile spectrum B in Table 3.5-8 (see discussion in Section 3.8.4.1.2).

(2) Normal stresses are given for working stress design in ACI Code 318-63 or ACI code 318-71 (See Section 3.8.4.2.2).

(3) Both conditions of L, having its full value or being completely absent, are checked.

Table 3.8.4-13 Diesel Generator Building Doors And Bulkheads Loads, Loading Combinations, And Allowable Stresses (Page 1 of 2)

Structural Parts			
		Allowable Stresses(psi)	
No.	Load Combinations	Tension and Compression(4)	Shear
	Door Open or Closed		
I	Dead load	$0.6\ F_y$	$0.4\ F_y$
	Door Closed		
II	Dead Load plus OBE	$0.6\ F_y$	$0.4\ F_y$
III	Dead load plus SSE	$0.9\ F_y$	$0.6\ F_y$
Mechanical Parts			
		Allowable Stresses (psi)	
No.	Load Combinations	Tension and Compression(4)	Shear
	Door Open or Closed		
I	Dead load	$\frac{\text{Ult}}{5}$	$\frac{2 \times \text{Ult}}{15}$
	Door Closed		
II	Dead load plus OBE	$0.6\ F_y$	$0.4\ F_y$
III	Dead load plus SSE	$0.9\ F_y$	$0.6\ F_y$
Concrete Bulkheads			
		Allowable Stresses	
No.	Load Combinations	Concrete	Reinforcing Steel
I	Dead Load	1.0 ACI 318	1.0 ACI 318
II	Dead Load plus Wind(2) or OBE	1.0 ACI 318	$0.5\ F_y$
III	Dead Load plus SSE	1.67 ACI 318	$0.9\ F_y$
IV	Dead Load plus Tornado(2)	(3)	(3)

Table 3.8.4-13 Diesel Generator Building Doors And Bulkheads Loads, Loading Combinations, And Allowable Stresses (Page 2 of 2)

Notes:

(1) Acts in one horizontal direction only at any given time and acts in vertical and horizontal directions simultaneously.

(2) The steel doors and steel bulkheads are protected from wind, snow, ice, rain, tornado, and wind and tornado missiles by precast concrete bulkheads as discussed in Section 3.8.4.1.2.

(3) The structure may be allowed to yield for load combination IV when considering impactive loads from missiles.

(4) The value indicated for the allowable compression stresses is the maximum value permitted when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of Fy when buckling controls.

$$F_{cr} = F_Y\left[1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^2}\right] \text{ when } \frac{Kl}{r} \le C_c \qquad 1$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \text{ when } \frac{Kl}{r} > C_c \qquad 2$$

Table 3.8.4-14 Deleted by Amendment 39

Table 3.8.4-15 Primary and Refueling Water Pipe Tunnels Loads, Load Combinations, Allowable Stresses, and Material Properties

Loads

D = Dead load of structure plus any permanent load contributing stress, such as vertical soil pressure, hydrostatic pressure from ground water Elevation 710, walls; Elevation 726, uplift on slab.

L = Surcharge loading from trucks and other equipment operating above tunnel.

D' = Hydrostatic pressure from reservoir Elevation 729, maximum level buildings remain unflooded.

R_o = Temperature effects on pipe restraints inside tunnel.

E = Operating basis earthquake.

E' = Safe shutdown earthquake.

Y_r = Pipe restraint load due to main steam pipe rupture.

W_t = Tornado missile striking earth above top slab of tunnel.

Case	Load Combination	Allowable Stresses
I	D+L (Construction Condition)	f_c = .45 f'_c (ACI 318-71) f_s = .5 f_y
II	D+L+R_o+E	f_c = .45 f'_c f_s = .5 f_y
III	D'+L+R_o	f_c = .75 f'_c f_s = .9 f_y
IV	D+L+R_o+E'	f_c = .75 f'_c f_s = .9 f_y
V	D+L+R_o+E'+Y_r	f_c = .75 f'_c f_s = .9 f_y
VI	D+L+R_o+W_t	f_c = .75 f'_c f_s = .9 f_y

Material Properties

Concrete: f'c = 3000 and 4000 psi
w = 145 pcf

Reinforcing Steel: f_y = 60,000 psi (ASTM A615, grade 60)

Table 3.8.4-16 Class 1e Electric Systems Structures Loads, Load Combinations, Allowable Stresses, and Material Properties (Page 1 of 2)

Structures - Manholes	
Design Cases	Allowable Stresses
I. SEISMIC OPERATING	
a. Dry earthfill plus 1/2 safe shutdown earthquake (1/2 SSE)	$f_c = 0.45 f'_c$ $f_s = 0.50 f_y$
b. Earthfill with ground water at elevation 726.0 or finished grade, whichever is lower, plus 1/2 SSE	$f_c = 0.45 f'_c$ $f_s = 0.50 f_y$
II. FULL SEISMIC	
a. Dry earthfill plus SSE	$f_c = 0.75 f'_c$ $f_s = 0.90 f_y$
b. Earthfill with ground water at elevation 726.0 or finished grade, whichever is lower, plus SSE	$f_c = 0.75 f'_c$ $f_s = 0.90 f_y$
III. NORMAL OPERATING	
Earthfill with ground water at elevation 726.0 or finished grade, whichever is lower, plus 200 psf surcharge (or concentrated surcharge where applicable).	$f_c = 0.45 f'_c$ $f_s = 0.40 f_y$
IV. TEST	
Dry earthfill, one compartment of manhole filled with water, water surface elevation 743.5	$f_c = 0.50 f'_c$ $f_s = 0.60 f_y$
V. FLOOD	
Earthfill plus probable maximum flood. No water inside structure	$f_c = 0.75 f'_c$ $f_s = 0.90 f_y$
VI. TORNADO LOADING	
$D+L+W_T$ (Vertical Missile)	See FSAR Section 3.5.
$D+L+W_T$ (Differential Pressure)	$f_c = 0.75 f'_c$ $f_s = 0.90 f_y$

Table 3.8.4-16 Class 1e Electric Systems Structures Loads, Load Combinations, Allowable Stresses, and Material Properties (Page 2 of 2)

Structures - Duct Banks	
Design Cases	Required Strength
I. SEISMIC OPERATING	
1/2 SSE (E)	U = 1.4D + 1.7L + 1.9E
II. FULL SEISMIC	
SSE (E')	U = D + L + E'
III. TORNADO GENERATED MISSILES	U = D +L + W_T
IV. SURCHARGE LOAD L (CRANE OR TRAIN)	U = 1.7L + 1.4D
(D = Dead Load)	
Material Properties	
Concrete: f'c = 3000 psi w = 145 pcf	
Reinforcing Steel: f_y = 60,000 psi (ASTM A615, grade 60)	

Table 3.8.4-17 North Steam Valve Room Loading Combinations And Allowable Stresses

Load Combinations	WSD Allowable Stresses	USD Load Factors
Case I = D+L	f_c =.45 f'_c f_s =.40 f_y	1.4D+1.7L
Case II = D+L+E	f_c =.45 f'_c f_s =.50 f_y	1.4D+1.7L+1.9E
Case III = D+L+E'	*f_c =.75 f'_c f_s =.90 f_y	1.0(D+L+E')
Case IV = D+L+W_t	*f_c =.75 f'_c f_s =.90 f_y	1.0(D+L+W_t)
Case V = D+L+P_a	*f_c =.75 f'_c f_s =.90 f_y	1.0D+1.0L+1.5 P_a
Case VI = D+L+P_a+Y_r+Y_j+Y_m+E	*f_c =.75 f'_c f_s =.90 f_y	1.0D+1.0L+1.25 P_a + 1.0(Y_r+Y_j+Y_m) +1.25E
Case VII = D+L+P_a+Y_r+Y_j+Y_m+E'	*f_c =.75 f'_c f_s =.90 f_y	1.0(D+L+P_a+Y_r+Y_j+Y_m+E')

*Concrete stresses other than flexure = 1.67 x normal

Loads
The following terms are used in the load combination equations:

D - Dead load of structure and any permanent equipment loads or hydrostatic loads

L - Live loads, including any moveable equipment loads such as soil pressure

E - Operational Basis Earthquake

E' - Safe Shutdown Earthquake

W_t - Tornado, including wind pressure with missiles

P_a - Pressure from postulated main steam pipe break

Y_r - Pipe anchor force due to postulated pipe break

Y_j - Jet force due to postulated pipe break

Y_m - Missile impact force due to postulated pipe break

Table 3.8.4-18 North Steam Valve Room Structural Steel Loading Combinations and Allowable Stresses For Structural Steel

(Page 1 of 3)

Loading Combinations	Tension Net Section	Shear on Gross Section	Compression on Gross Section	Bending	Concrete Bearing
Case I DL+LL+OBE	0.60 FY	0.40 FY	See Note 1	0.66 FY to 0.60 FY	0.25 f^1c
Case II	0.90 FY	$\frac{0.9F_y}{\sqrt{3}}$	See Note 2	0.90 FY	0.595 f^1c
DL+LL+SSE			See Note 3		See Note 3

Table 3.8.4-18 North Steam Valve Room Structural Steel Loading Combinations and Allowable Stresses For Structural Steel

(Page 2 of 3)

Note 1 - Varies with slenderness ratio; see AISC "Maunal of Steel Constructions," 7th Edition, Table 1-36, page 5-84.
Note 2 - Varies with slenderness ratio:

Main and secondary members where $KL/r \leq C_C$:

$$F_a = 0.9F_Y\left[1 - \frac{(KL/r)^2}{2C_c^2}\right] \qquad (A)$$

Main members where $C_C < KL/r < 200$:

$$F_a = \frac{0.9\pi^2 E}{(KL/r)^2} \qquad (B)$$

Secondary members where $120 < KL/r \leq 200$:

$$F_{as} = \frac{F_a\,[\text{ by Formula (A) or (B)}]}{1.6 - L/200r}$$

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$$

where:

E = Modulus elasticity of steel (29,000 kips per square inch)
Fa = Axial compressive stress permitted in the absence of bending moment
F_{as} = Axial compressive stress permitted in the absence of bending moment , for bracing and other secondary members.
F_Y = Specified minimum yield stress of material (kips per square inch)
f'c = Compressive strength of concrete (kips per square inch)
K = Effective length factor
L = Actual unbraced length (inches)
r = Governing radius of gyration (inches)

Material Properties

Steel: C_C = 126.1

E = 29,000,000 psi

F_Y = 36,000 psi

Table 3.8.4-18 North Steam Valve Room Structural Steel Loading Combinations and Allowable Stresses For Structural Steel (Page 3 of 3)

Note 3- When the supporting surface is wider on all sides than the loaded area, the permissible bearing stress on the loaded area may be multiplied by $\sqrt{A_2 / A_1}$ 1 but not more than 2.

Where: A1 = Loaded area (square inches)

A2 = Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area (square inches).

Table 3.8.4-19 ERCW Structures Loads, Load Combinations, Allowable Stresses, And Material Properties

LOADS

D = Dead Load
L = Live Loads (loads which vary in intensity and occurrence)
E = Operating Basis earthquake (one-half safe shutdown earthquake)
E' = Safe Shutdown Earthquake
W_t = Tornado Loading (Wind and Missiles and pressure differential as applicable)
W = Loads Generated by the Design Wind for the Plant
L_c = Construction Live Load

Structure - Slab and Beams Supporting ERCW Pipes

LOAD COMBINATIONS

Design Cases		ASTM A36 Allowable Stresses
Structure - Slabs Supporting ERCW Pipes		
I	u = 1.4 D + 1.7 L	0.6 yield
II	u = 1.4 D + 1.7 L + 1.9 E	0.6 yield
III	u = D+L+E'	0.9 yield
Structure - ERCW Standpipe Structure		
I	u = 1.4 D + 1.7 L + 1.9 E	
II	u = D+L+E'	
III	u = D+W_t	
IV	u = 1.4D + 1.7L + 1.7W	
Structure - ERCW Discharge Overflow Structure		
I	u = 1.4 D + 1.7 L	
II	u = 1.4 D + 1.7 L + 1.9 E	
III	u = D+L+E'	
IV	u = D+W_t	
V_A	u = 1.4 D + 1.7 L + 1.7 W	
V_B	u = 1.2 D + 1.7 W	
VI	u = 1.4 D + 1.4 Lc	
Structure - ERCW Valve Covers		
I	U = 1.4 D + 1.7 L	
II	U = 1.4 D + 1.7 L + 1.9 E	
III	U = D+L+E'	
IV	U = D+W_t	

MATERIAL PROPERTIES

Concrete: F'_c = 3000 or 4000 psi
w = 145 pcf

Reinforcing Steel: f_y = 60 ksi (ASTM A615, Grade 60)

Table 3.8.4-20 Refueling Water Storage Tank Foundation Loads, Load Combinations And Material Properties

LOADS

D = Dead Load
L = Live Load Including Soil Pressure
E = Operating Basis Earthquake
W = Design Wind
E' = Safe Shutdown Earthquake
W_t = Tornado Loading (Wind and Missile)
Y_j = Jet Impingement Associated with High-Energy Pipe Break
Y_m = Missile Impact Generated by High-Energy Pipe Break

LOAD COMBINATIONS

Design Cases

1	U = 1.4 D + 1.7 L
2	U = 1.4 D + 1.7 L + 1.9 E
3	U = 1.4 D + 1.7 L + 1.7 W
4	U = 1.2 D + 1.9 E
5	U = 1.2 D + 1.7 W
6	U = D+L+E'
7	U = $D+L+W_t$
8	U = D+L
9	U = $D+L+Y_j+Y_m+1.25\,E$
10	U = $D+L+Y_j+Y_m+E'$

MATERIAL PROPERTIES

Concrete: f'_c = 3000 psi
w = 145 pcf

Reinforcing Steel: f_y = 60 ksi (ASTM A615, Grade 60)

Table 3.8.4-21 Spent Fuel Pool Gates Loads, Loading Combinations, and Allowable Stresses

		Allowable Stresses lb/in^2 Loading Conditions	
No.	Load Combinations[1]	Bending	Shear
1	D+L	0.6 F_y	0.4 F_y
2	D+L+OBE	0.6 F_y	0.4 F_y
3	D+L+W	0.6 F_y	0.4 F_y
4	$D+L+T_o+R_o+SSE$	0.9 F_y	0.6 F_y
5	$D+L+T_o+R_o+W_t$	0.9 F_y	0.6 F_y
6	$D+L+T_a+R_a+P_a$	0.9 F_y	0.6 F_y

Notes:

(1) T_o, R_o, T_a, R_a, P_a = 0

Table 3.8.4-22 Additional Diesel Generator Building Loads, Loading Combinations, Definitions Of Load Terms (Page 1 of 5)

Definition of Load Terms

The following terms are used in the load combination equations for the Additional Diesel Generator Building:

Normal loads, which are those loads to be encountered during normal plant operation and shutdown, include:

D - Dead loads or their related internal moments and forces including any permanent equipment loads; all hydrostatic loads; and earth loads applied to horizontal surfaces.

L - Live loads or their related internal moments and forces including any movable equipment loads and other loads which vary with intensity and occurrence, such as lateral soil pressures.

200 lb/ft^2 or equipment load (floors)

50 lb/ft^2 on roof

L_c - Construction live load = 50 lb/ft^2

T_o - Thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

R_o - Pipe reactions during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

Severe environmental loads include:

E - Loads generated by the OBE.

W - Loads generated by the design wind specified for the plant. See Section 3.3.

Table 3.8.4-22 Additional Diesel Generator Building Loads, Loading Combinations, Definitions Of Load Terms (Page 2 of 5)

Extreme environmental loads include:

E' - Load generated by the SSE.

W_t - Loads generated by the design tornado specified for the plant. Tornado loads include loads due to the tornado wind pressure, the tornado-created differential pressure, and to tornado-generated missiles.

Where:
$W_t = W_w$ (tornado wind, see Section 3.3).
$W_t = Wp$ (tornado pressure differential, see Section 3.3)
$W_t = W_m$ (tornado missile, see Table 3.5-17)
$W_t = W_w + .5\ W_p$
$W_t = W_w + W_m$
$W_t = W_w + .5\ W_p + W_m$

Abnormal Loads

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

T_a = Thermal loads under themal conditions generated by the postulated break and including T_o.

R_a = Pipe reaction under thermal conditions generated by the postulated break and including R_o.

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

Y_j = Jet impingment equivalent static load on a structure generated by the postulated break, and including an approriate dynamic load factor to account for the dynamic nature of the load.

Y_m = Missile impact equivalent static load on a structure generated by or during the postulated break, as from pipe whipping, and including an appropriate dynamic load factor to account for the dynamic nature of the load.

Other Loads:

F = Hydrostatic load from design basis flood.

Table 3.8.4-22 Additional Diesel Generator Building Loads, Loading Combinations, Definitions Of Load Terms (Page 3 of 5)

Fa = Flood load generated by a postulated pipe break.

Load Combinations

Concrete Structures

a. For service load conditions, the strength design method was used and the following load combinations were considered.

1. U = 1.4 D + 1.7 L
2. U = 1.4 D + 1.7 L + 1.9 E
3. U = 1.4 D + 1.7 L + 1.7 W

If thermal stresses due to T_o and R_o are present, the following combinations were also considered.

1a. U = (0.75) (1.4 D + 1.7 L + 1.7 T_o + 1.7 R_o)

2a. U = (0.75) (1.4 D + 1.7 L + 1.9 E + 1.7 T_o + 1.7 R_o)

3a. U = (0.75) (1.4 D + 1.7 L + 1.7 W + 1.7 T_o + 1.7 R_o)

Both cases of L having its full value or being completely absent were checked. In addition, the following combinations were considered.

2a'. U = 1.2 D + 1.9 E

3a'. U = 1.2 D + 1.7 W

Table 3.8.4-22 Additional Diesel Generator Building Loads, Loading Combinations, Definitions Of Load Terms (Page 4 of 5)

Where D or L reduce the effect of the loads given above, the corresponding coefficients were taken as 0.90 for D and zero for L. The vertical pressure of liquids was considered as dead load with due regard to variation in liquid depth.

b. For factored load conditions, which represent extreme environmental, abnormal, abnormal/severe environmental and abnormal/extreme environmental conditions, the strength design method was used and the following load combinations were considered.

4. $U = D + L + T_o + R_o + E'$
5. $U = D + L + T_o + R_o + W_t$
6. U = D + L + Ta + Ra + 1.5 Pa
7. U = D + L + Ta + Ra + 1.25Pa + 1.0 (Yr + Yj + Ym) + 1.25E
8. U = D + L + Ta + Ra + 1.00Pa + 1.0 (Yr + Yj + Ym) + 1.00E'

c. Other load conditions:

9. $U = 1.4\ D + 1.4\ L_c$
10. U = D + L + F
11. U = D + Fa

Steel Structures

a. For service load conditions, the elastic working stress design methods for Part 1 of the AISC specifications were used and the following load combinations were considered.

1. S = D + L
2. S = D + L + E
3. S = D + L + W

If thermal stresses due to T_o and R_o are present, the following combinations were also considered:

1a. $1.5S = D + l + T_o + R_o$
2a. $1.5S = D + L + T_o + R_o + E$
3a. $1.5S = D + L + T_o + R_o + W$

Both cases of L having its full value or being completely absent were checked.

Table 3.8.4-22 Additional Diesel Generator Building Loads, Loading Combinations, Definitions Of Load Terms (Page 5 of 5)

b. For factored load conditions, the following load combinations were considered:

4. $1.6S = D + L + T_o + R_o + E'$
5. $1.6S = D + L + T_o + R_o + W_t$
6. $1.6S = D + L + T_a + R_a + P_a$
7. $1.6S = D + L + T_a + R_a + P_a + 1.0\ (Yj + Yr + Ym) + E$
8. $1.7S = D + L + T_a + R_a + P_a + 1.0\ (Yj + Yr + Ym) + E'$
9. $1.6S = D + P_a$

S = AISC Allowables

In the above factored load combinations, thermal loads were neglected when it was shown that they are secondary and self-limiting in nature and where the material is ductile.

Uplift, Overturning, Sliding, and Flotation

Notation

The following terms were used in calculation of loads for uplift, overturning, sliding, and flotation:

D, E, W, E', W_t	= As defined on Sheet 1
H	= Lateral earth pressure
F'	= Buoyant force from design basis flood
F_b	= Buoyant force from normal ground water

Requirements of Category I Structures

The following minimum factors apply for the load conditions given.

	Minimum Factors of Safety		
Load Combination	Overturning	Sliding	Flotation
D + H + E	1.5	1.5	---
D + H + W	1.5	1.5	---
D + H + E'	1.1	1.1	---
D + H + W_t	1.1	1.1	---
D + F'	---	---	1.1
D + F_b	---	---	1.5

Table 3.8.4-23 Watertight Equipment Hatch Covers Loads, Loading Combinations, and Allowable Stresses (Page 1 of 2)

		Allowable Stresses (psi)		
No.	Load Combination	Tension	Compression*	Shear
Hatch Closed				
I	D + 200 lb/ft^2 live load	$0.6F_y$	$0.6F_y$	$0.4F_y$
II	$D + L_1$	$0.9F_y$	$0.9F_y$	$0.6F_y$
III	$D + L_2$	$0.9F_y$	$0.9F_y$	$0.6F_y$
IV	$D + L_1$ + OBE	$0.6F_y$	$0.6F_y$	$0.4F_y$
V	$D + L_1$ + SSE	$0.9F_y$	$0.9F_y$	$0.6F_y$

Where:

D - Dead Loads or their related internal moments and forces including permanent equipment
L_1 - Live Load due to flood to El 711.0
L_2 - Live Load due to pressure of 3 psi from below
OBE - Loads due to the operating basis earthquake
SSE - Loads due to the safe shutdown earthquake

*The value indicated for the allowable compression stresses is the maximum value permitted when buckling does not control. The critical buckling stress, F_{cr}, shall be used in place of F_y when buckling controls.

$$F_{cr} = F_Y\left(1 - \frac{\left(\frac{Kl}{r}\right)^2}{2C_c^{\,2}}\right) \quad \text{when} \quad \frac{Kl}{r} \le C_c \qquad 1$$

or

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} \quad \text{when} \quad \frac{Kl}{r} > C_c \qquad 2$$

Table 3.8.4-23 Watertight Equipment Hatch Covers Loads, Loading Combinations, and Allowable Stresses (Page 2 of 2)

Material	Serial Designation of the Specifications of the ASTM
Structural Steel	A36
Pipe	A53 or A103 Grade B
Headed Concrete Anchors	1/2" diam x 5-3/16", A108
Steel Screws	A193, Grade B
Seals	Natural or synthetic rubber or combination of natural and synthetic rubber (This is not an ASTM designation)

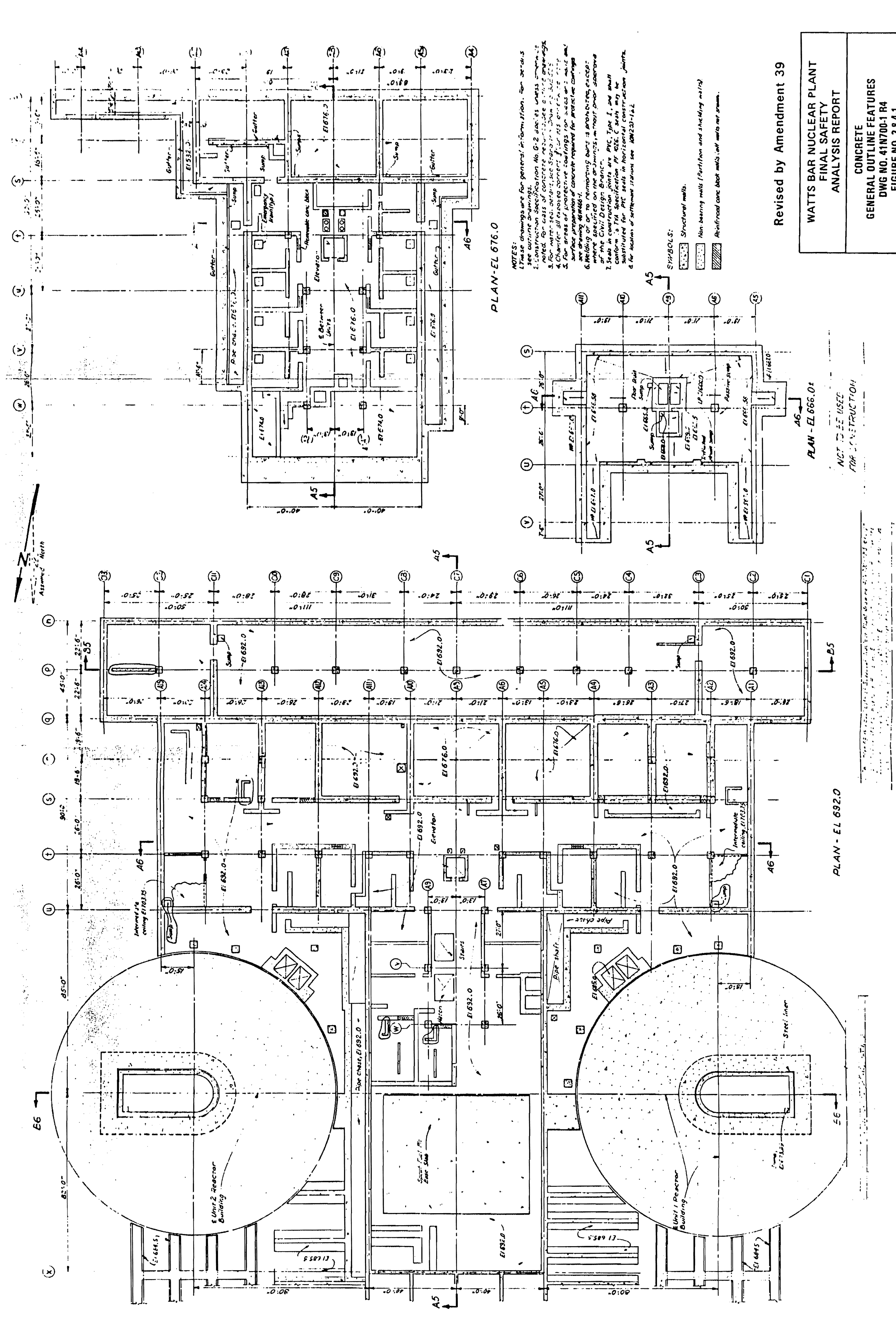


Figure 3.8.4-1 Concrete General Outline Features

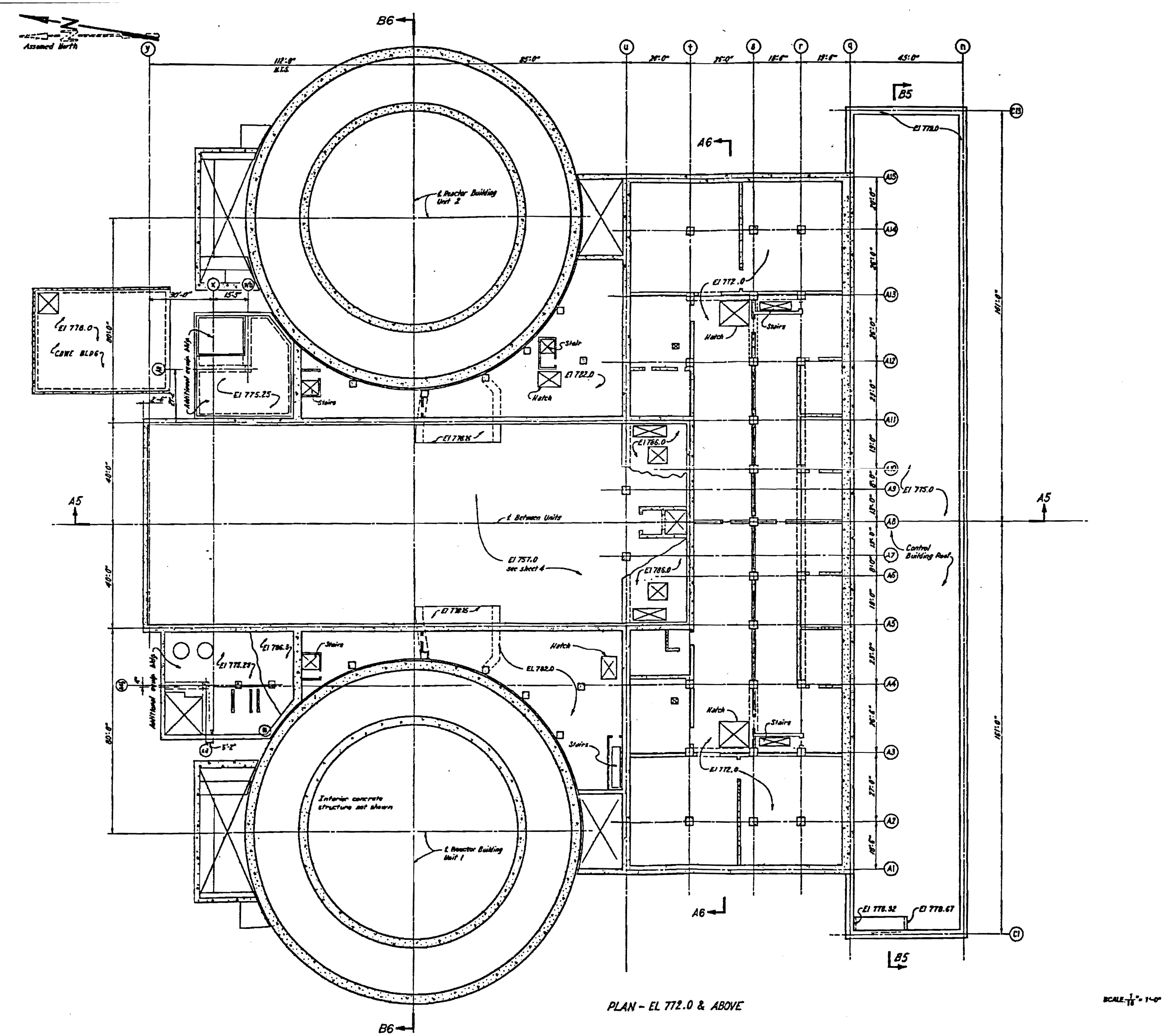
Figure 3.8.4-2 Reactor, Auxiliary & Control Buildings General Outline Features

Figure 3.8.4-3 Concrete General Outline Features

Figure 3.8.4-4 Reactor, Auxiliary & Control Buildings Concrete General Outline Features

Figure 3.8.4-5 Concrete General Outline Features

Figure 3.8.4-6 Reactor, Auxiliary & Control Buildings Concrete General Outline Features



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WATTS BAR
FINAL SAFETY
ANALYSIS REPORT

REACTOR, AUXILIARY &
CONTROL BUILDINGS
CONCRETE
GENERAL OUTLINE FEATURES
DWG NO. 41N700-7 R5
FIGURE 3.8.4-7

Figure 3.8.4-7 Reactor, Auxiliary & Control Buildings Concrete General Outline Features

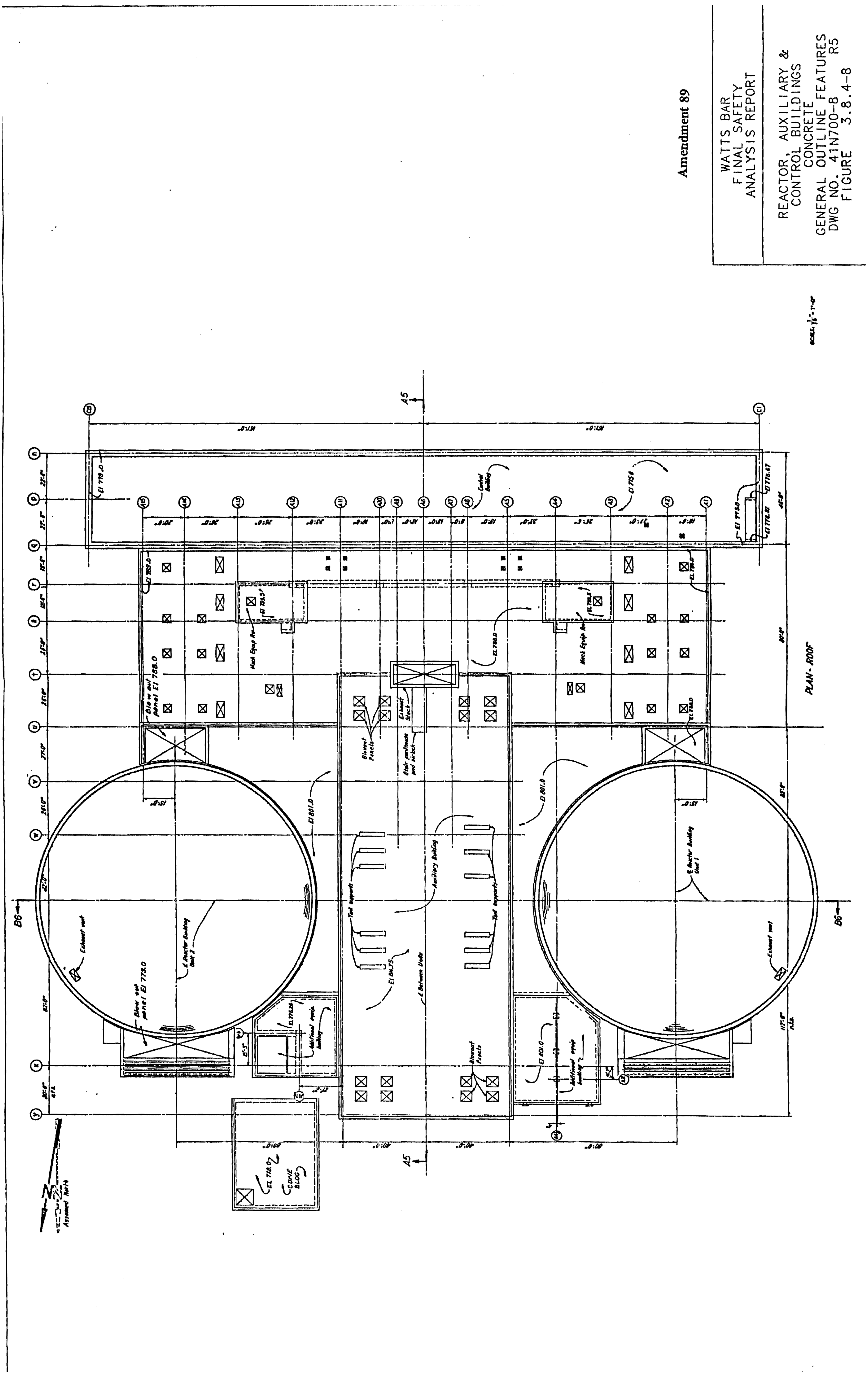


Figure 3.8.4-8 Reactor, Auxiliary & Control Buildings Concrete General Outline Features

Figure 3.8.4-9 Reactor, Auxiliary & Control Buildings Concrete Floor Design Data

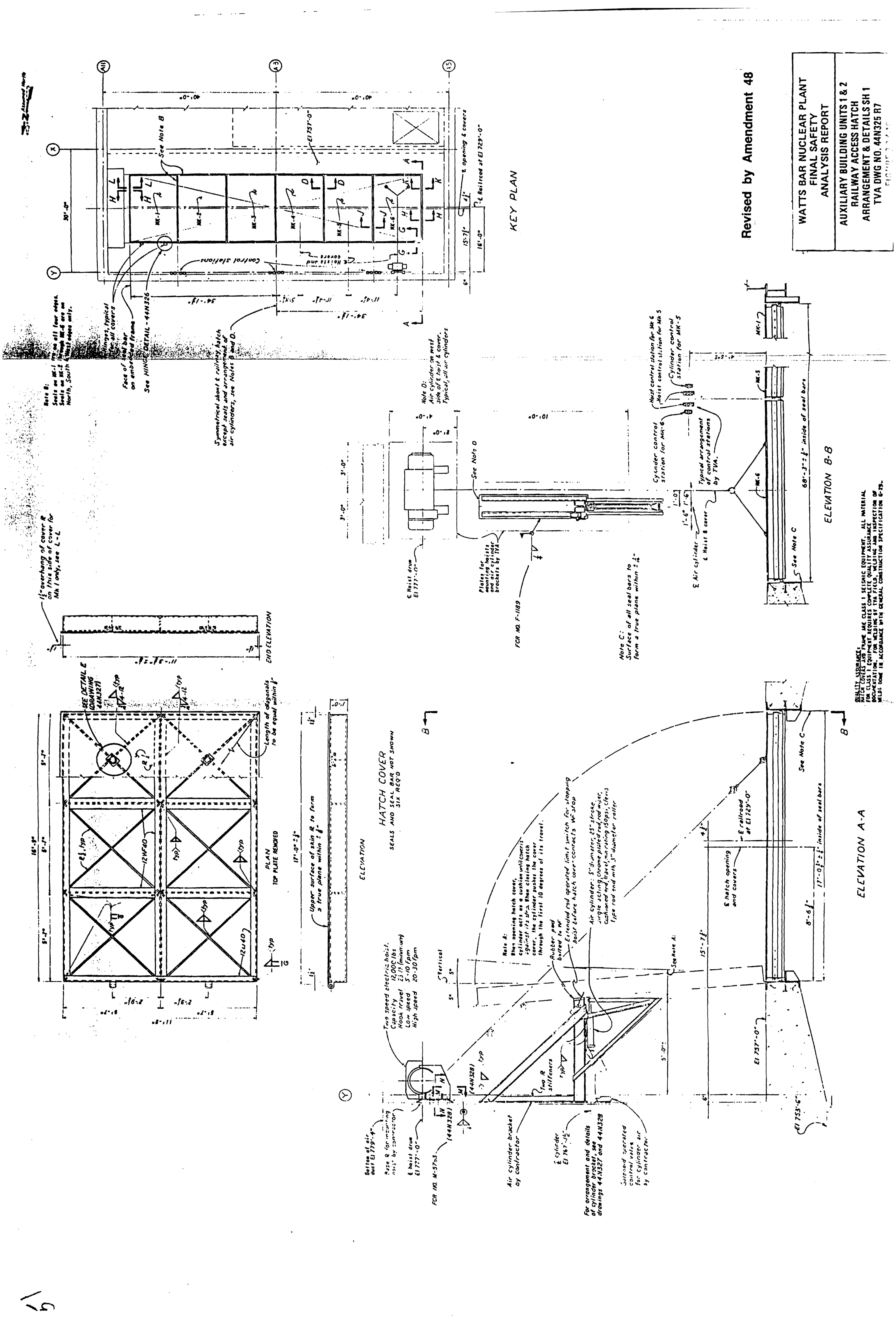


Figure 3.8.4-10 Auxiliary Buildings Units 1 & 2 Railway Access Hatch Arrangement & Details (Sheet 1)

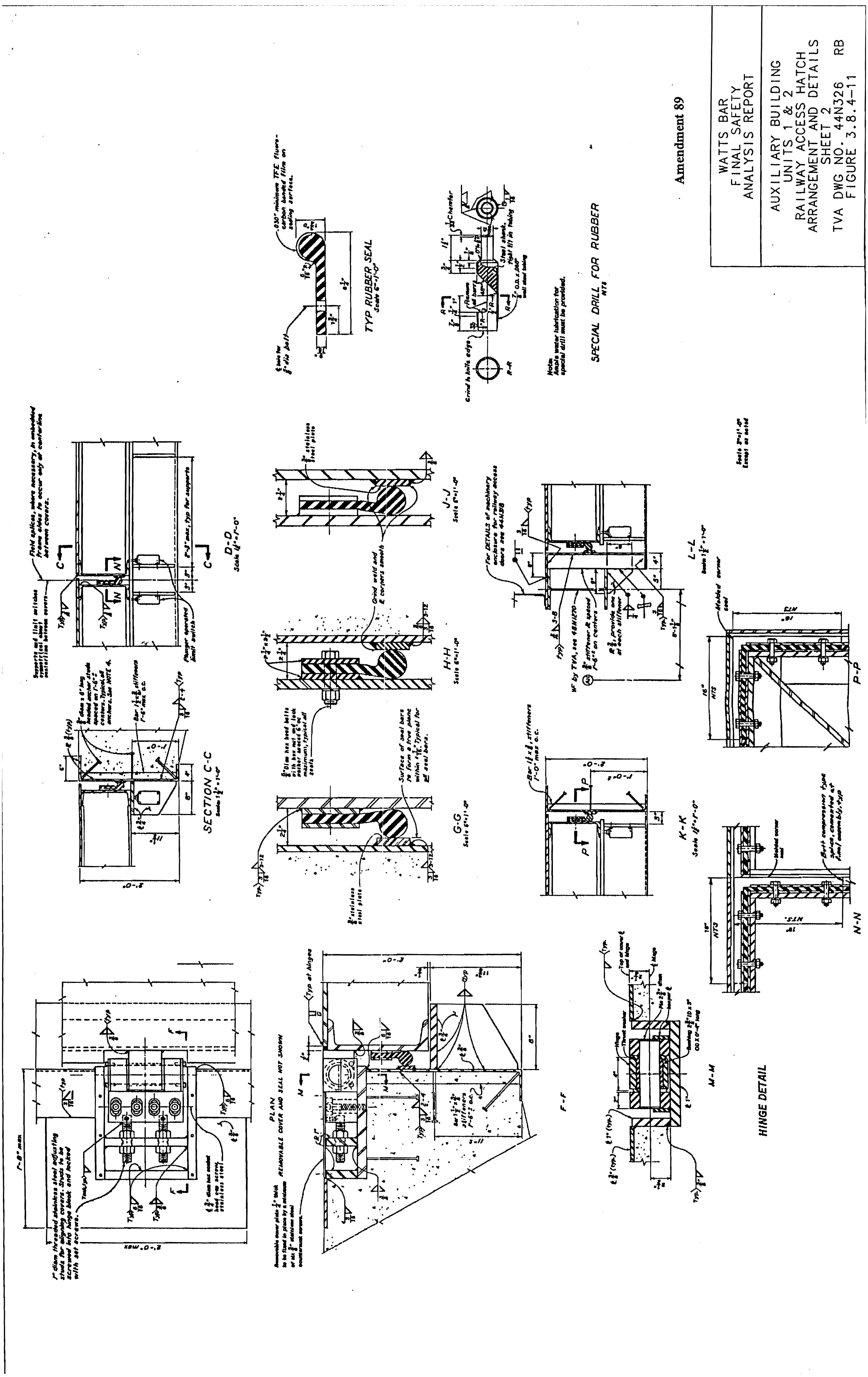


Figure 3.8.4-11 Auxiliary Building Units 1 & 2 Railway Access Hatch Arrangement and Details (Sheet 2)

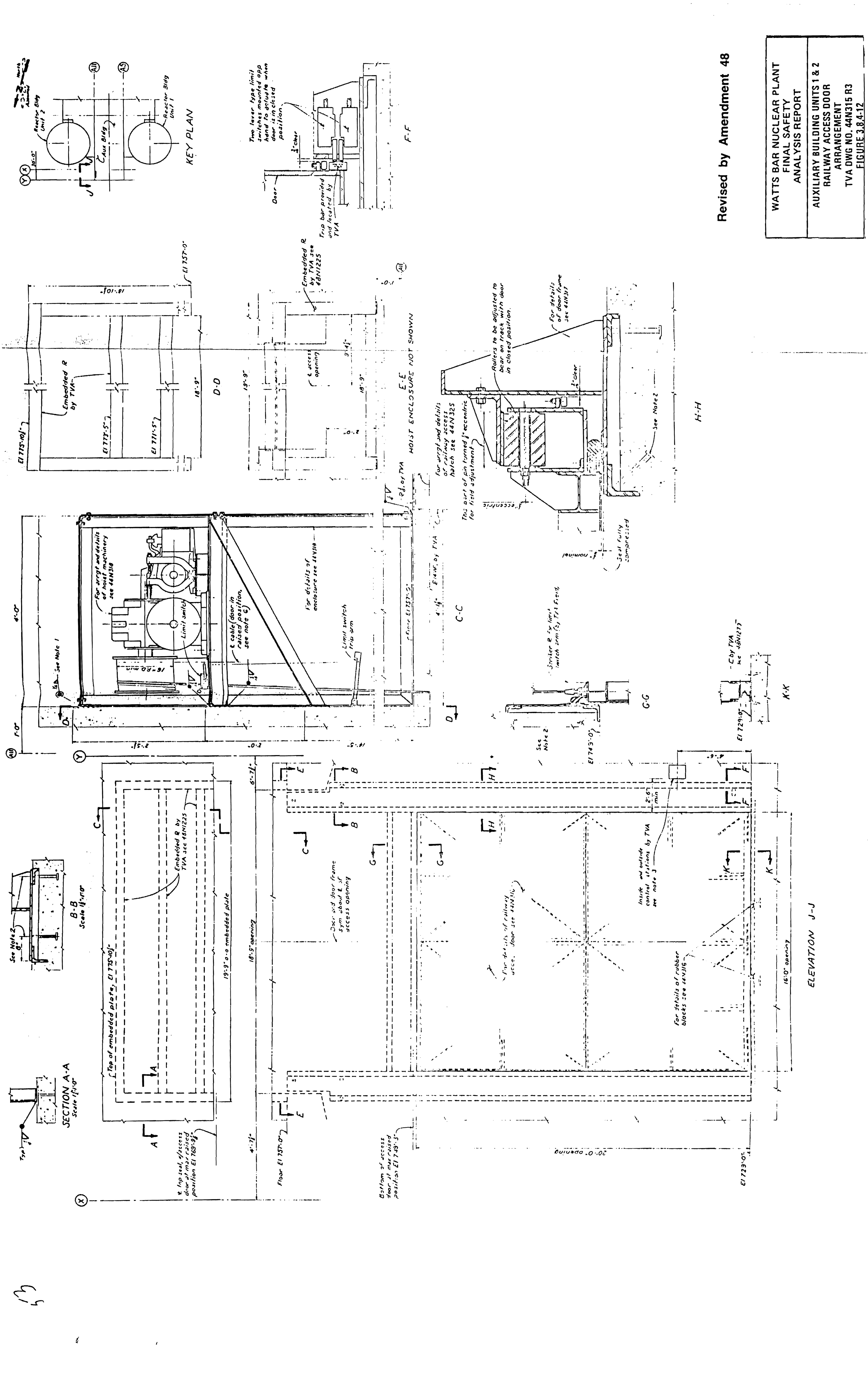


Figure 3.8.4-12 Auxiliary Building Units 1 & 2 Railway Access Door Arrangement

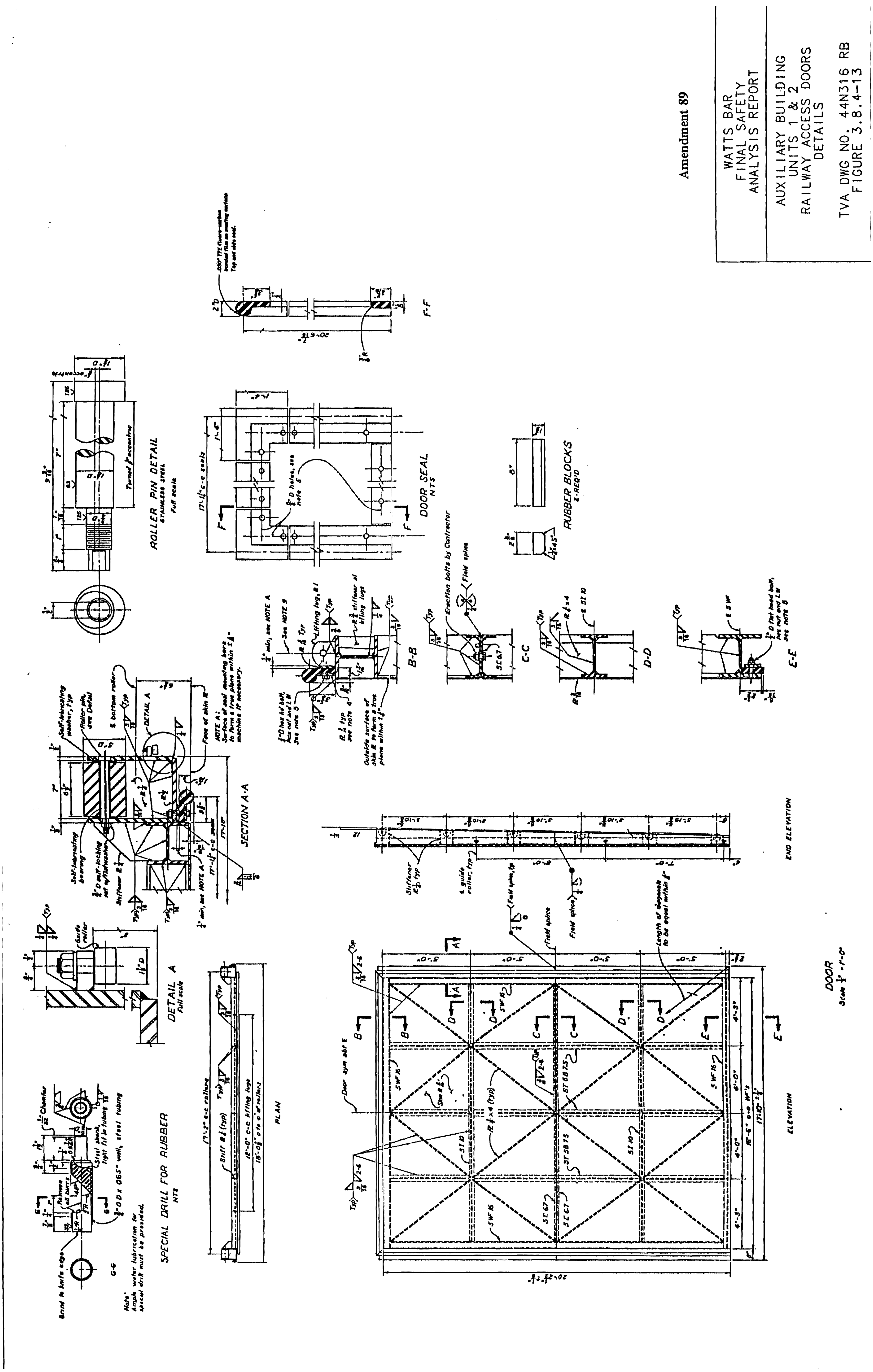


Figure 3.8.4-13 Auxiliary Building Units 1 & 2 Railway Access Doors Details

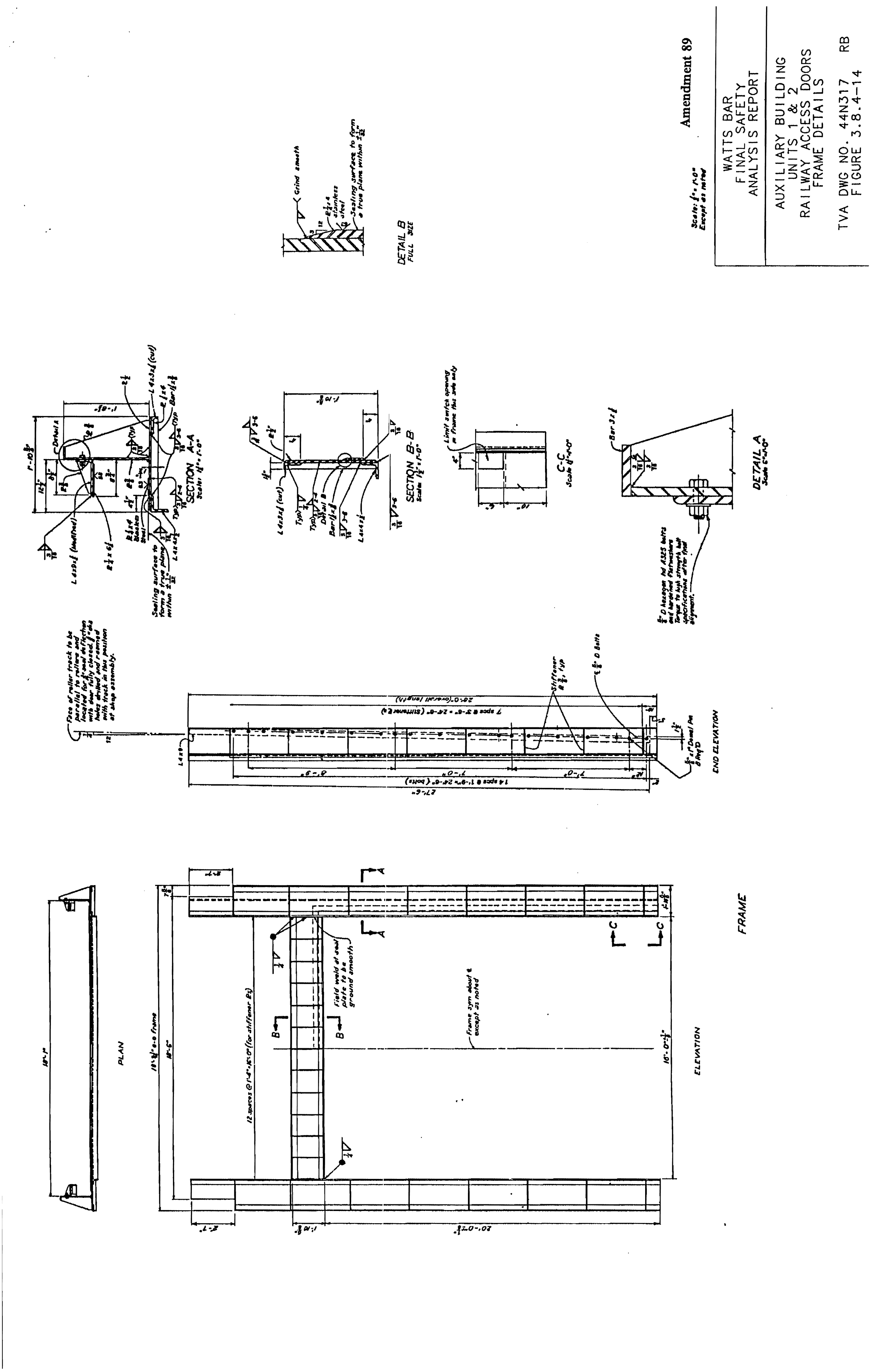


Figure 3.8.4-14 Auxiliary Building Units 1 & 2 Railway Access Doors Frame Details

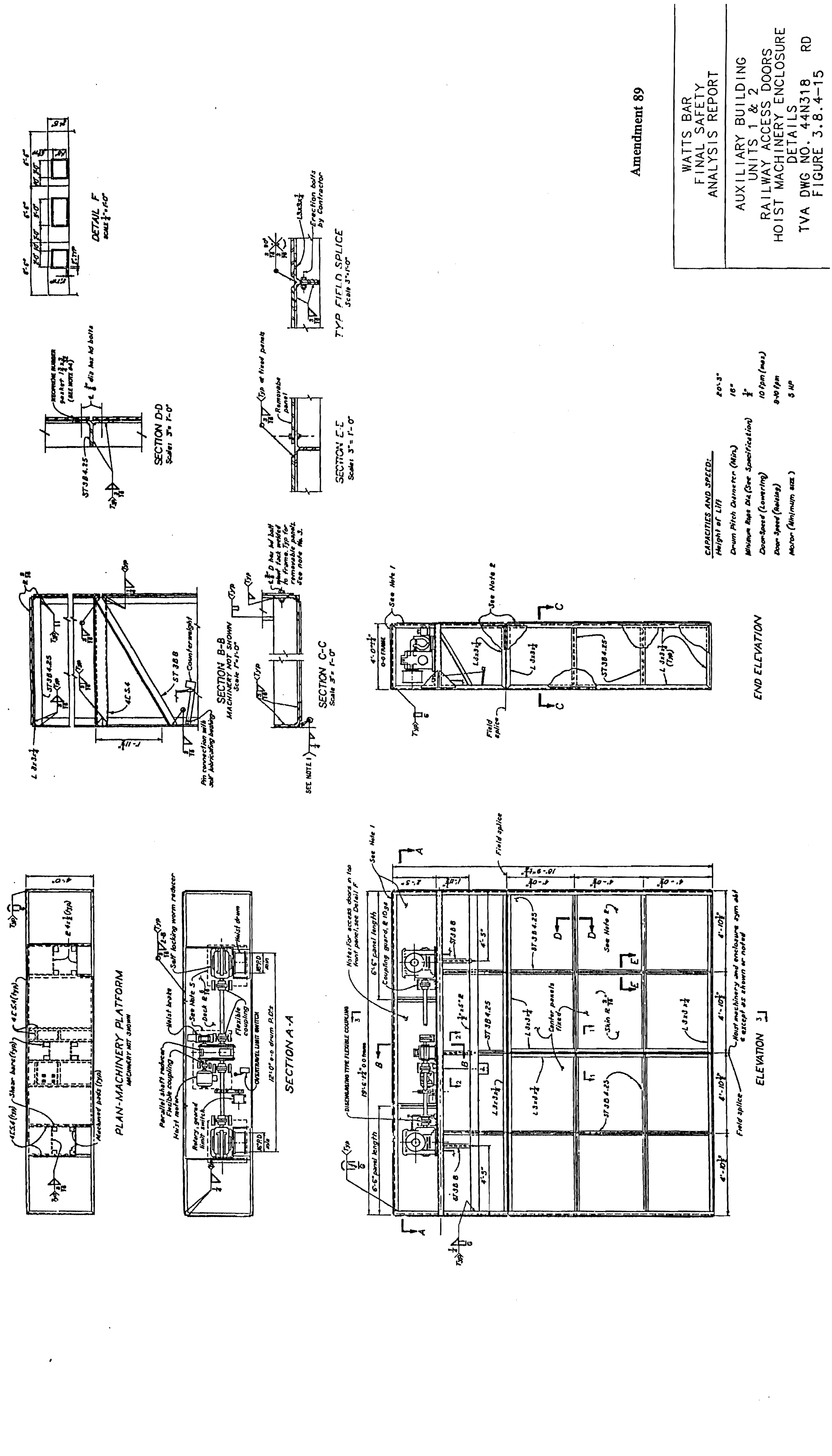


Figure 3.8.4-15 Auxiliary Building Units 1 & 2 Railway Access Doors Hoist Machinery Enclosure Details

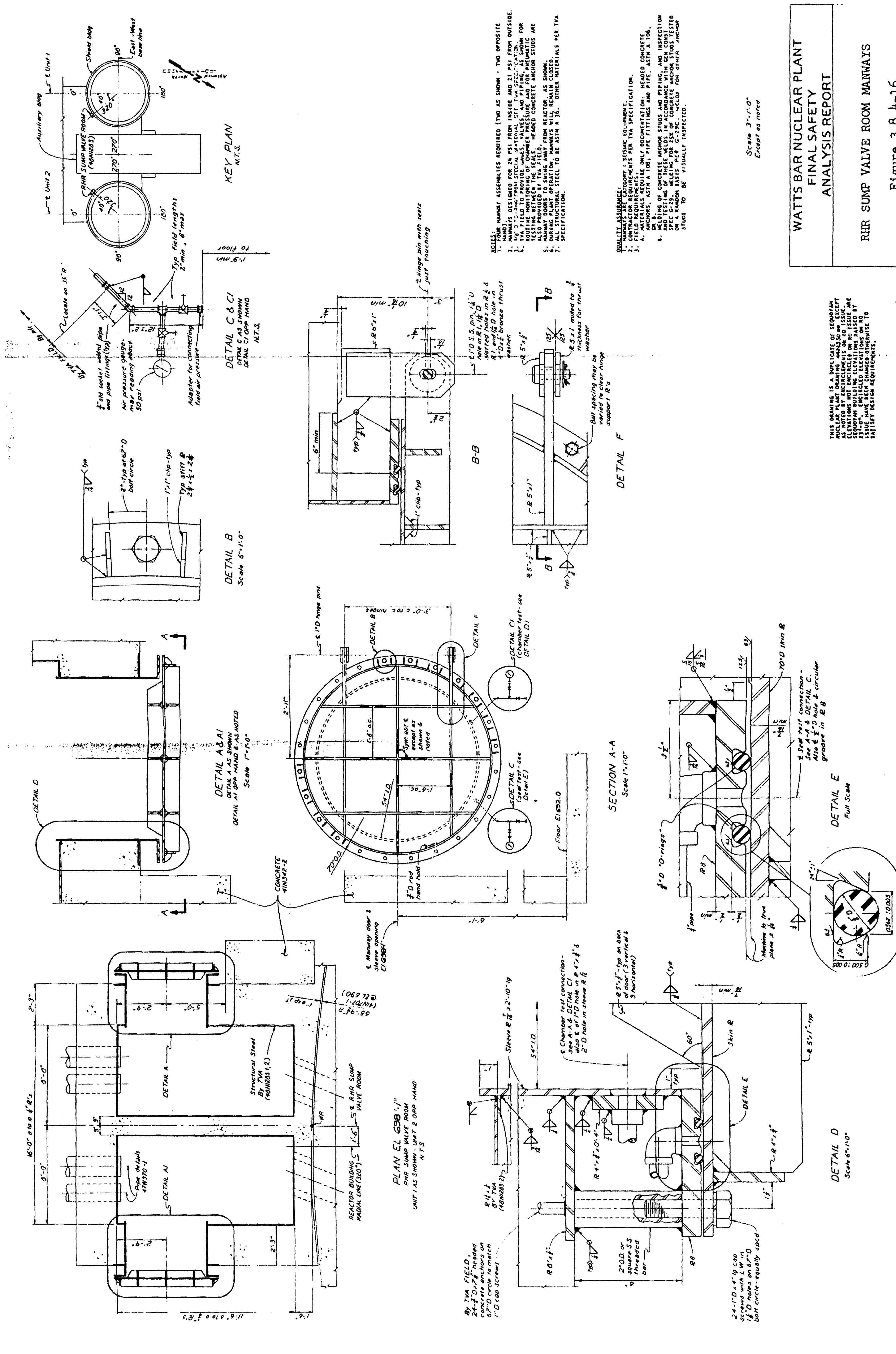


Figure 3.8.4-16 RHR Sump Valve Room Manways

Figure 3.8.4-17 Powerhouse, Auxiliary Reactor, & Control Buildings Architectural Plan Elevation 708.0 and 713.0

Figure 3.8.4-18 Powerhouse, Auxiliary Reactor, & Control Buildings Architectural Plan Elevation 729.0 and 737.0

Figure 3.8.4-19 Architectural Plan Elevation 755.0 and 757.0

Figure 3.8.4-20 Powerhouse, Auxiliary Reactor, & Control Buildings Architectural Plan Elevation 772.0, 782.0 and 786.0

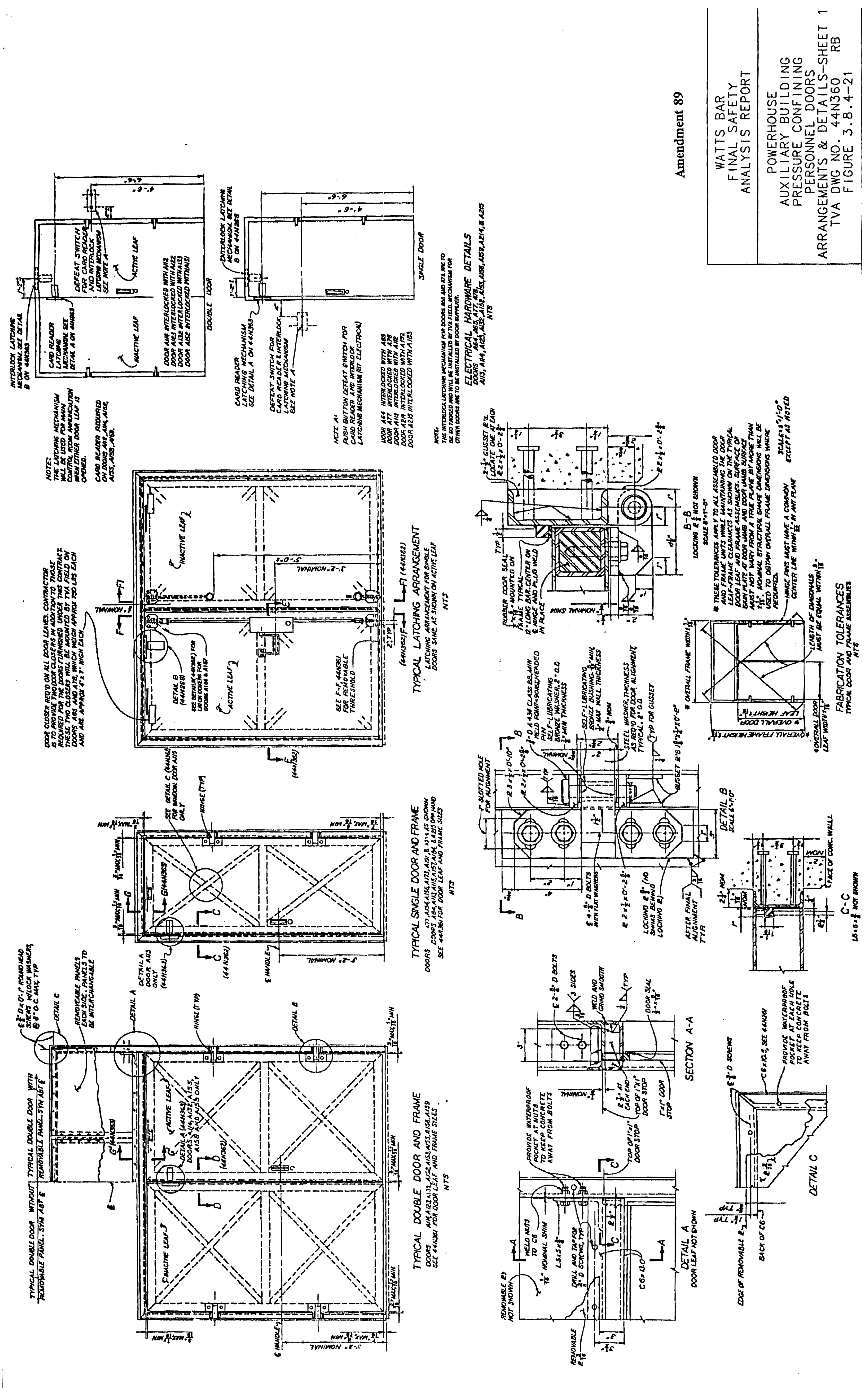


Figure 3.8.4-21 Powerhouse Auxiliary Building Pressure Confining Personnel Doors Arrangement and Details (Sheet 1)

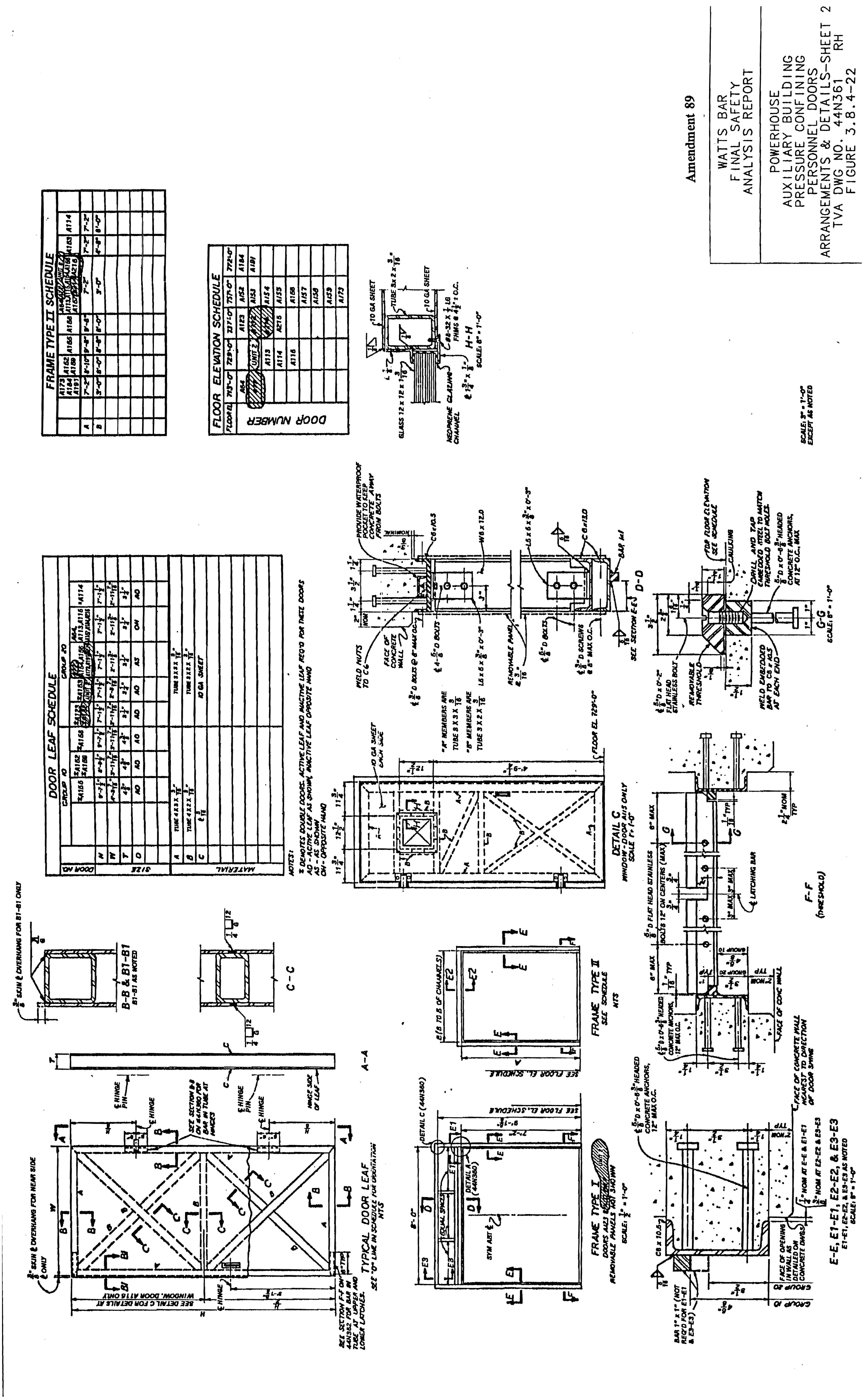


Figure 3.8.4-22 Powerhouse Auxiliary Building Pressure Confining Personnel Doors Arrangement and Details (Sheet 2)

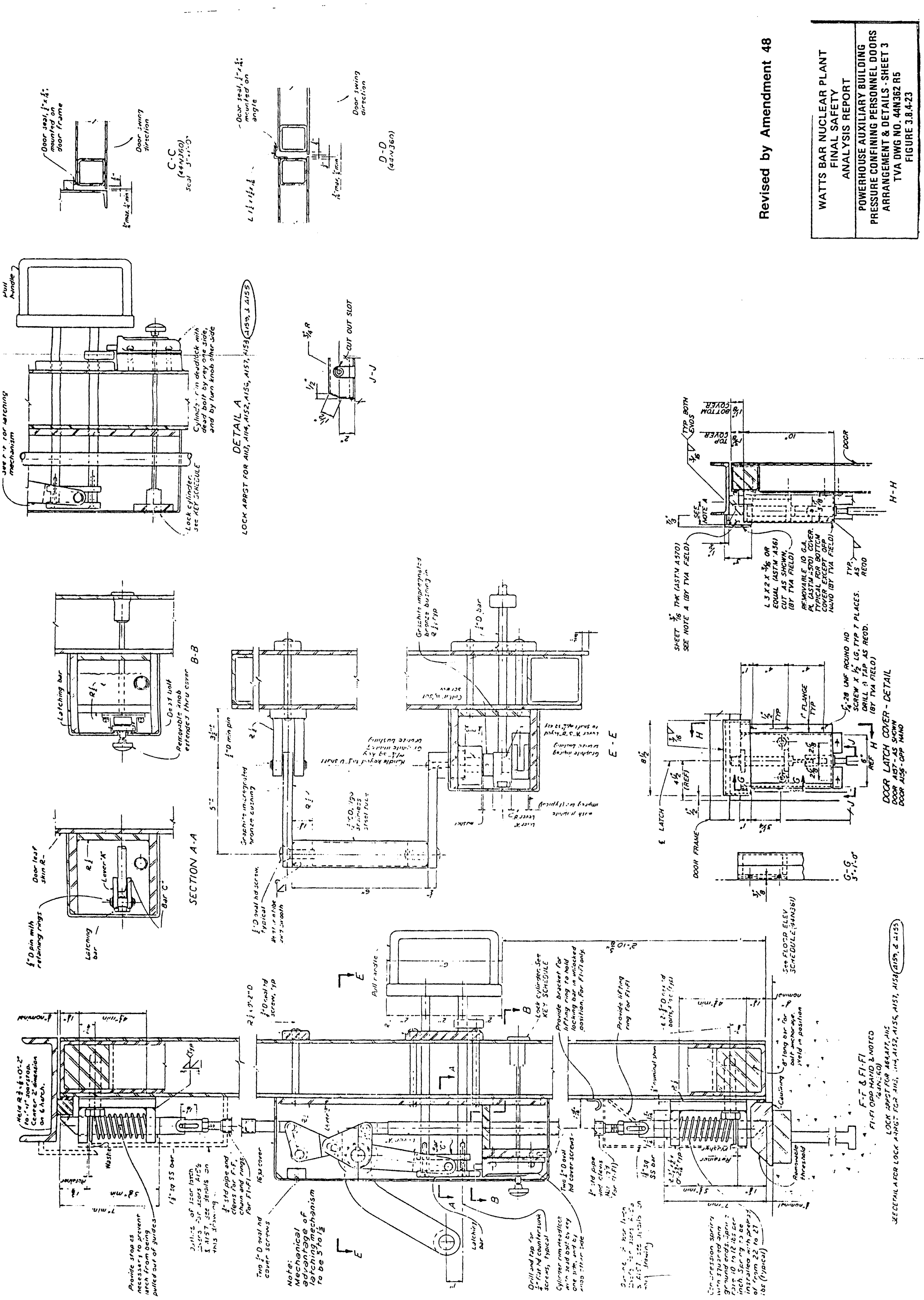


Figure 3.8.4-23 Powerhouse Auxiliary Building Pressure Confining Personnel Doors Arrangement and Details (Sheet 3)

Figure 3.8.4-24 Yard-Diesel Generator Building Concrete Floors and Walls Outline (Sheet 1)

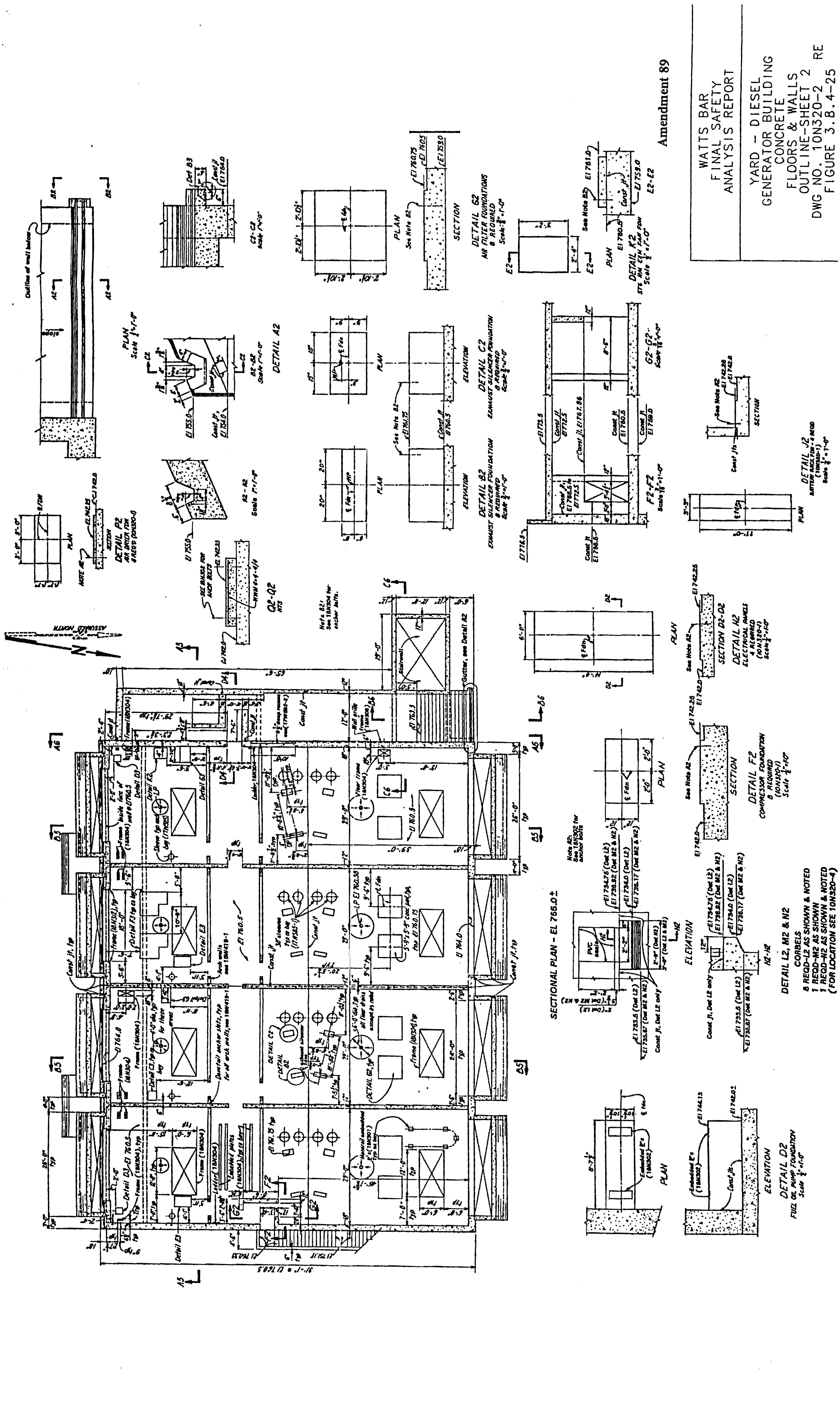


Figure 3.8.4-25 Yard-Diesel Generator Building Concrete Floors and Walls Outline (Sheet 2)

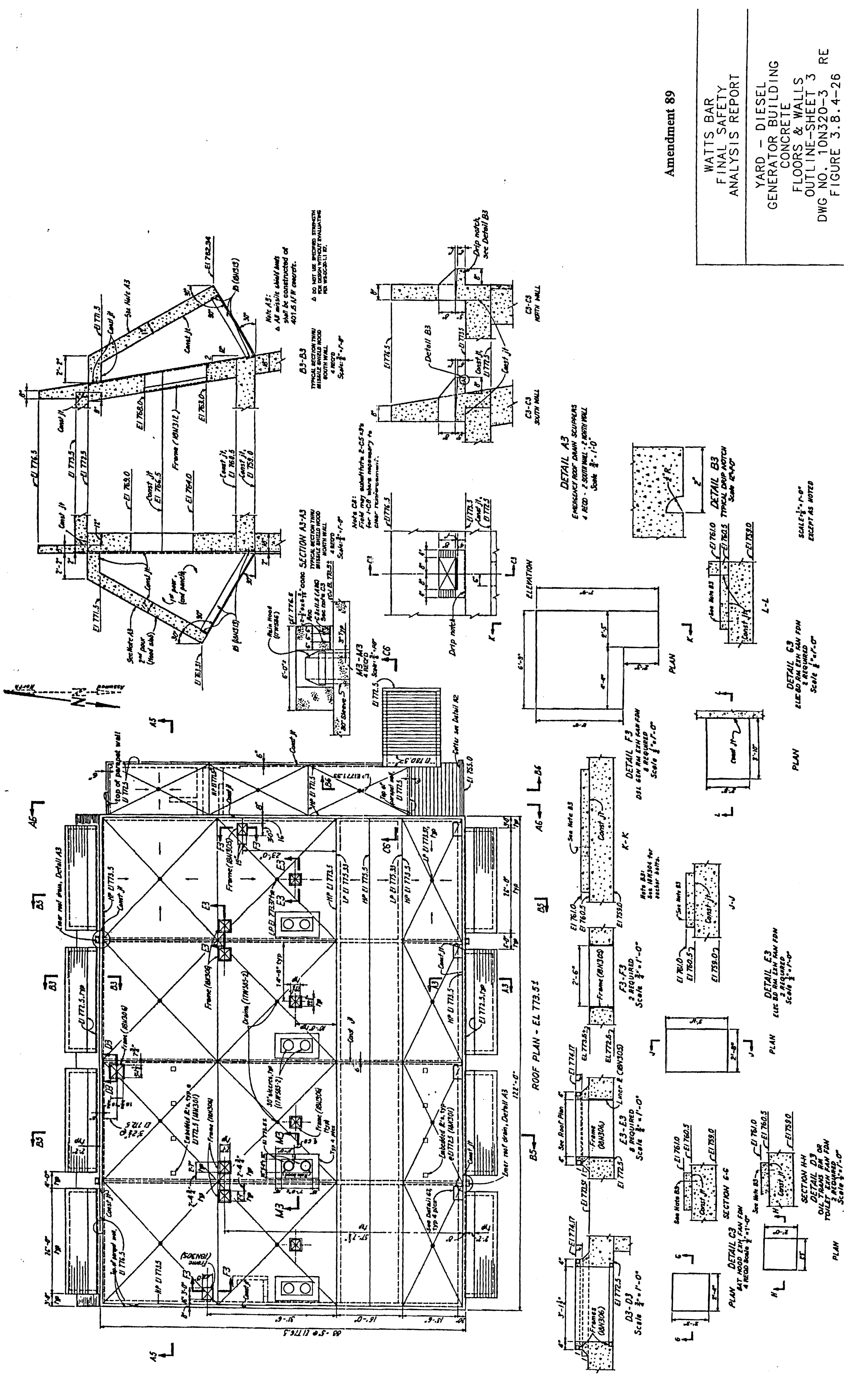
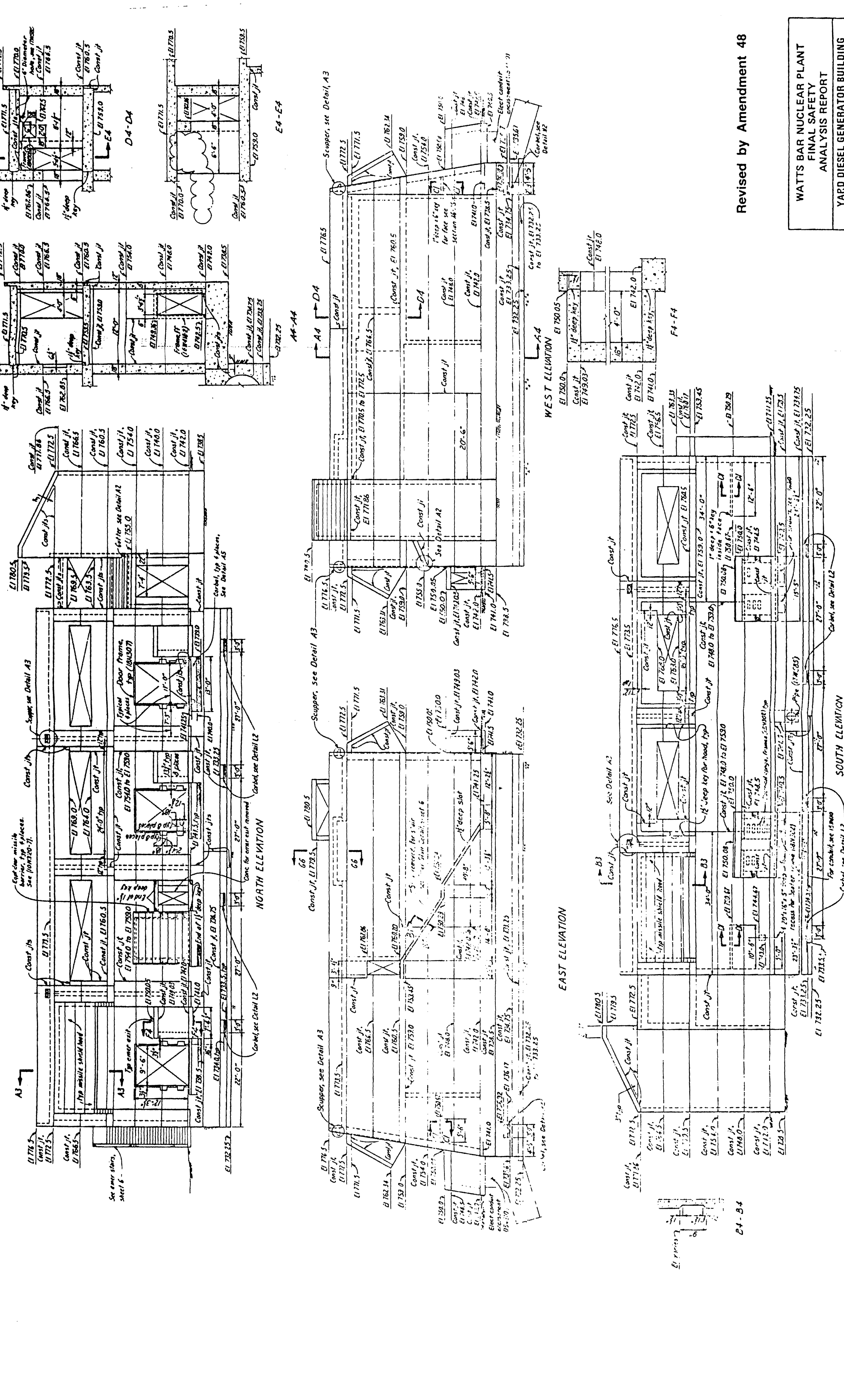


Figure 3.8.4-26 Yard-Diesel Generator Building Concrete Floors and Walls Outline (Sheet 3)



Revised by Amendment 48

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

YARD DIESEL GENERATOR BUILDING
CONCRETE FLOOR AND WALLS
OUTLINE - SHEET 4
TVA DWG NO. 10N320-4 R6
FIGURE 3.8.4-27

Figure 3.8.4-27 Yard-Diesel Generator Building Concrete Floor and Walls Outline (Sheet 4)

Figure 3.8.4-28 Diesel Generator Building Floor and Wall Outline (Sheet 5)

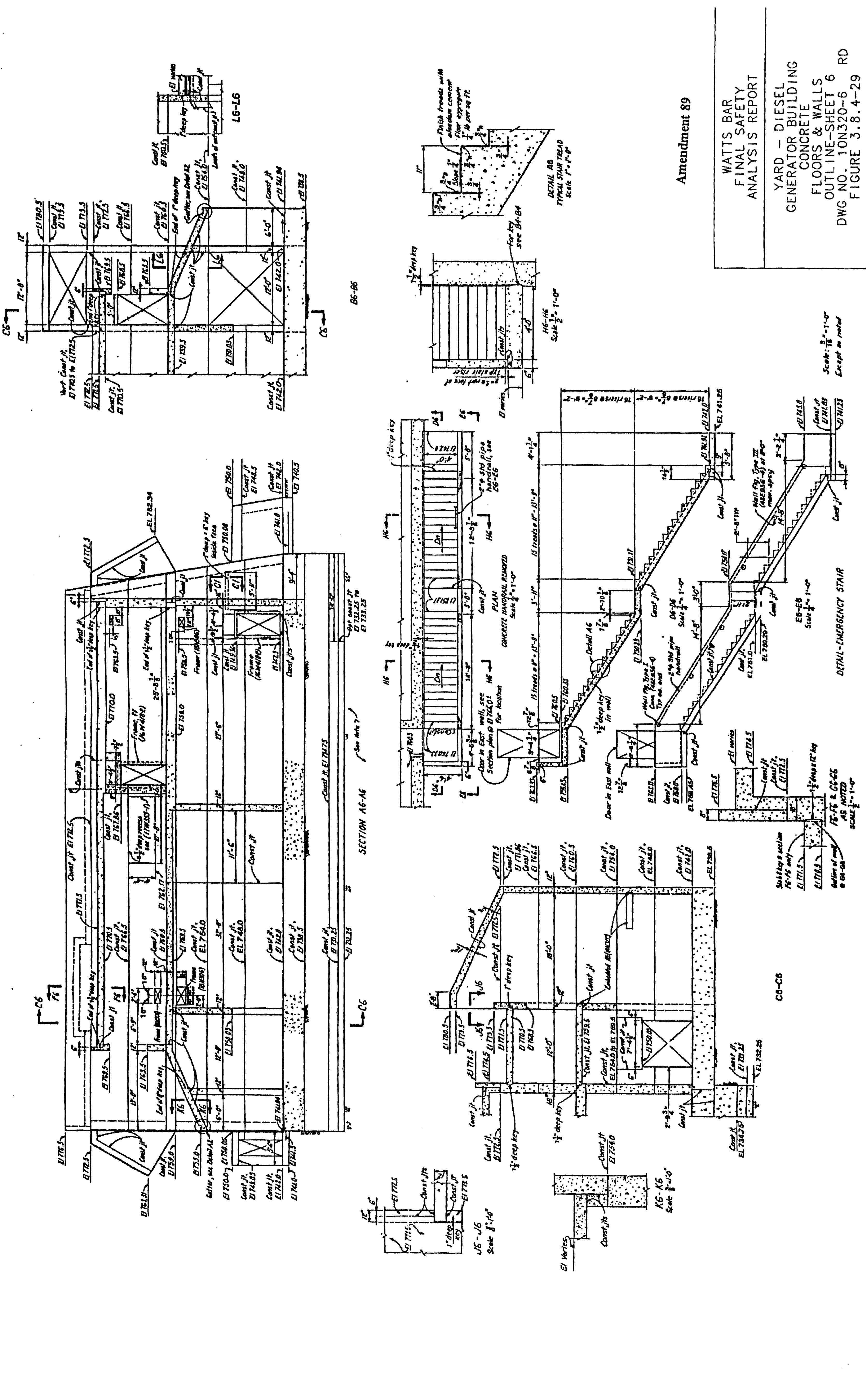


Figure 3.8.4-29 Yard-Diesel Generator Building Concrete Floors and Walls Outline (Sheet 6)

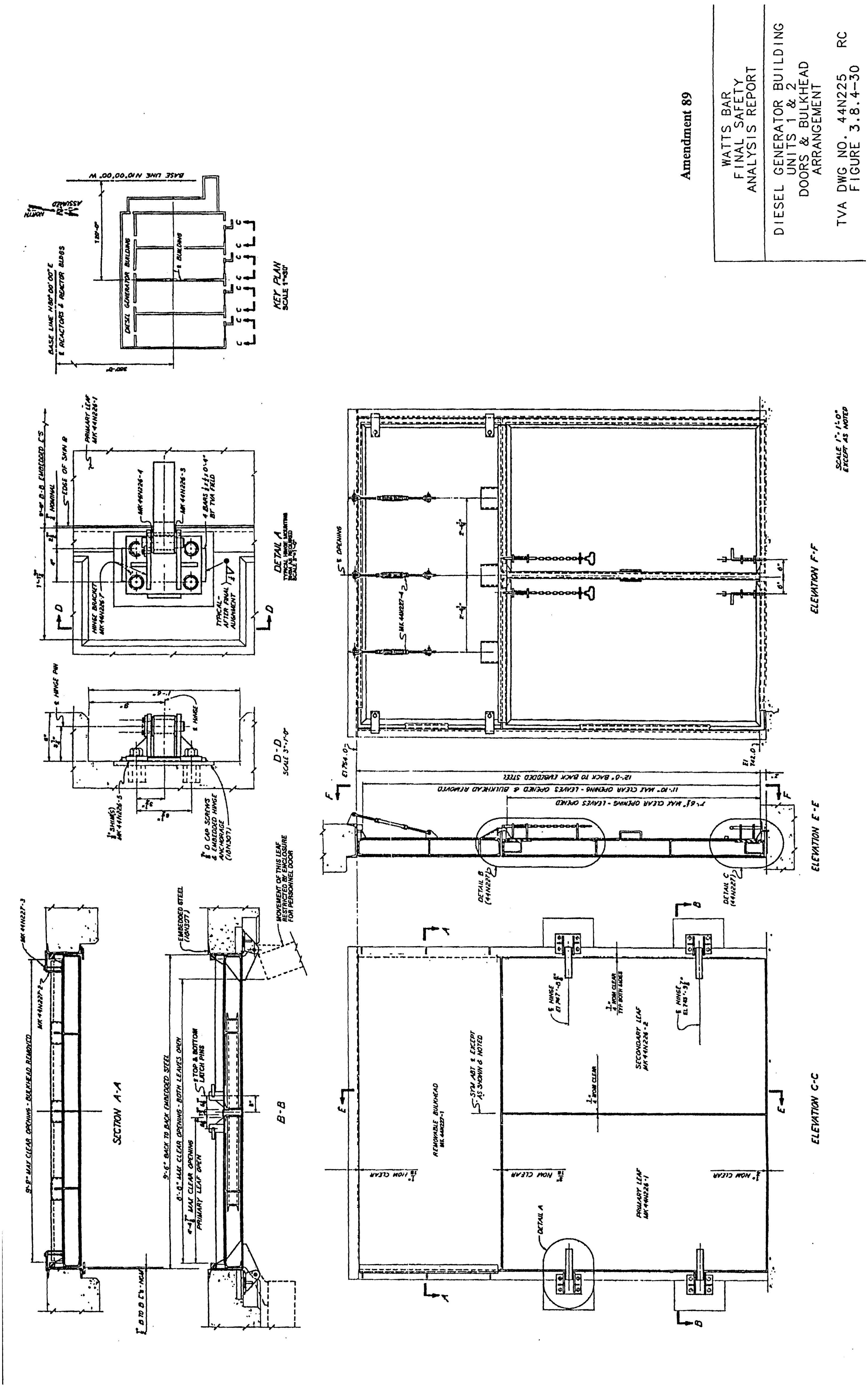


Figure 3.8.4-30 Diesel Generator Building Units 1 & 2 Doors & Bulkhead Arrangement

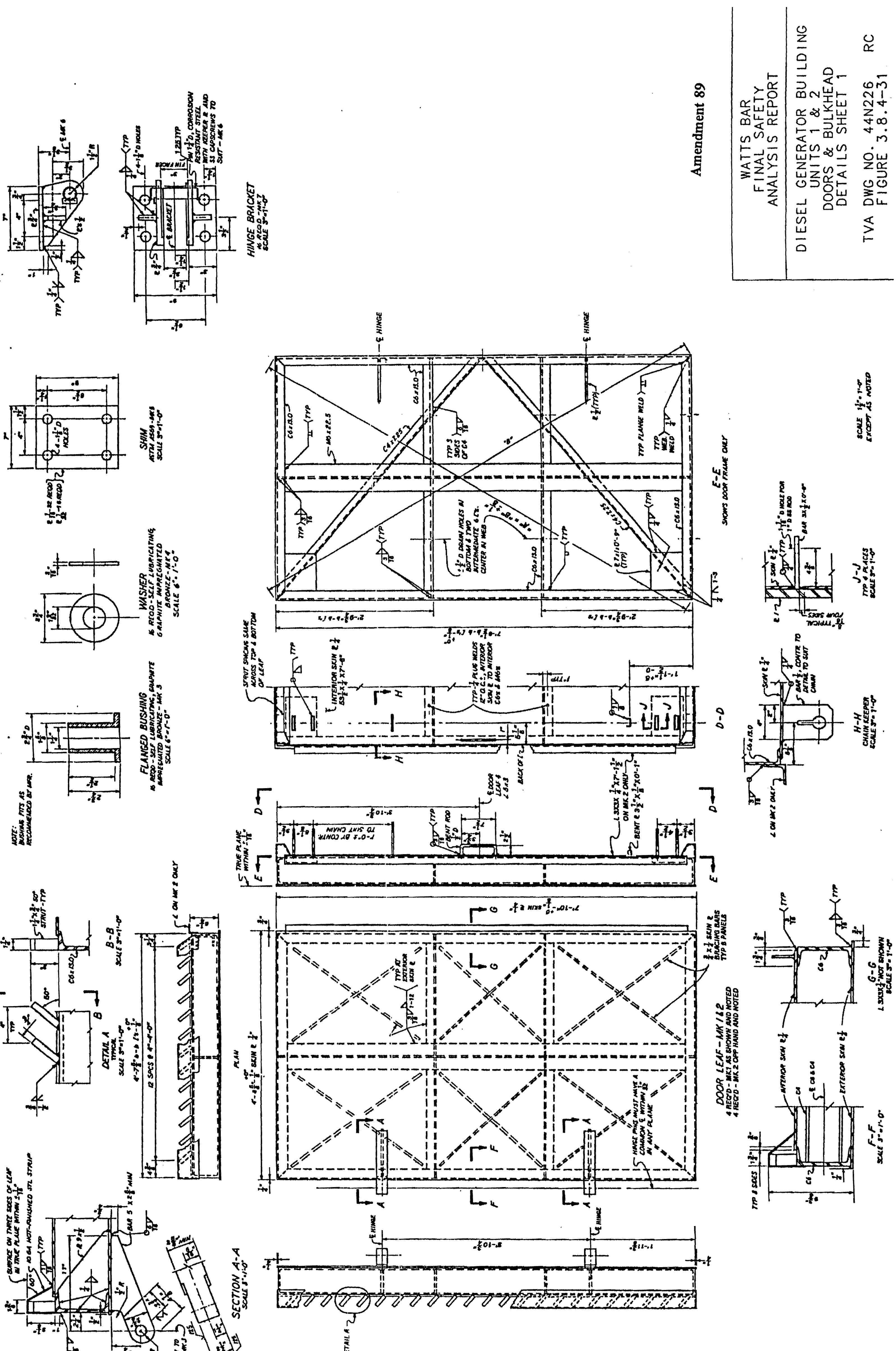


Figure 3.8.4-31 Diesel Generator Building Units 1 & 2 Doors and Bulkhead Details (Sheet 1)

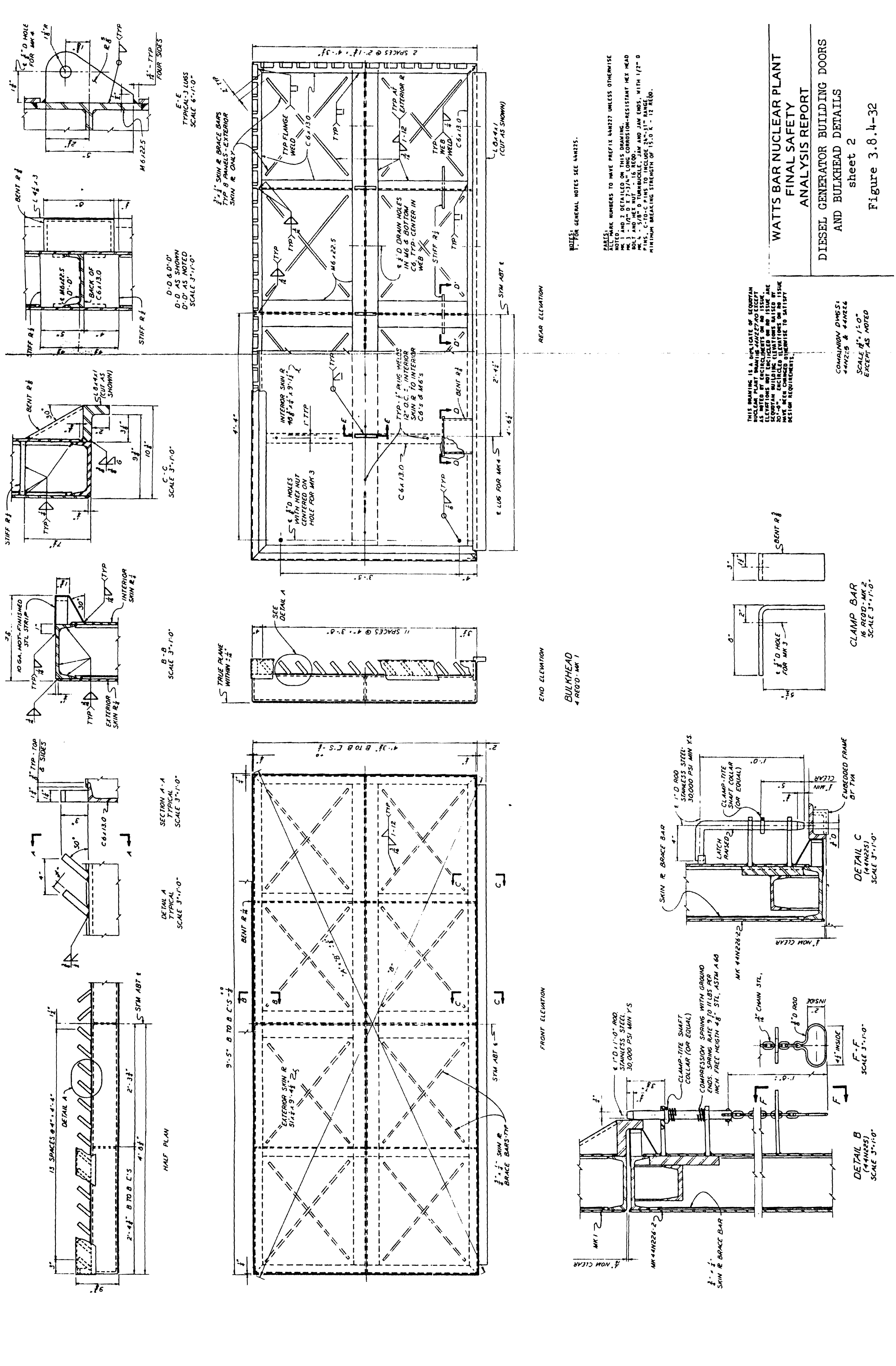


Figure 3.8.4-32 Diesel Generator Building Doors and Bulkhead Details (Sheet 2)

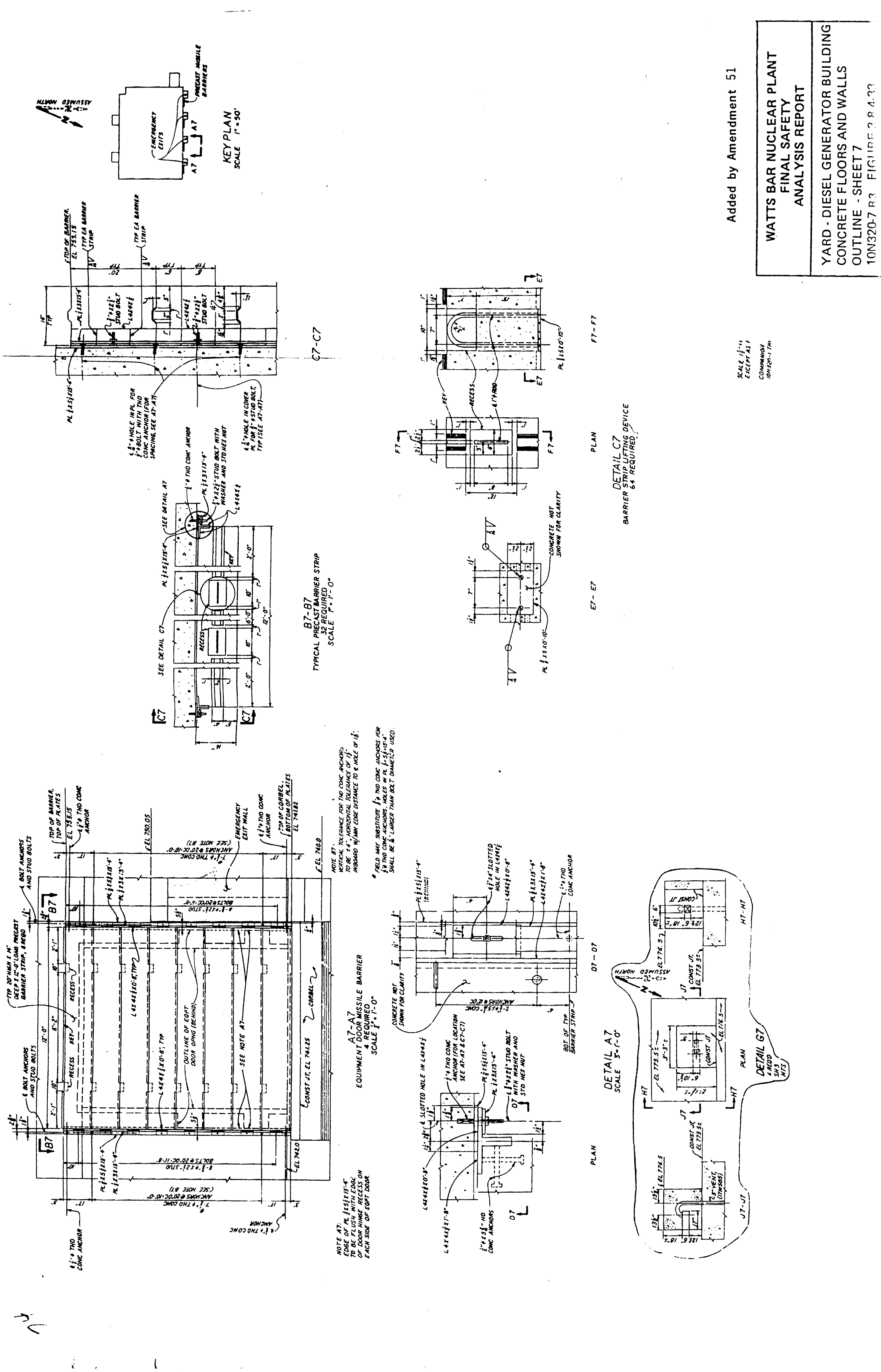


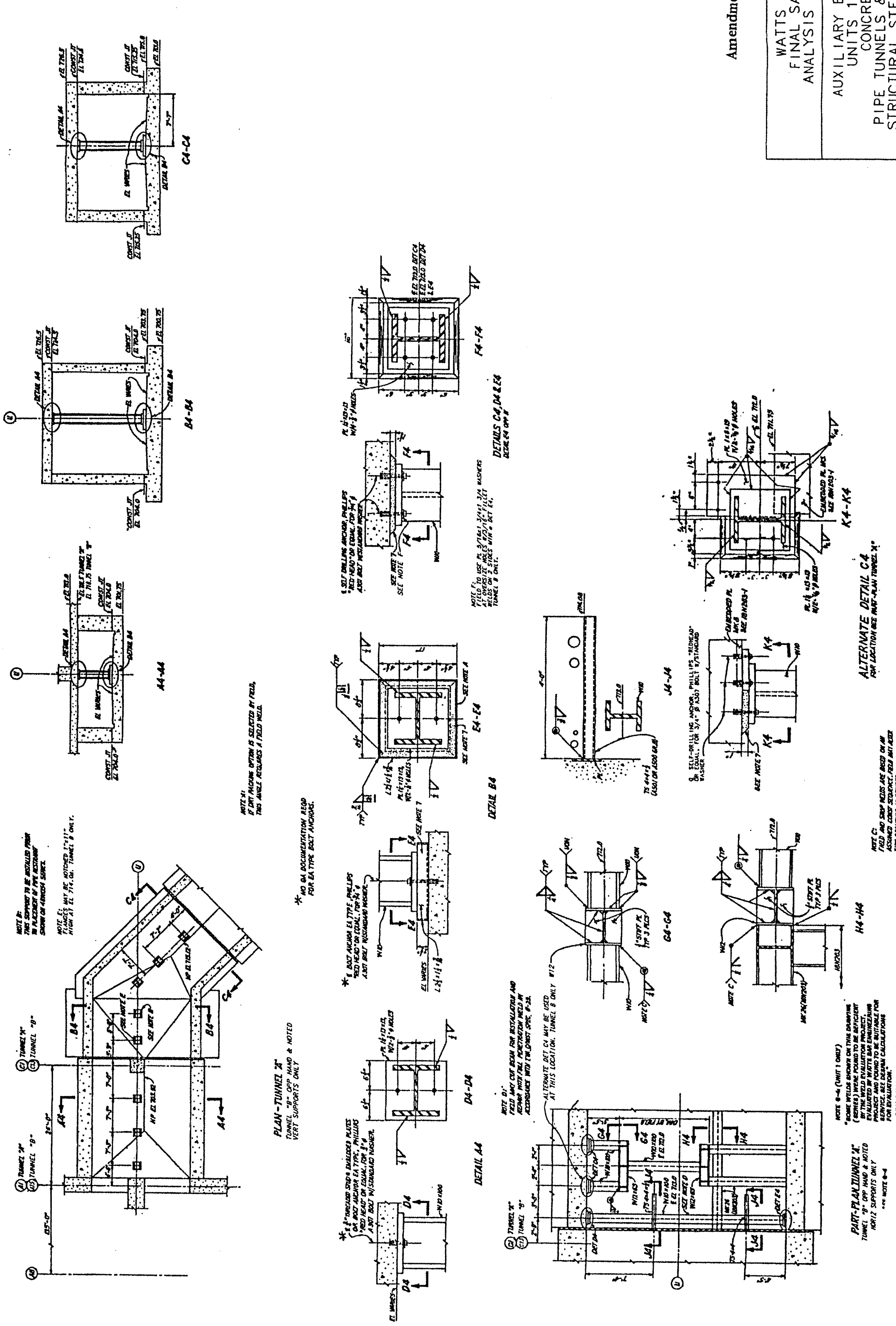
Figure 3.8.4-33 Yard-Diesel Generator Building Concrete Floors and Walls Outline (Sheet 7)

Figure 3.8.4-34 Deleted -Amendment 62

Figure 3.8.4-35 Auxiliary Building Units 1 & 2 Concrete Pipe Tunnels and Tank Foundations Outline (Sheet 1)

Figure 3.8.4-36 Auxiliary Building Units 1 & 2 Concrete Pipe Tunnels and Tank Foundations Outline (Sheet 2)

Figure 3.8.4-36a Auxiliary Building Units 1 & 2 Concrete Pipe Tunnels and Tank Foundations Outline



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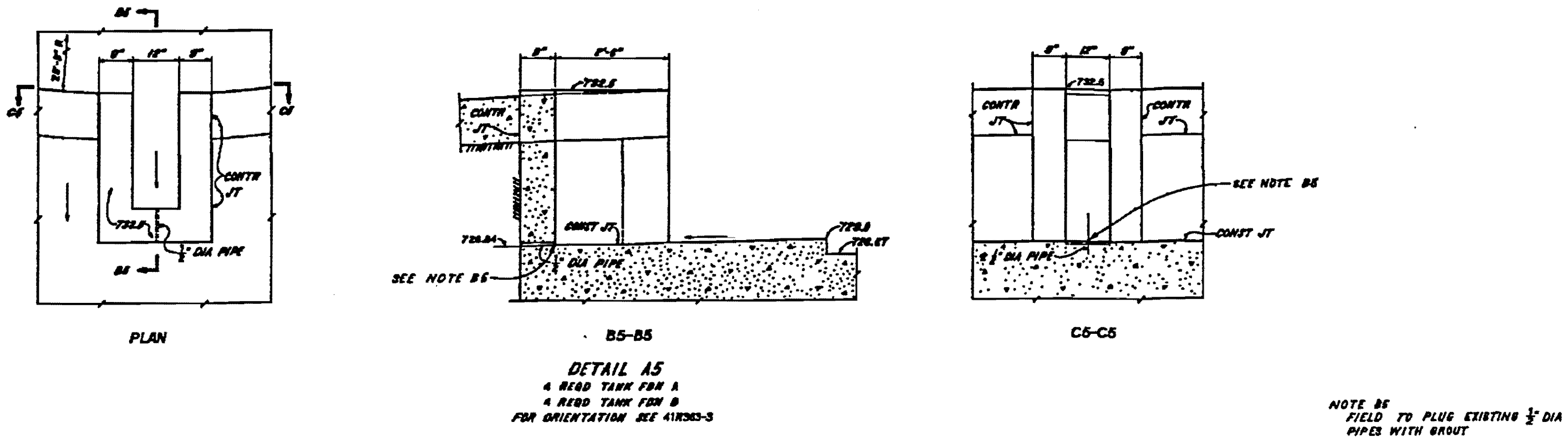
WATTS BAR
FINAL SAFETY
ANALYSIS REPORT

AUXILIARY BUILDING
UNITS 1 & 2
CONCRETE
PIPE TUNNELS & TANK FDNS
STRUCTURAL STEEL SUPPORTS
TVA DWG NO. 41N363-4 RC
FIGURE 3.8.4-36B

Figure 3.8.4-36b Auxiliary Building Units 1 & 2 Concrete Pipe Tunnels and Tank Foundations



Amendment 89

WATTS BAR
FINAL SAFETY
ANALYSIS REPORT

AUXILIARY BUILDING
UNITS 1 & 2
PIPE TUNNELS & TANK FDNS
OUTLINE
TVA DWG NO. 41N363-5 RC
FIGURE 3.8.4-36C

PROCADAM MAINTAINED DRAWING

Figure 3.8.4-36c Auxiliary Building Units 1 & 2 Concrete Pipe Tunnels and Tank Foundations Outline

Figure 3.8.4-37 Yard Units 1 & 2 Concrete Manholes and Duct Runs Outline and Reinforcement

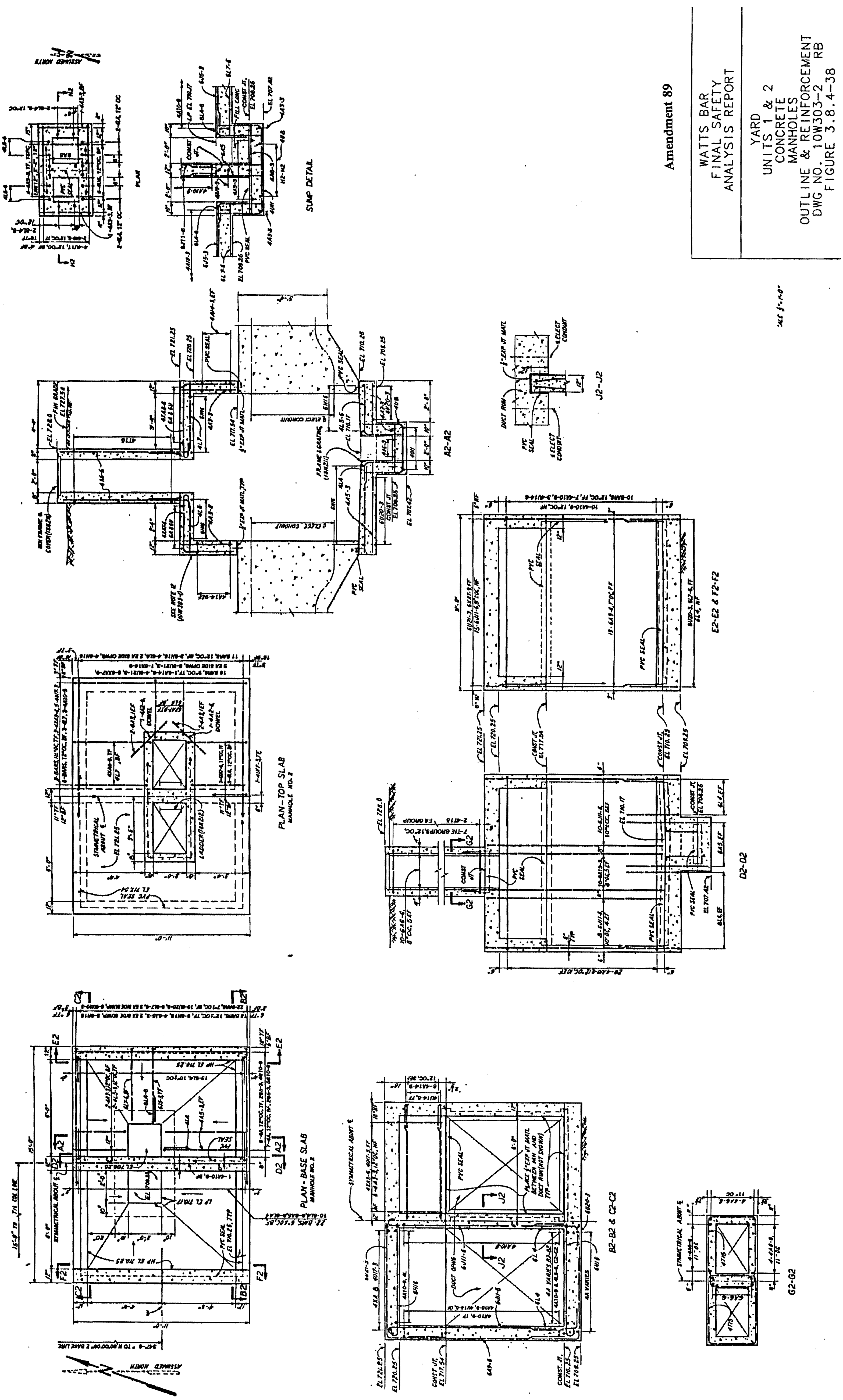


Figure 3.8.4-38 Yard Units 1 & 2 Concrete Manholes Outline and Reinforcement (Sheet 1)

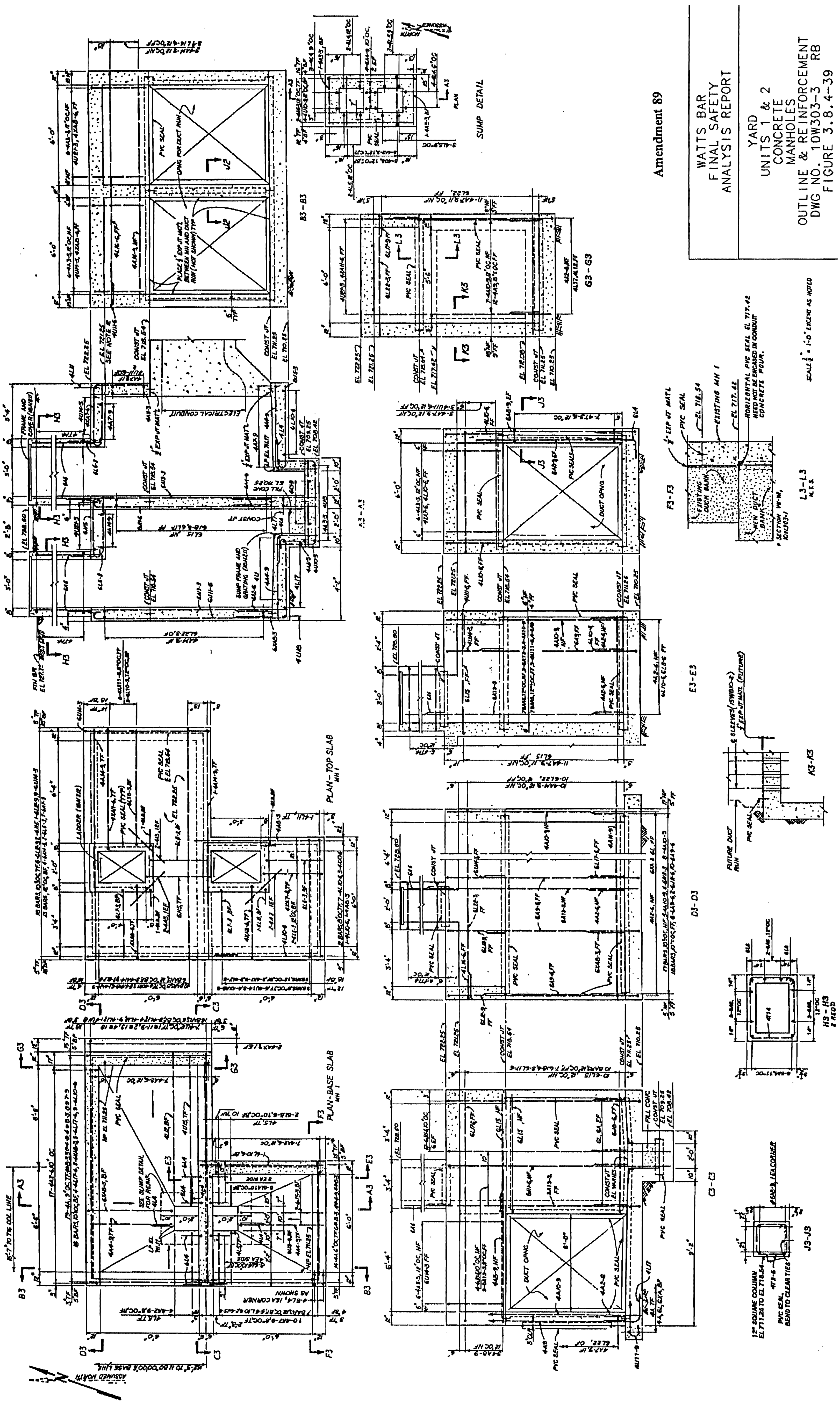


Figure 3.8.4-39 Yard Units 1 & 2 Concrete Manholes Outline and Reinforcement (Sheet 2)

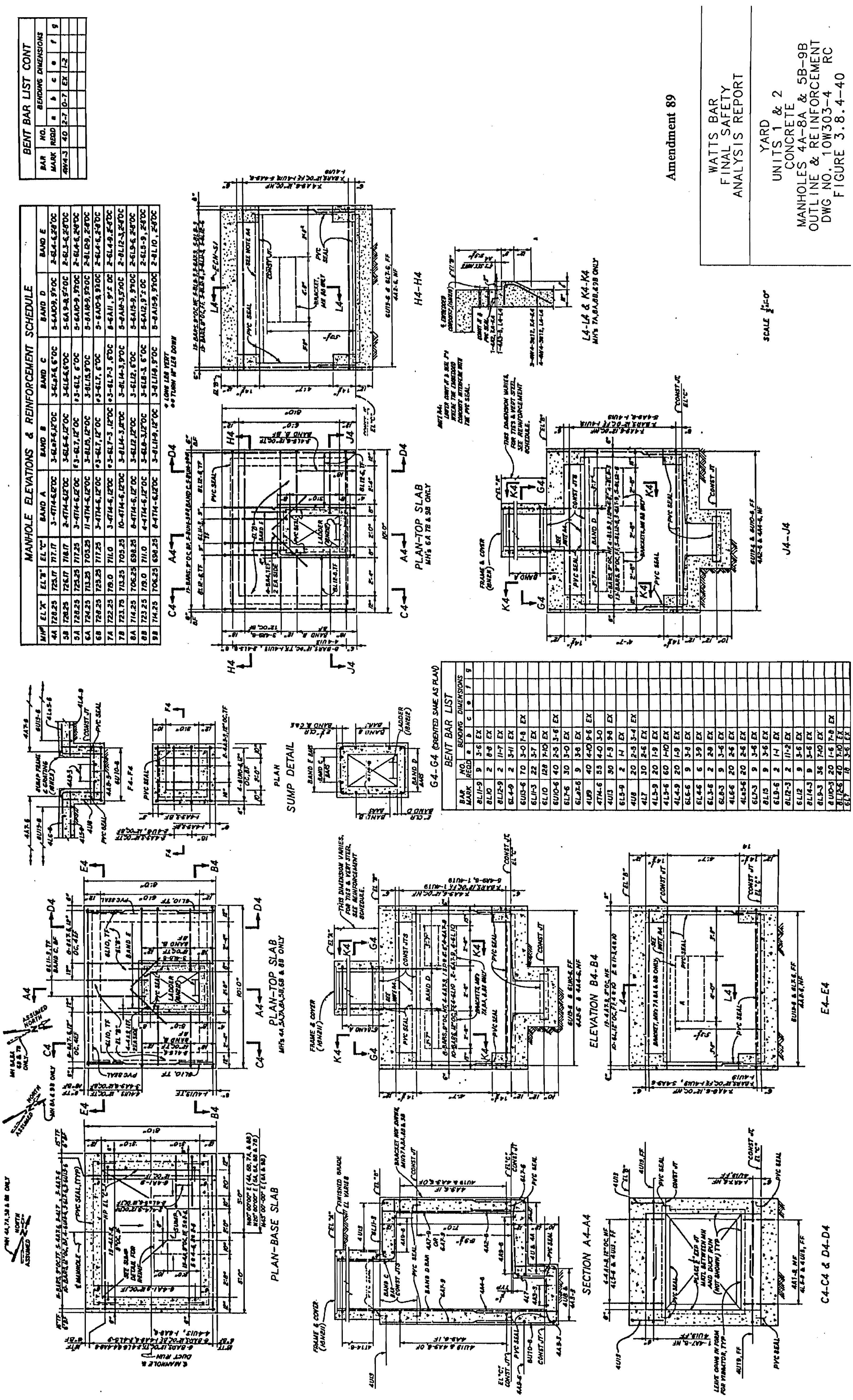


Figure 3.8.4-40 Yard Units 1 & 2 Concrete Manholes 4A-8A &5B-9B Outline and Reinforcement (Sheet 3)

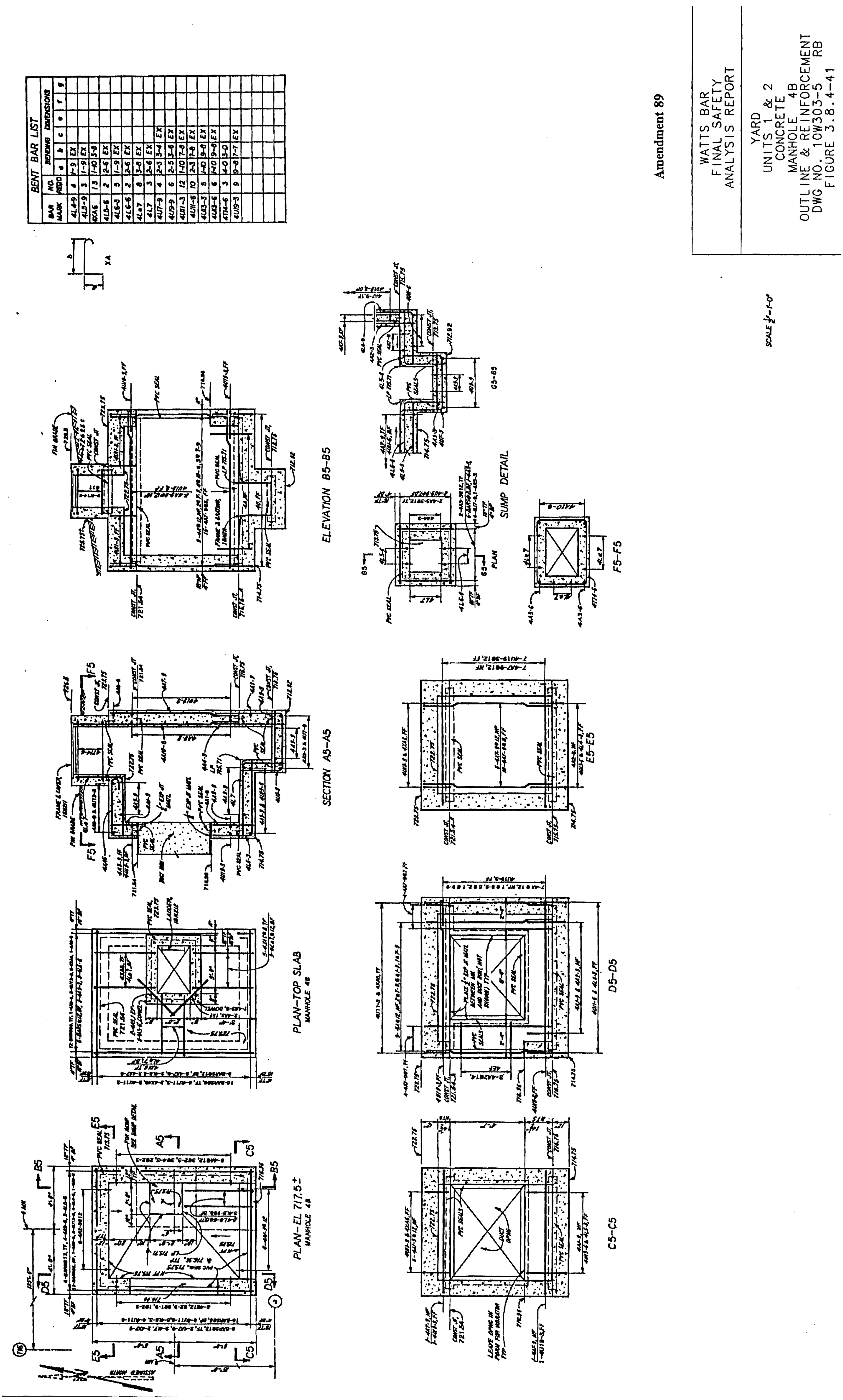


Figure 3.8.4-41 Yard Units 1 & 2 Concrete Manhole 4B Outline and Reinforcement (Sheet 4)

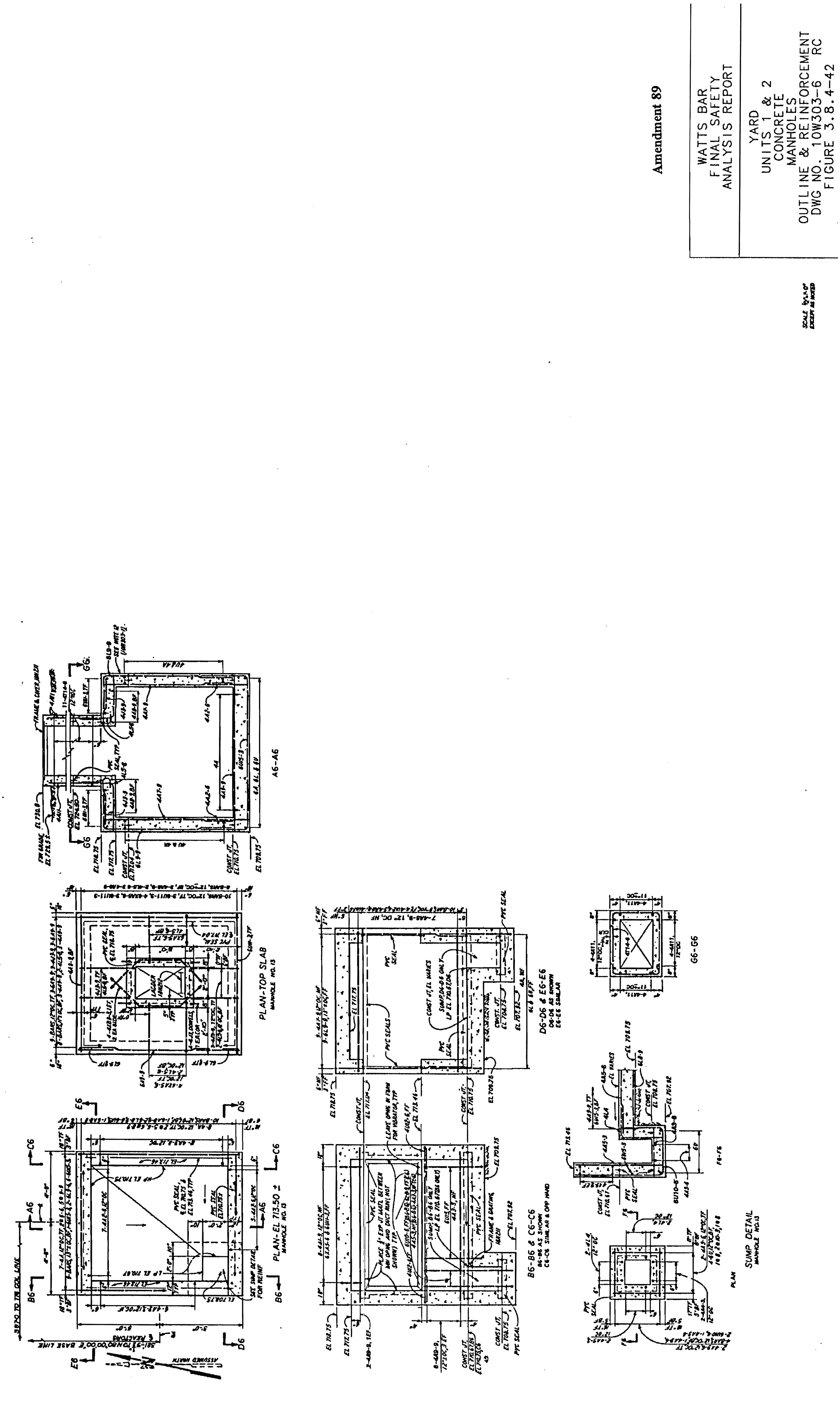


Figure 3.8.4-42 Yard Units 1 & 2 Concrete Manholes Outline and Reinforcement (Sheet 5)

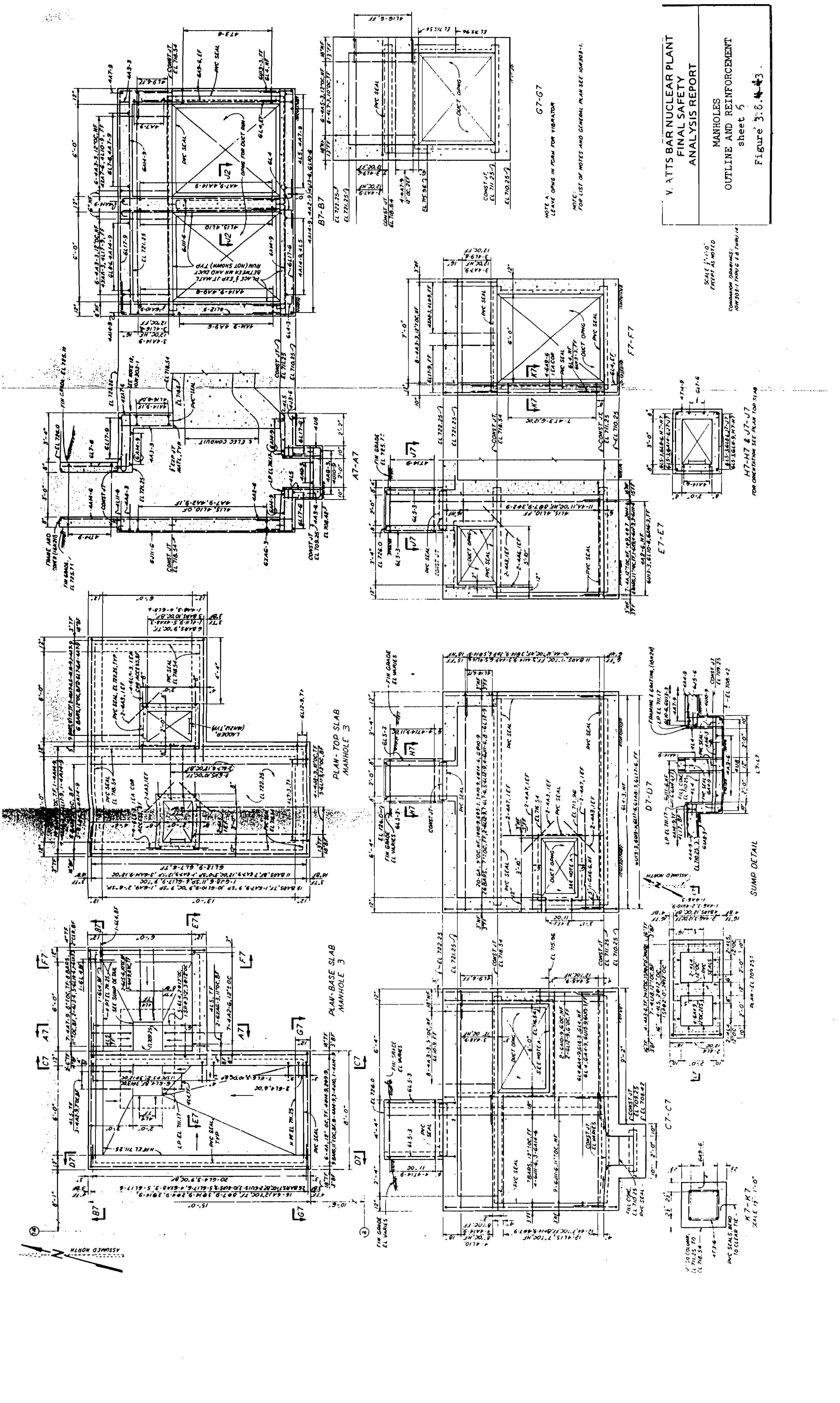


Figure 3.8.4-43 Manholes Outline and Reinforcement (Sheet 6)

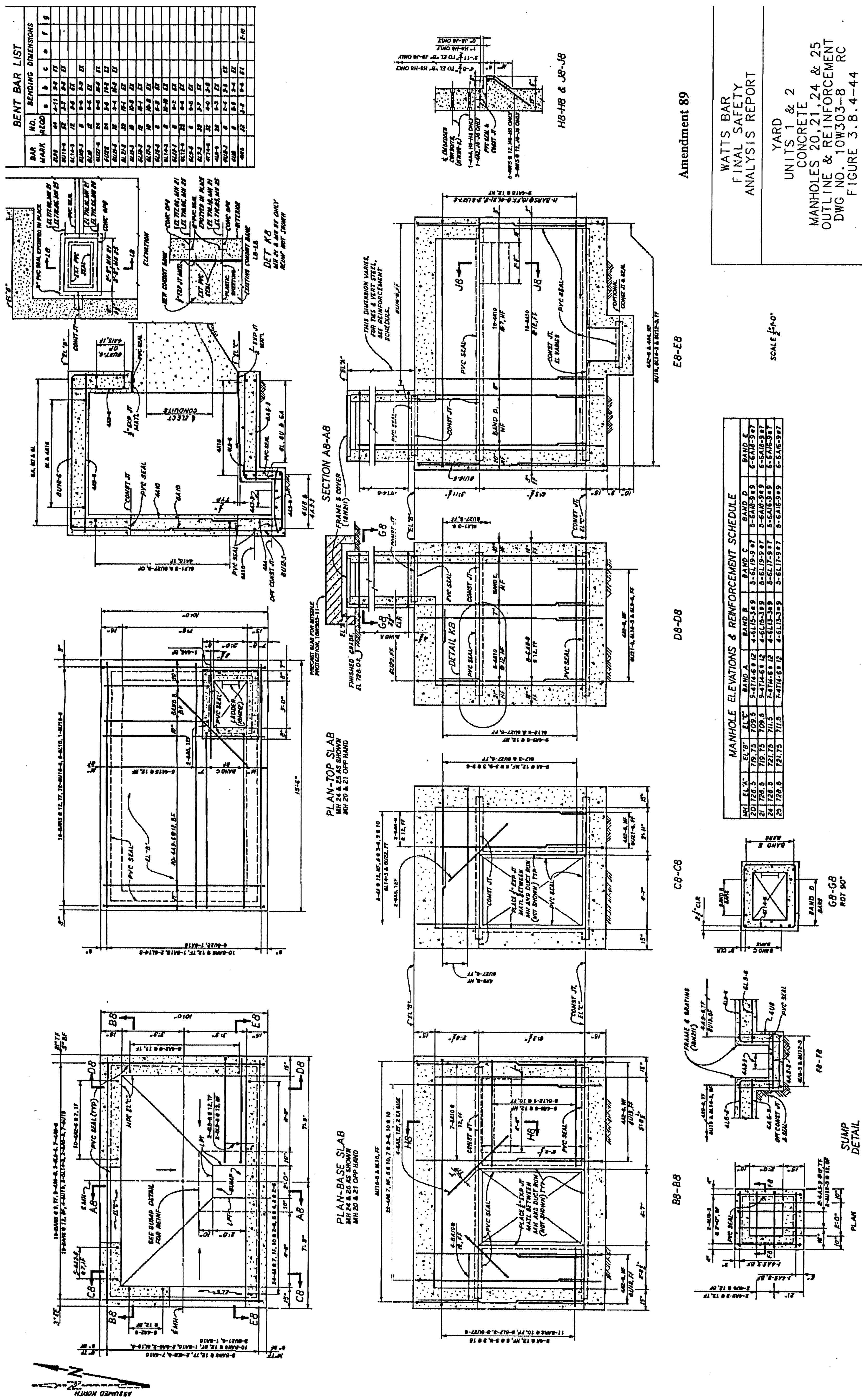


Figure 3.8.4-44 Yard Units 1 & 2 Concrete Manholes 20, 21, 24 & 25 Outline & Reinforcement

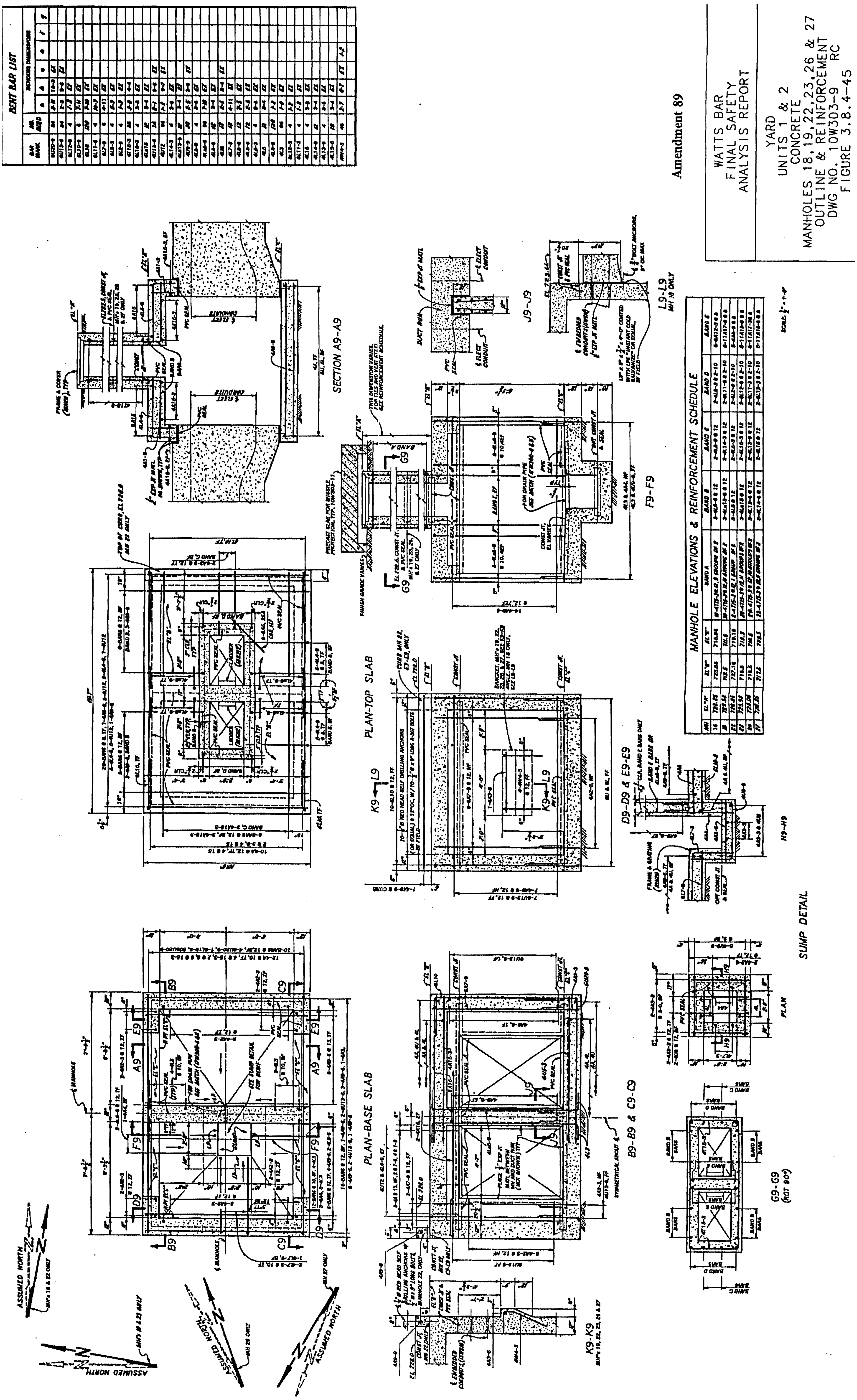


Figure 3.8.4-45 Yard Units 1 & 2 Concrete Manholes 18, 19, 22, 23, 26 & 27 Outline & Reinforcement

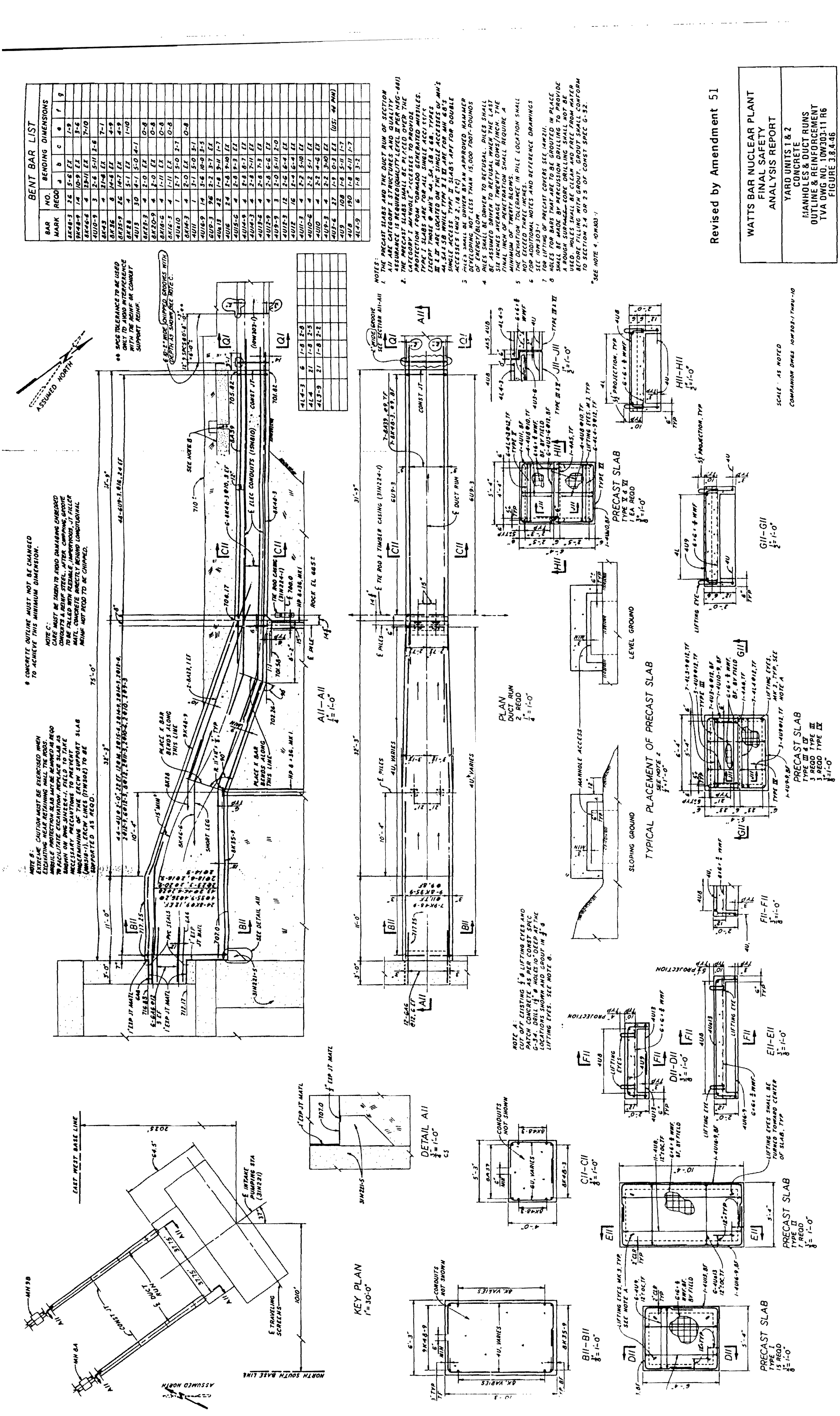


Figure 3.8.4-46 Yard Units 1 & 2 Concrete Manholes & Duct Runs Outline & Reinforcement

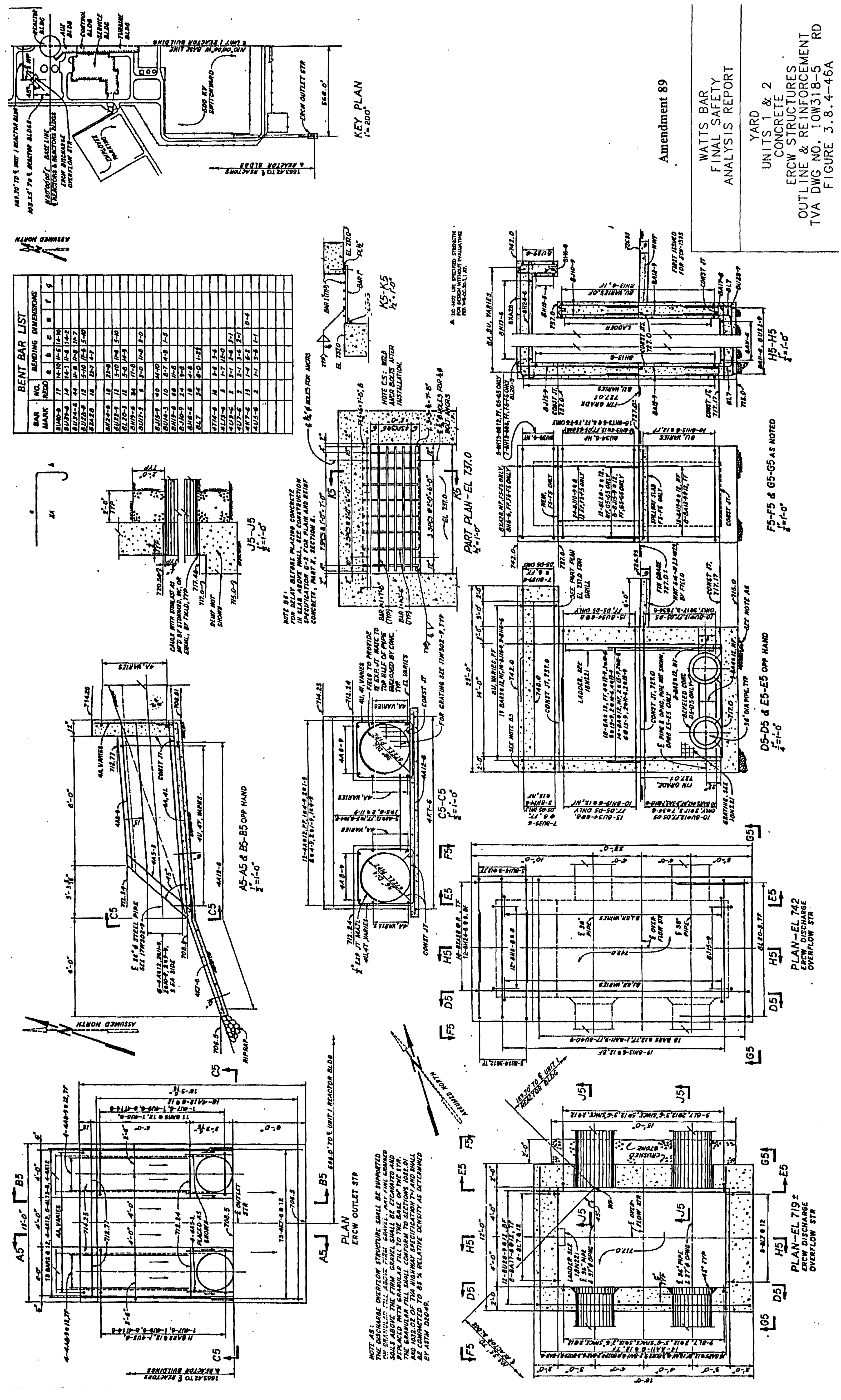


Figure 3.8.4-46a Yard Units 1 & 2 Concrete ERCW Structures Outline and Reinforcement

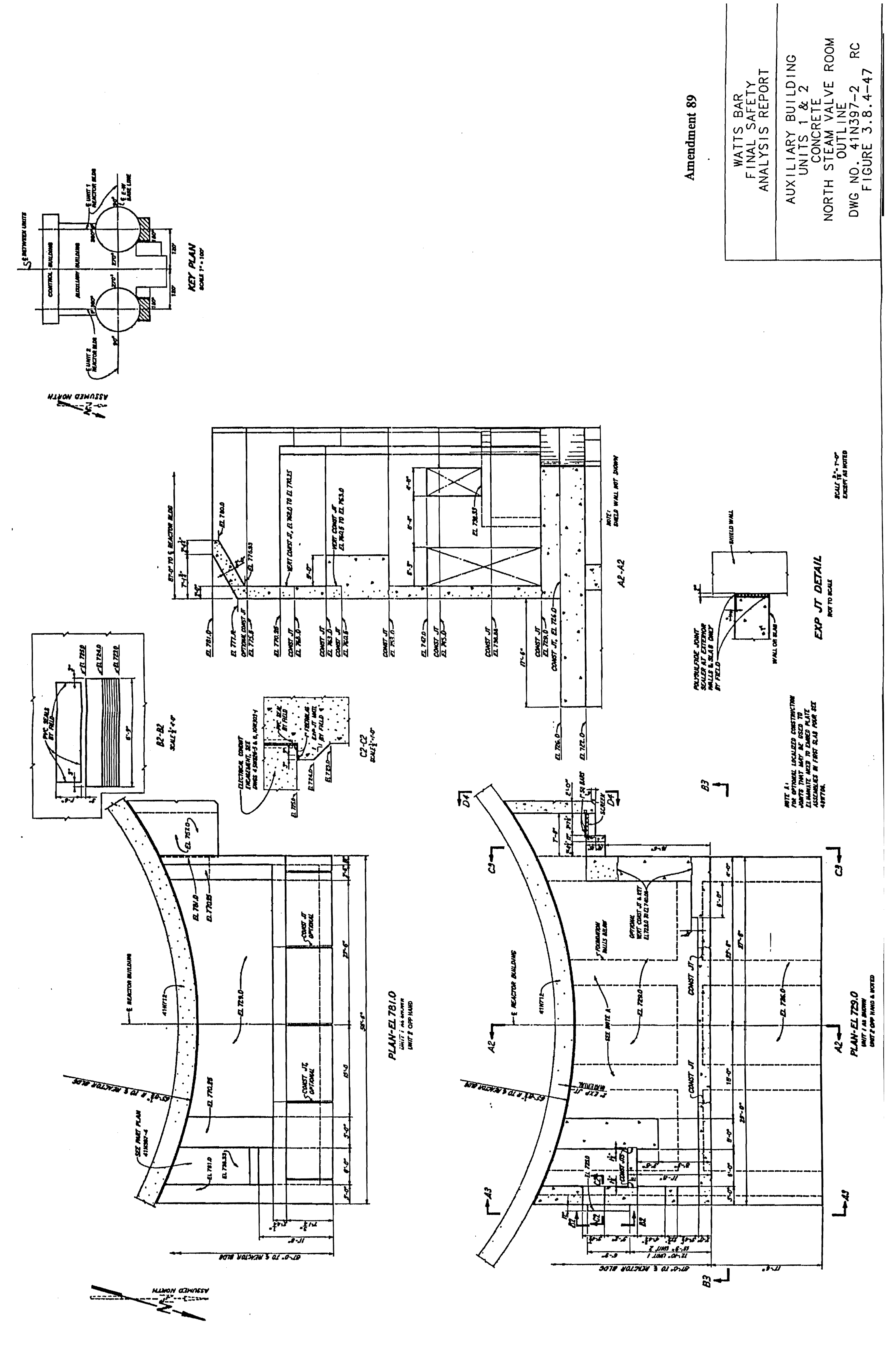


Figure 3.8.4-47 Auxiliary Building Units 1 & 2 Concrete - North Steam Valve Room Outline

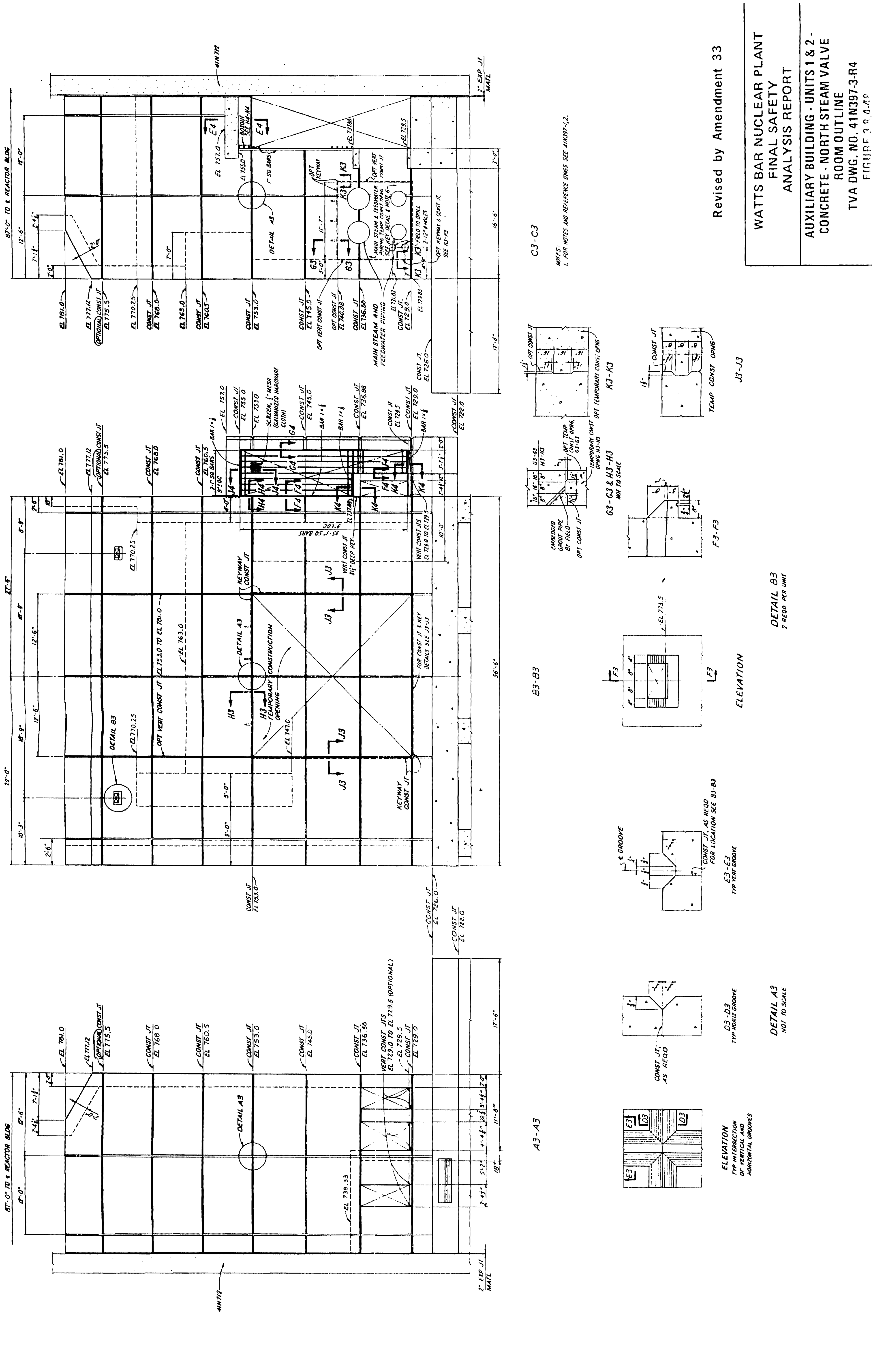


Figure 3.8.4-48 Auxiliary Building Units 1 & 2 Concrete - North Steam Valve Room Outline

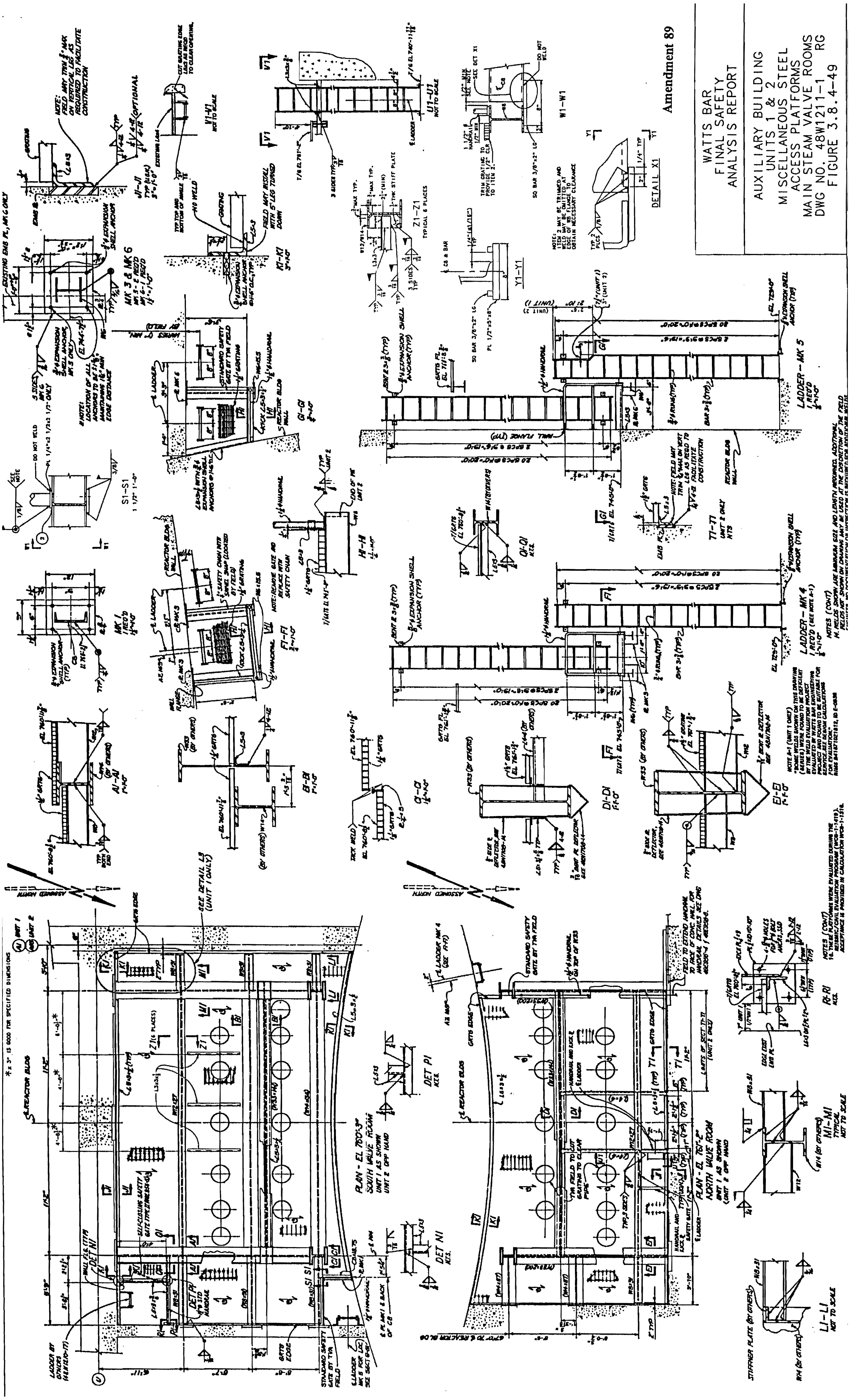


Figure 3.8.4-49 Auxiliary Building Units 1 & 2 Miscellaneous Steel Access Platforms - Main Steam Valve Rooms

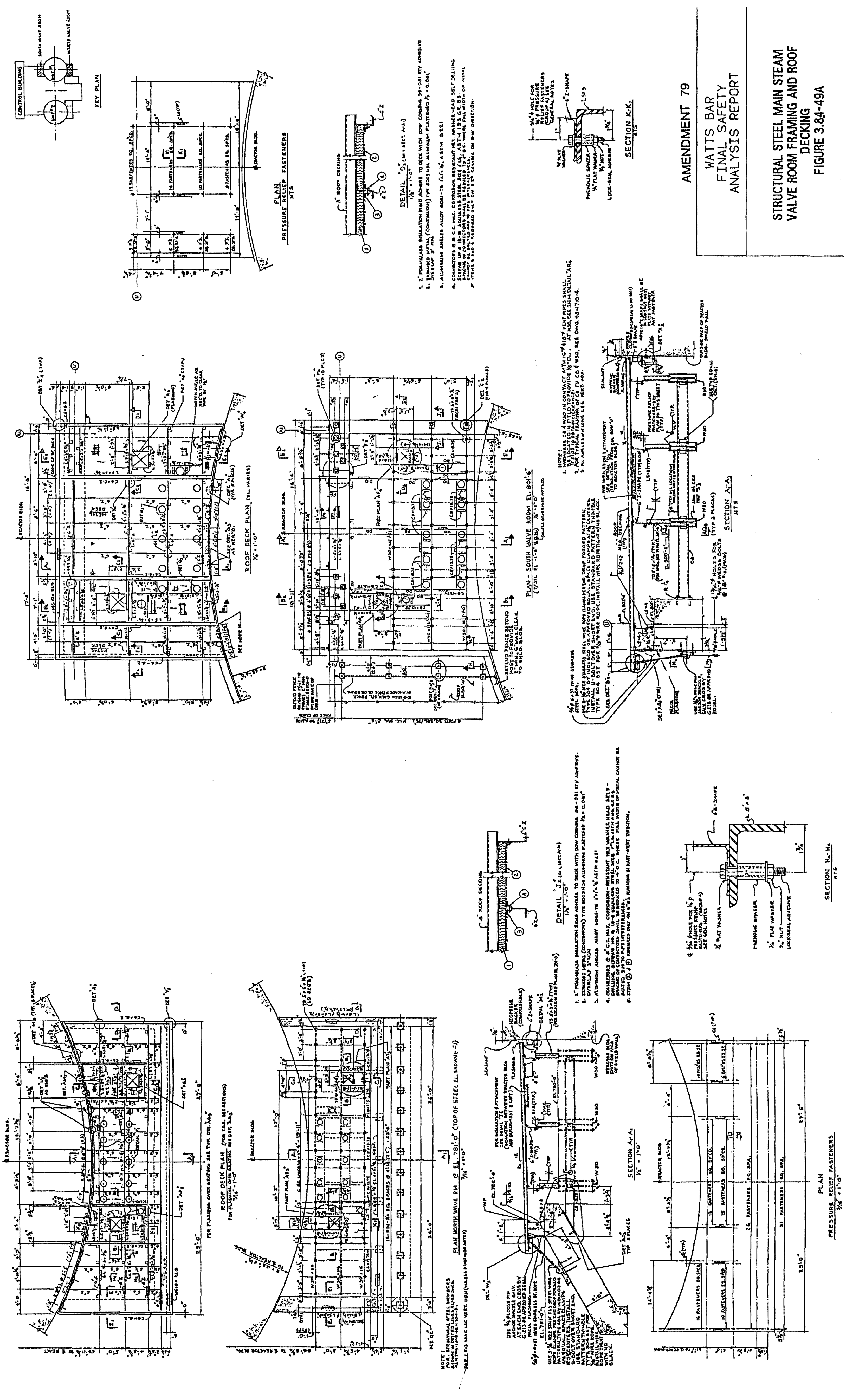


Figure 3.8.4-49a Structural Steel Main Steam Valve Room Framing and Roof Decking

Figure 3.8.4-49b Deleted by Amendment 79

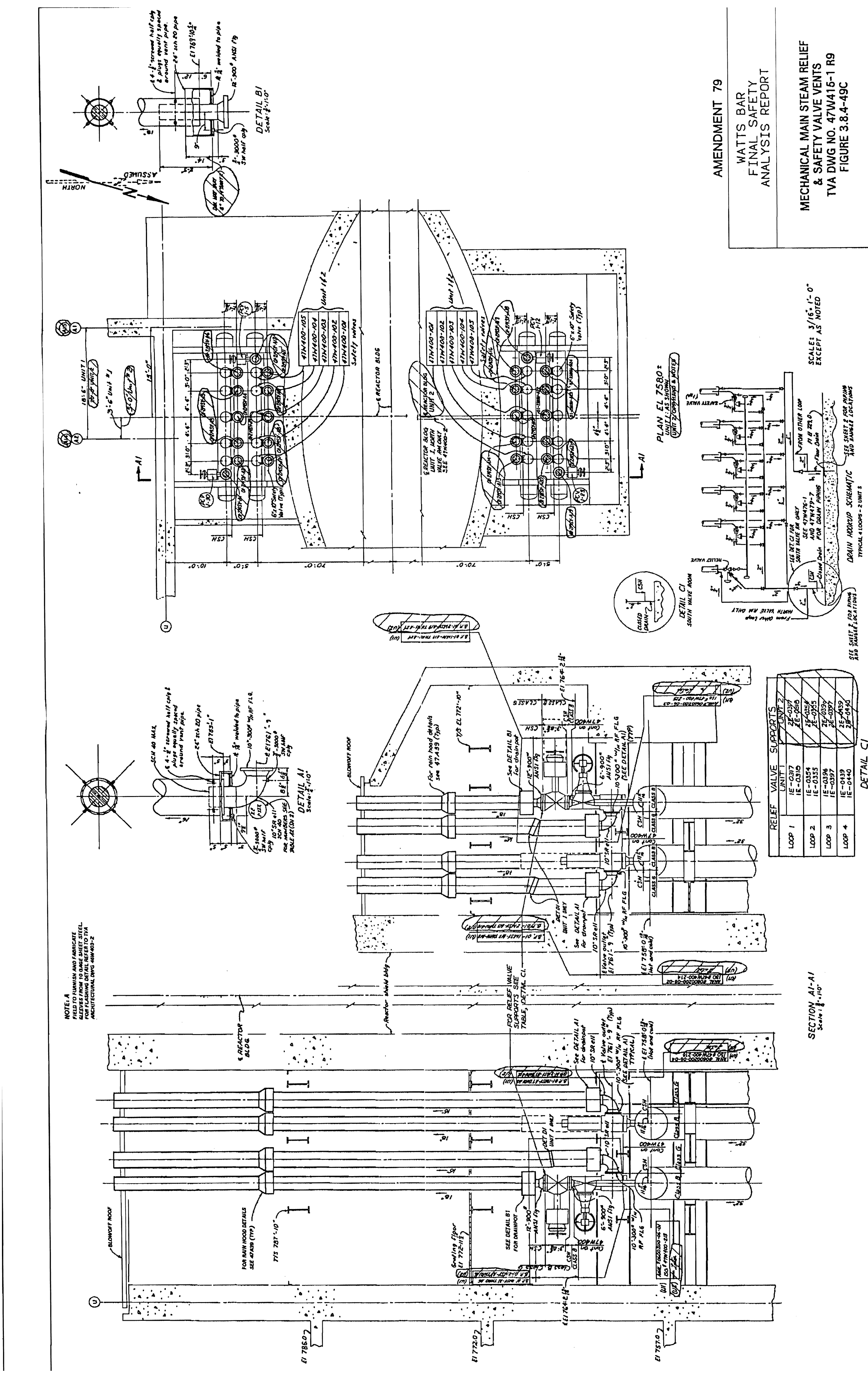


Figure 3.8.4-49c Mechanical Main Steam Relief & Safety Valve Vents

Figure 3.8.4-50 Intake Pumping Station General Outline Feature (Sheet 1)

Figure 3.8.4-51 Water Supply Units 1 & 2 Intake Pumping Station Concrete General Outline Feature (Sheet 2)

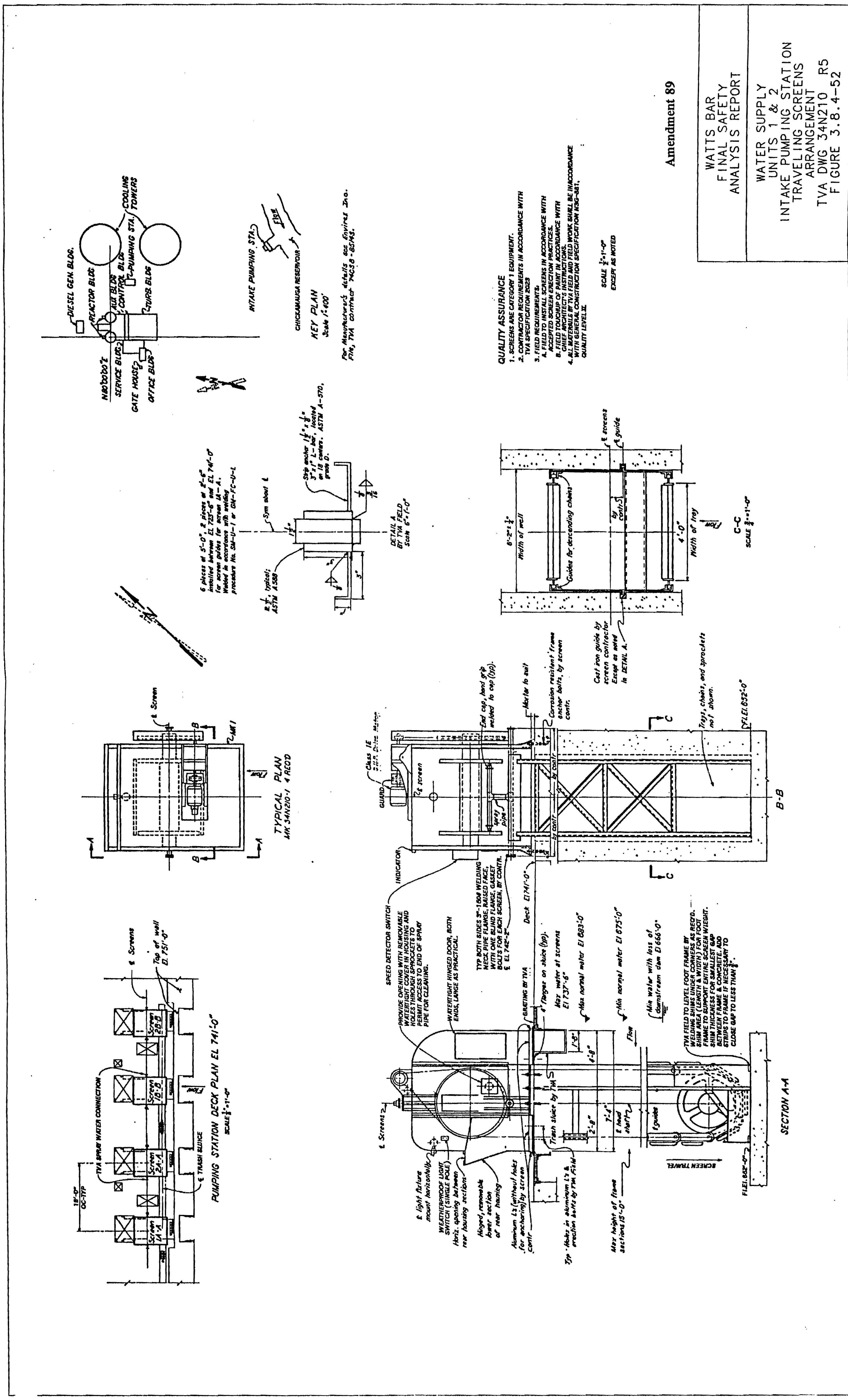


Figure 3.8.4-52 Water Supply Units 1 & 2 Intake Pumping Station Traveling Screens Arrangement

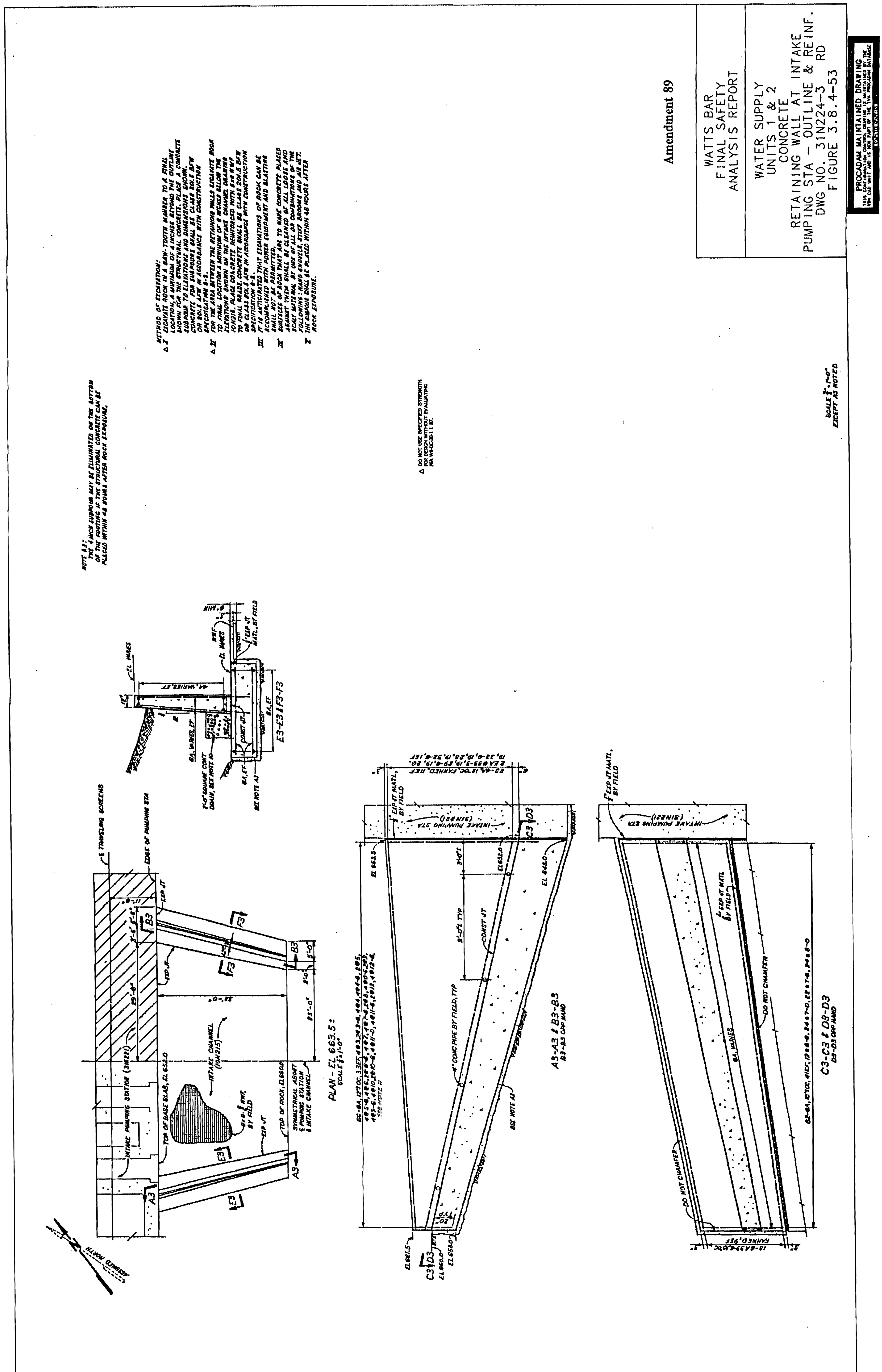


Figure 3.8.4-53 Water Supply Units 1 & 2 Concrete Retaining Wall at Intake Pumping Station - Outline and Reinforcement

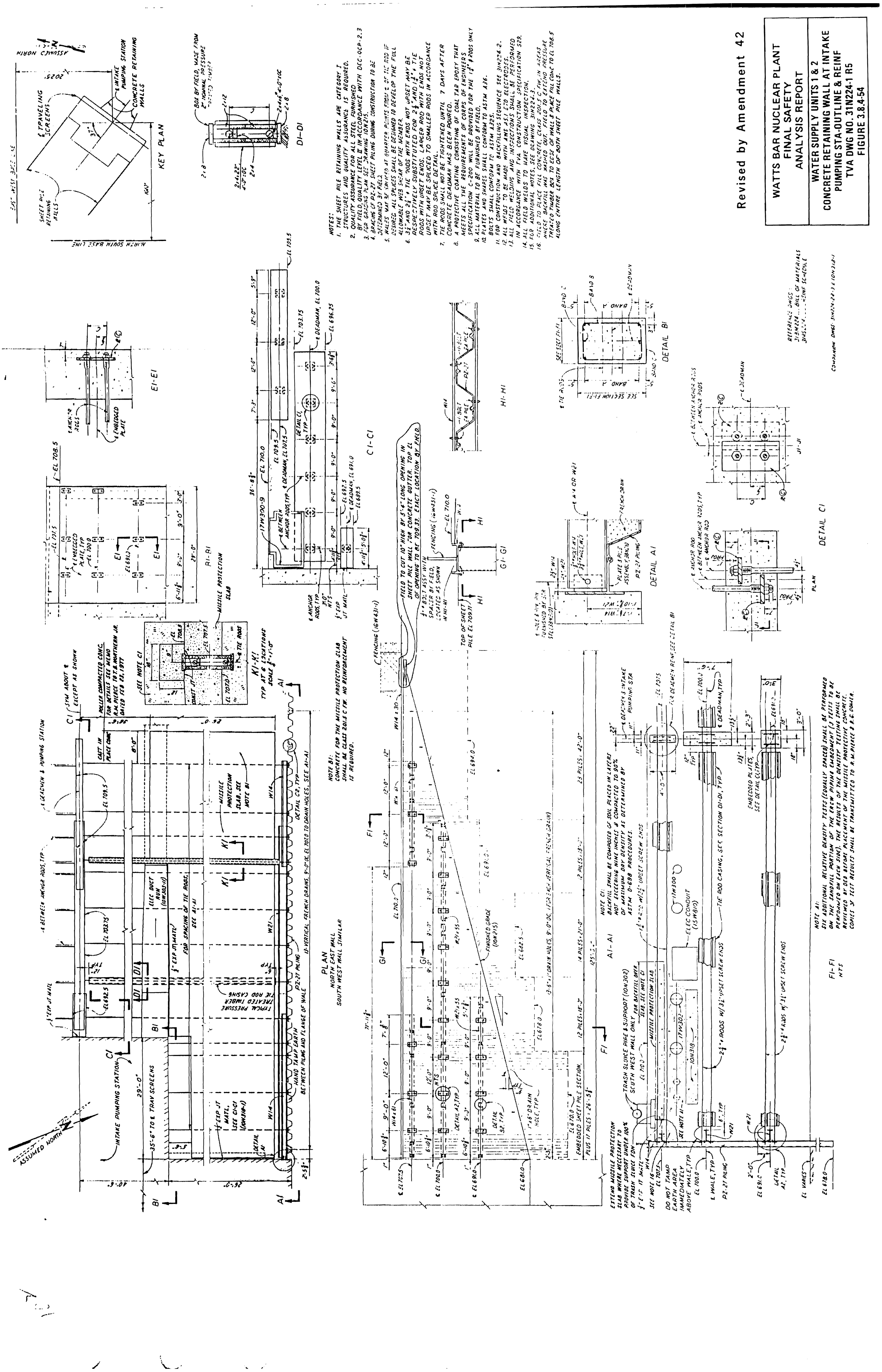


Figure 3.8.4-54 Water Supply Units 1 & 2 Concrete Retaining Wall at Intake Pumping Station - Outline and Reinforcement

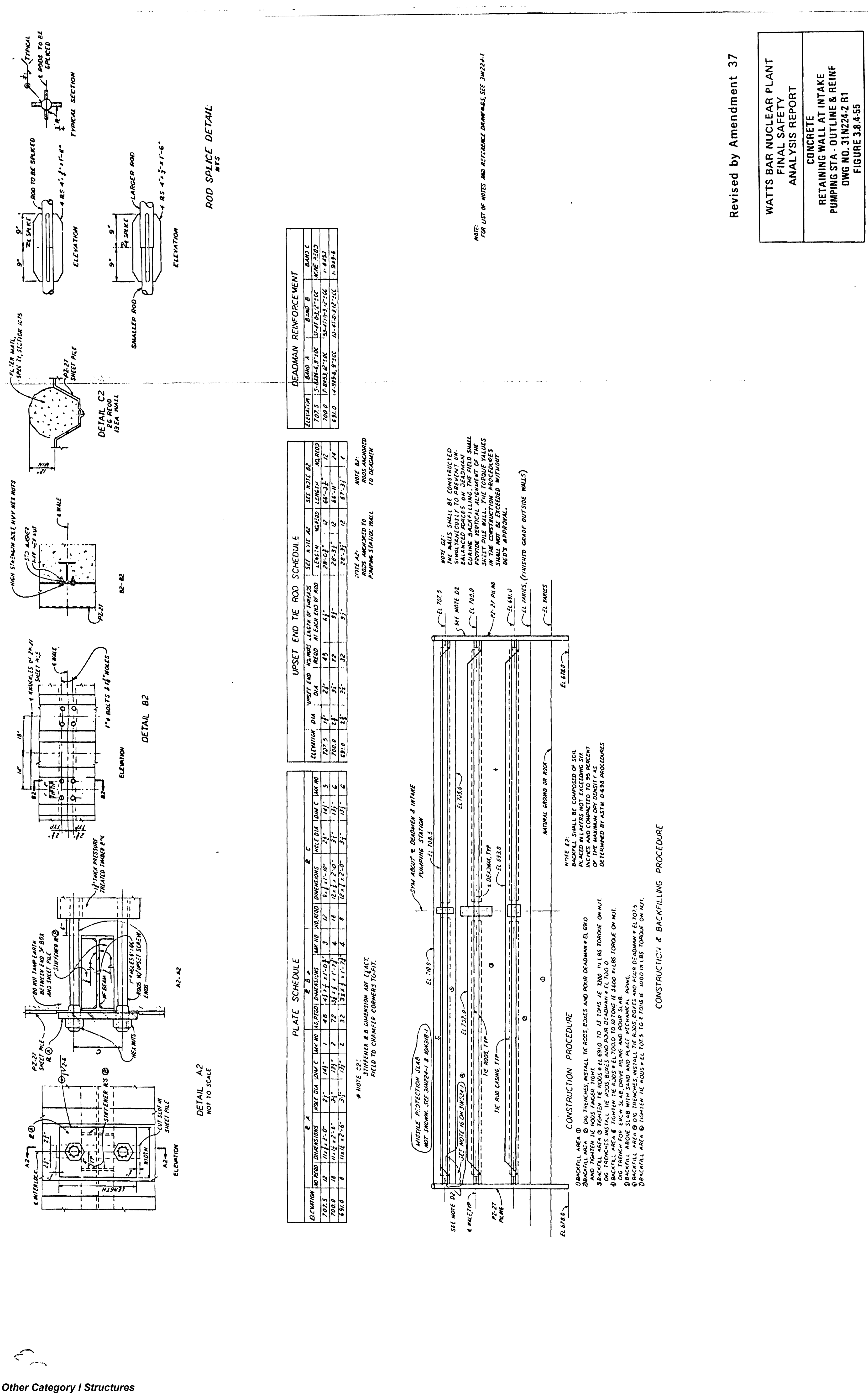


Figure 3.8.4-55 Concrete - Retaining Wall at Intake Pumping Station - Outline and Reinforcement

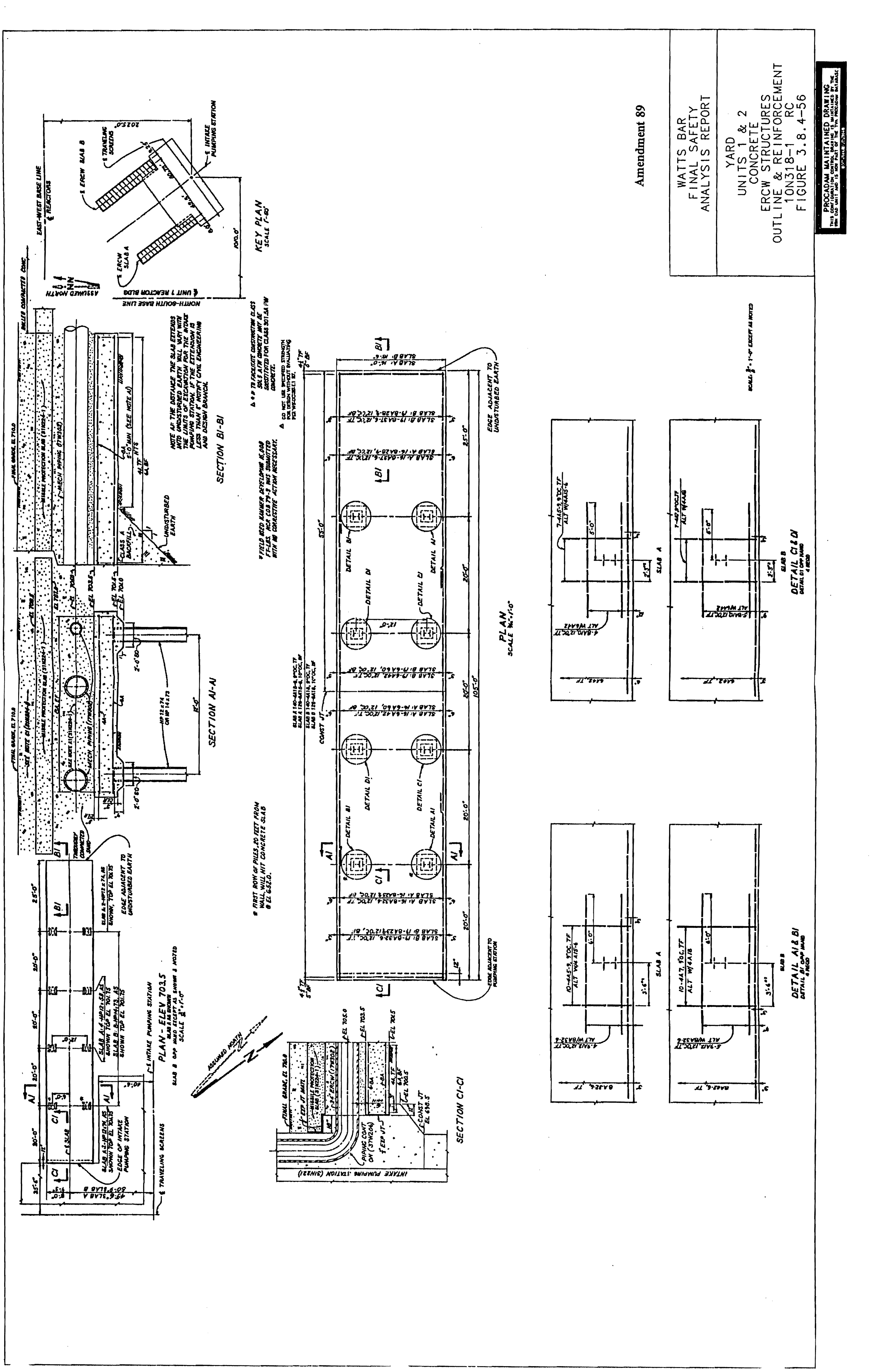


Figure 3.8.4-56 Yard Units 1 & 2 Concrete ERCW Structures Outline & Reinforcement

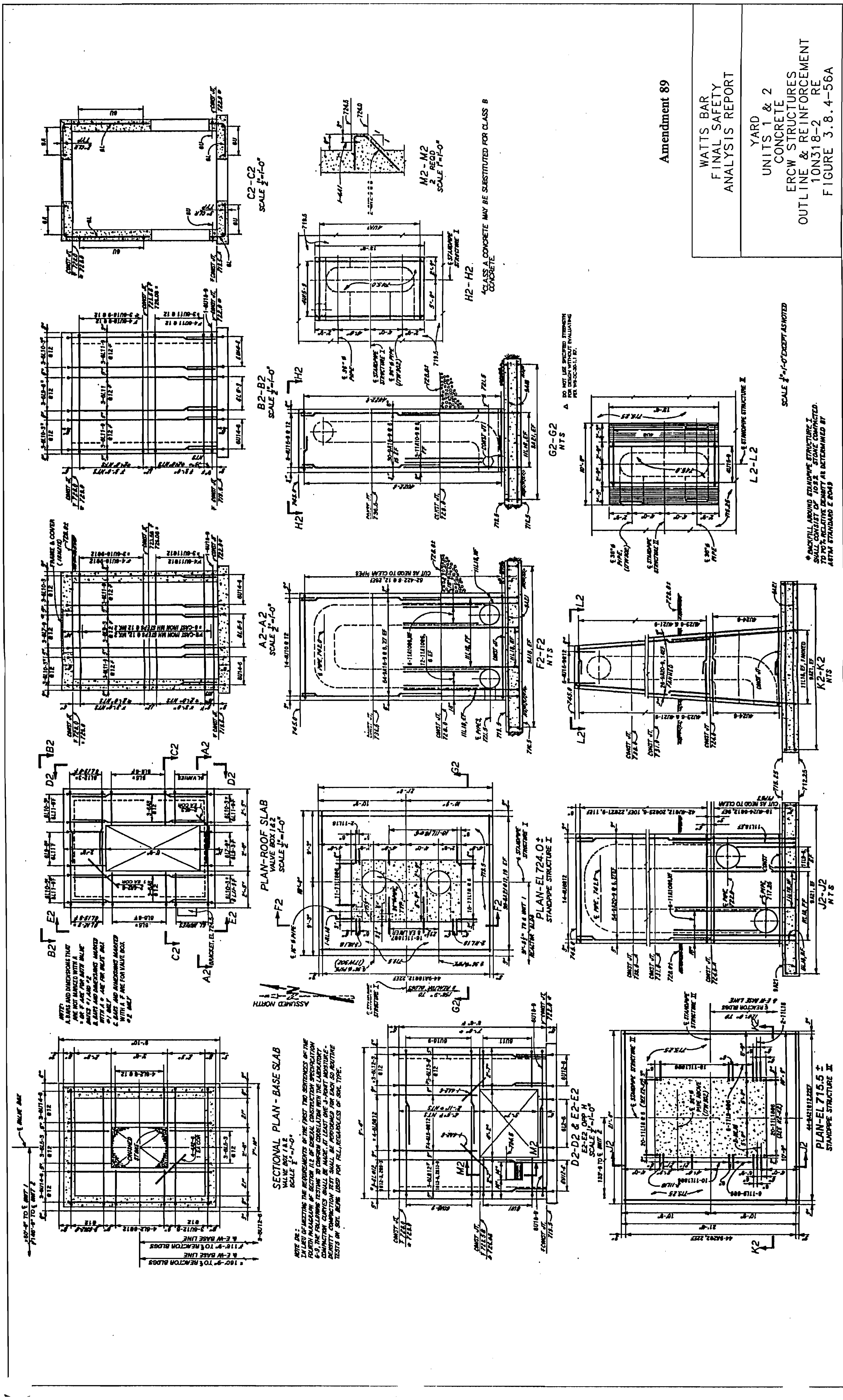


Figure 3.8.4-56a Yard Units 1 & 2 Concrete ERCW Structures Outline & Reinforcement

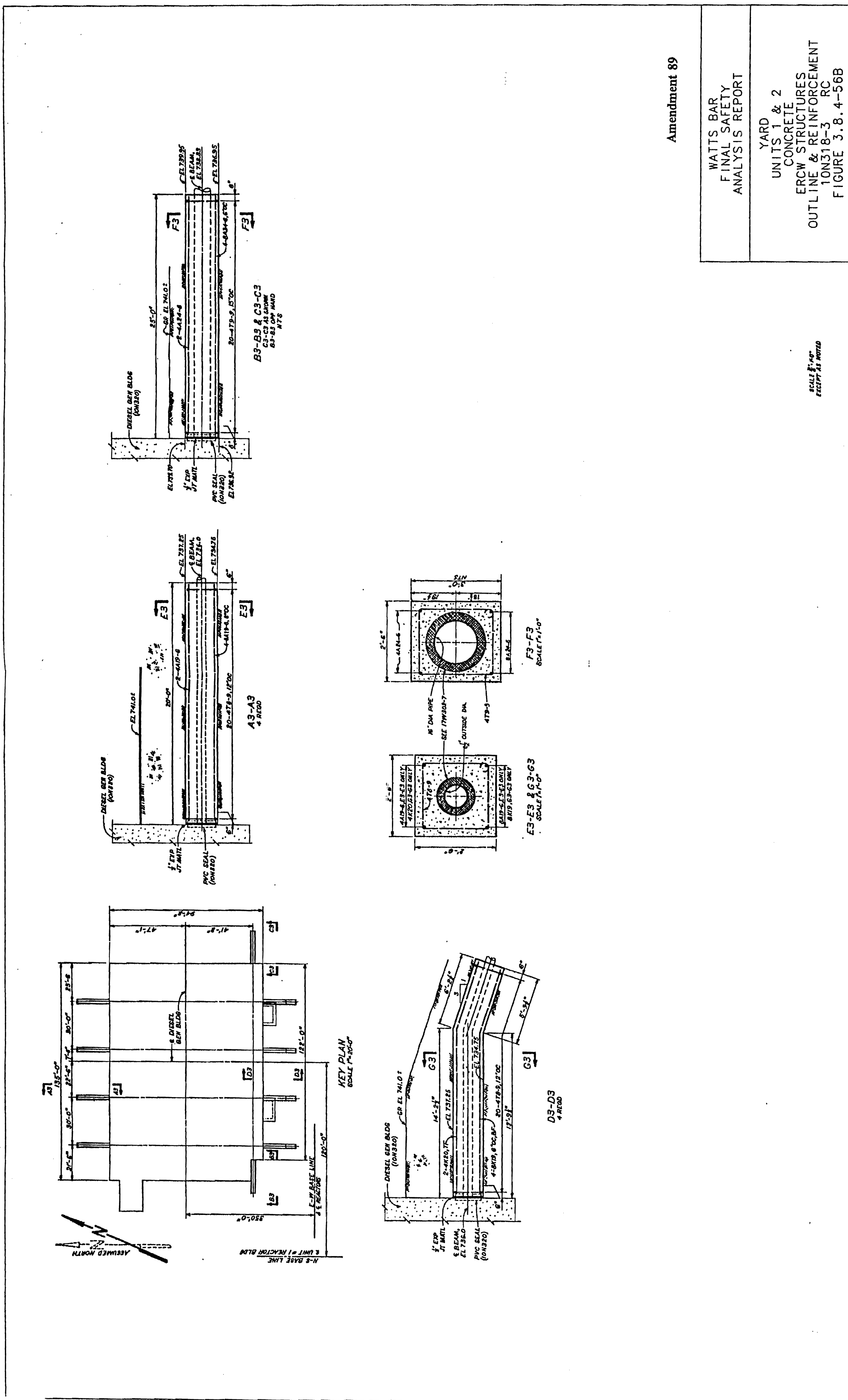


Figure 3.8.4-56b Yard Units 1 & 2 Concrete ERCW Structures Outline & Reinforcement

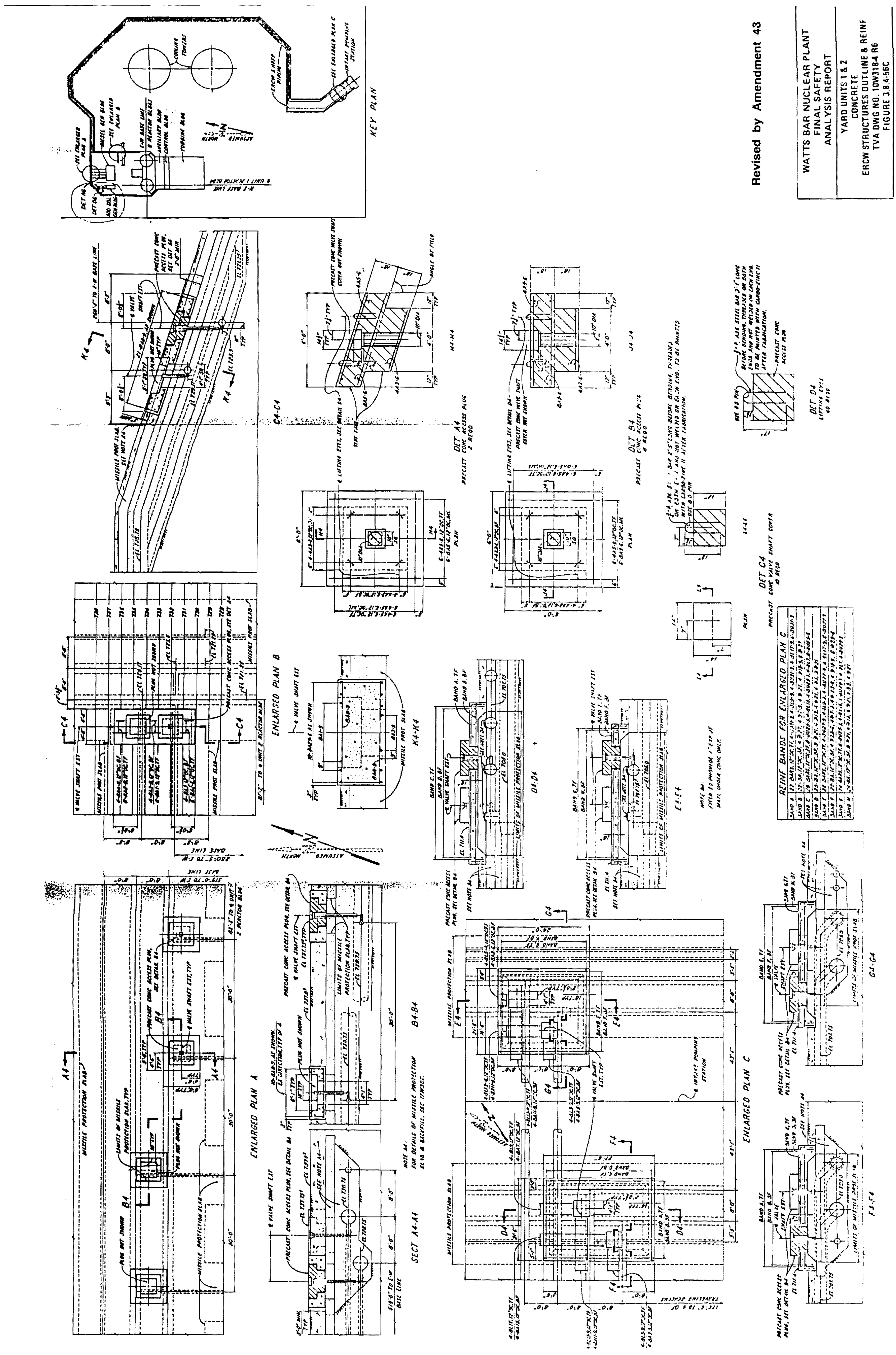


Figure 3.8.4-56c Yard Units 1 & 2 Concrete ERCW Structures Outline & Reinforcement

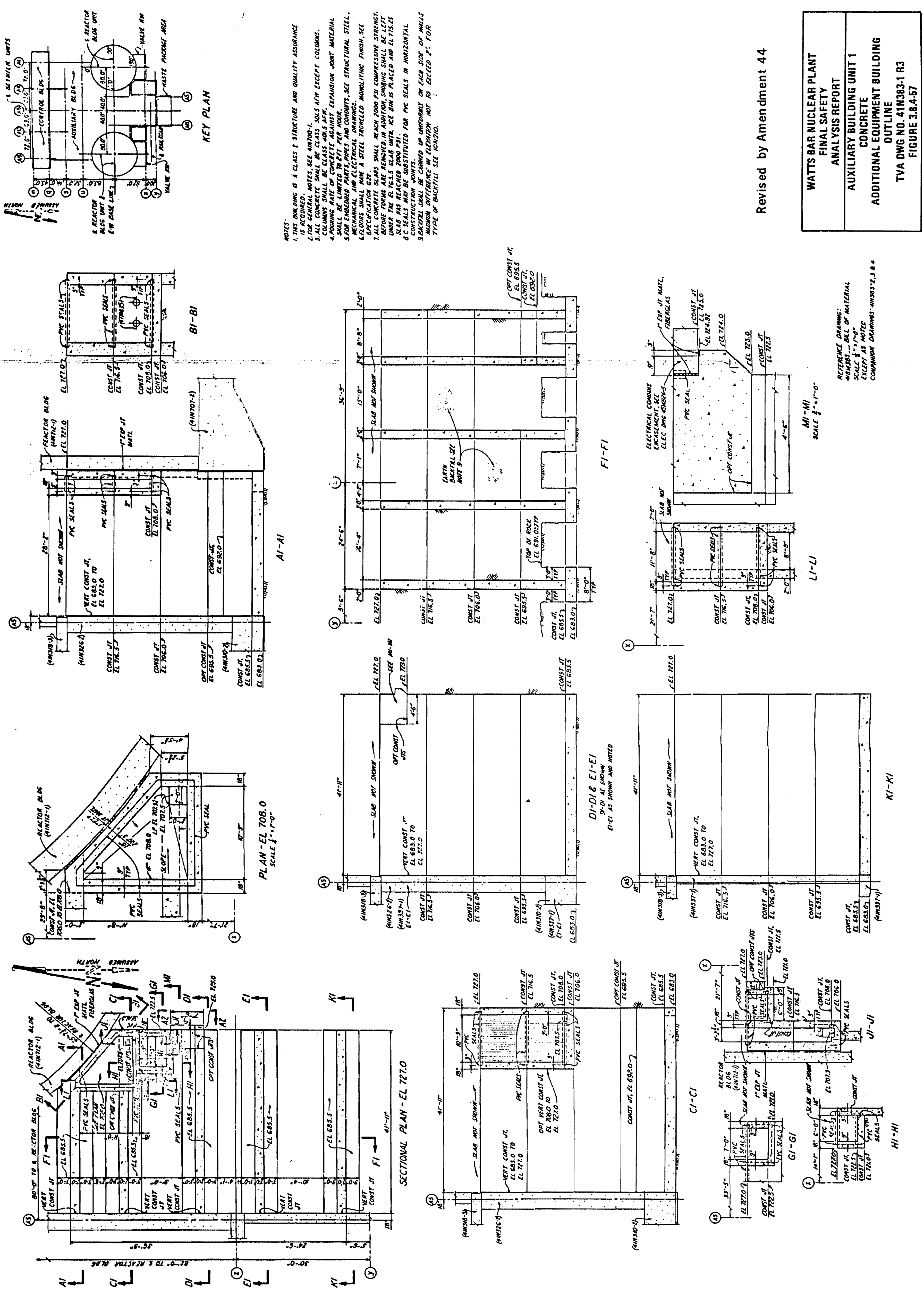


Figure 3.8.4-57 Auxiliary Building Unit 1 Concrete Additional Equipment Building Outline (Sheet 1)

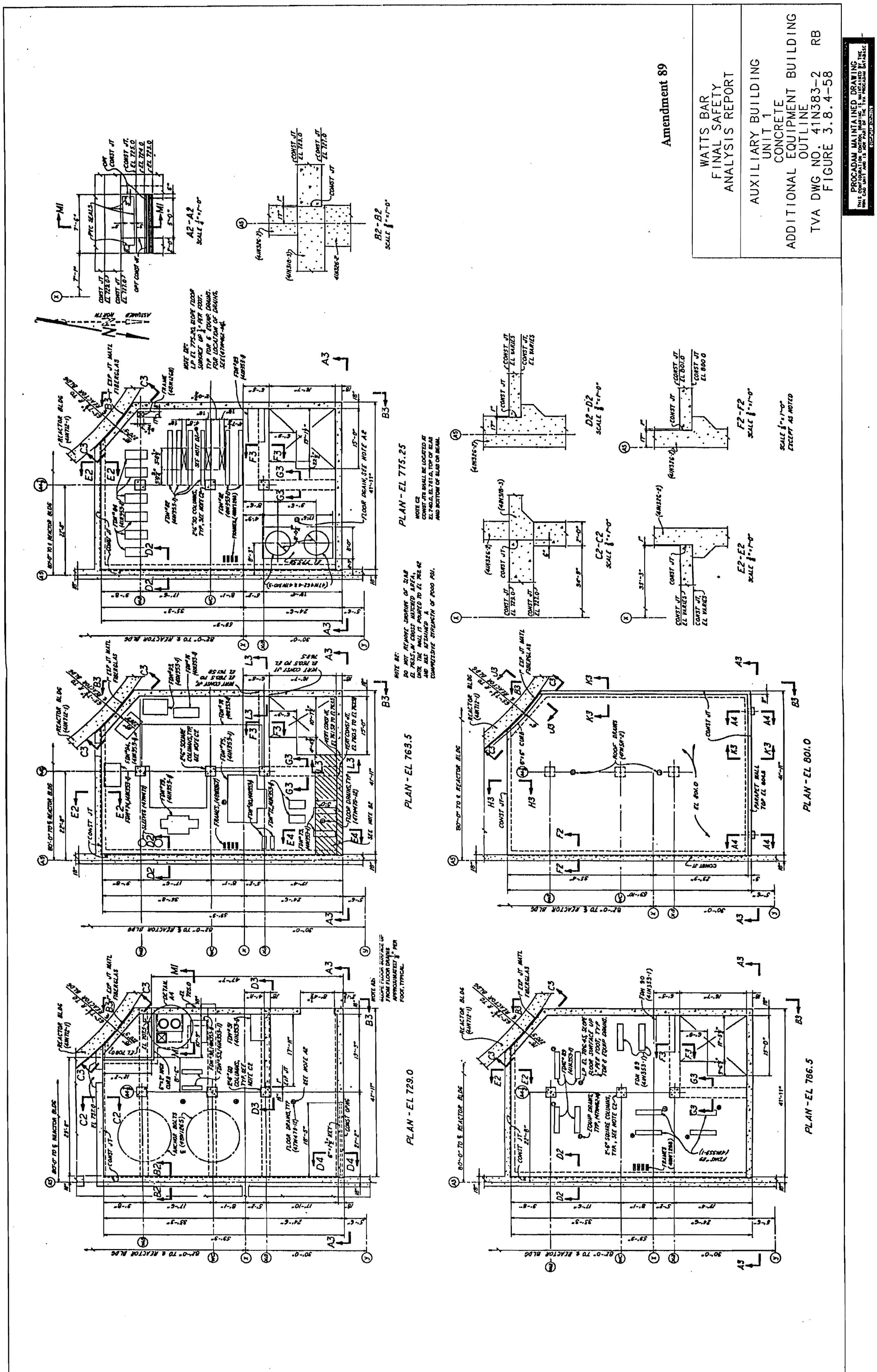


Figure 3.8.4-58 Auxiliary Building Unit 1 Concrete Additional Equipment Building Outline (Sheet 2)

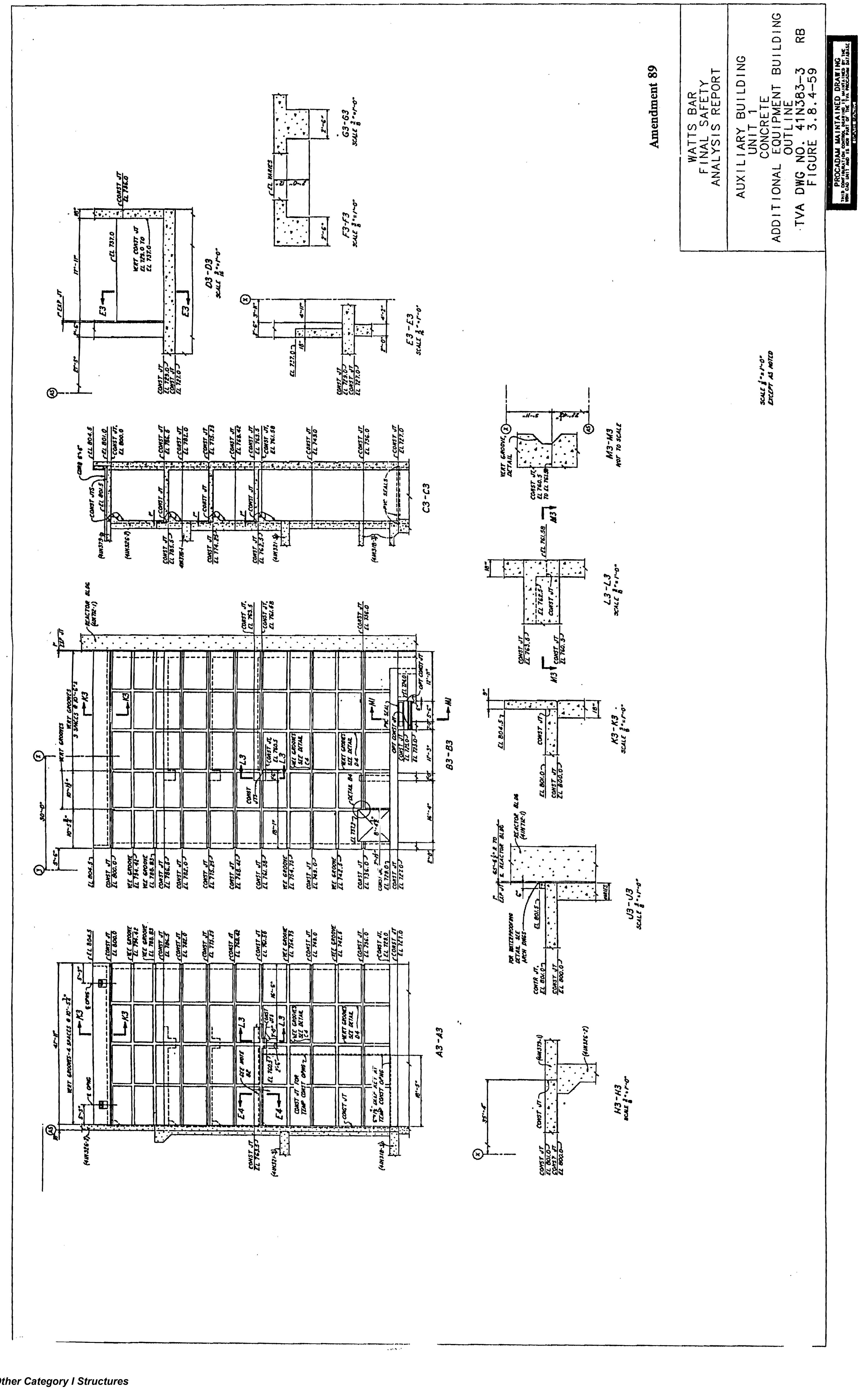


Figure 3.8.4-59 Auxiliary Building Unit 1 Concrete Additional Equipment Building Outline (Sheet 3)

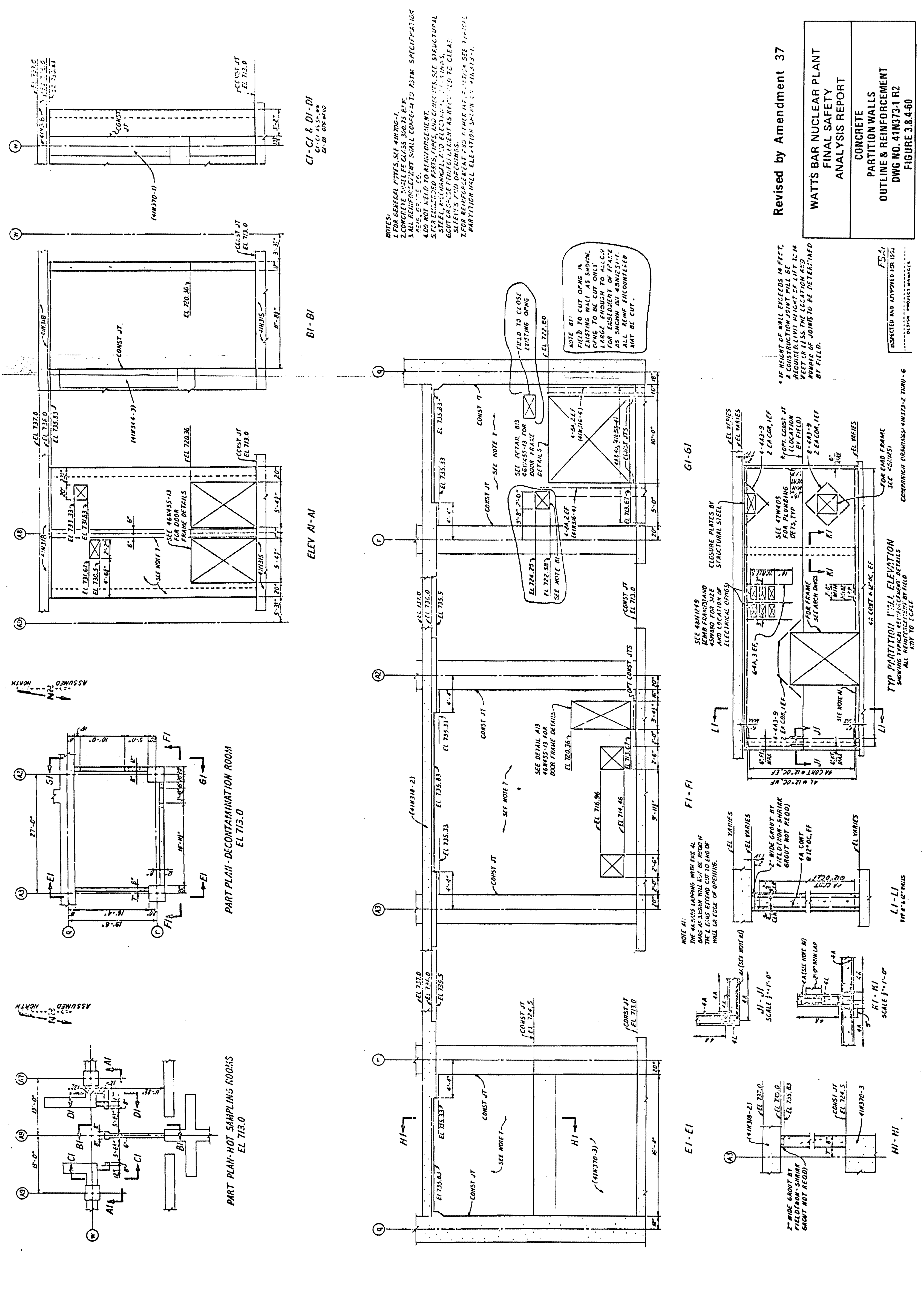


Figure 3.8.4-60 Concrete Partition Walls Outline and Reinforcement (Sheet 1)

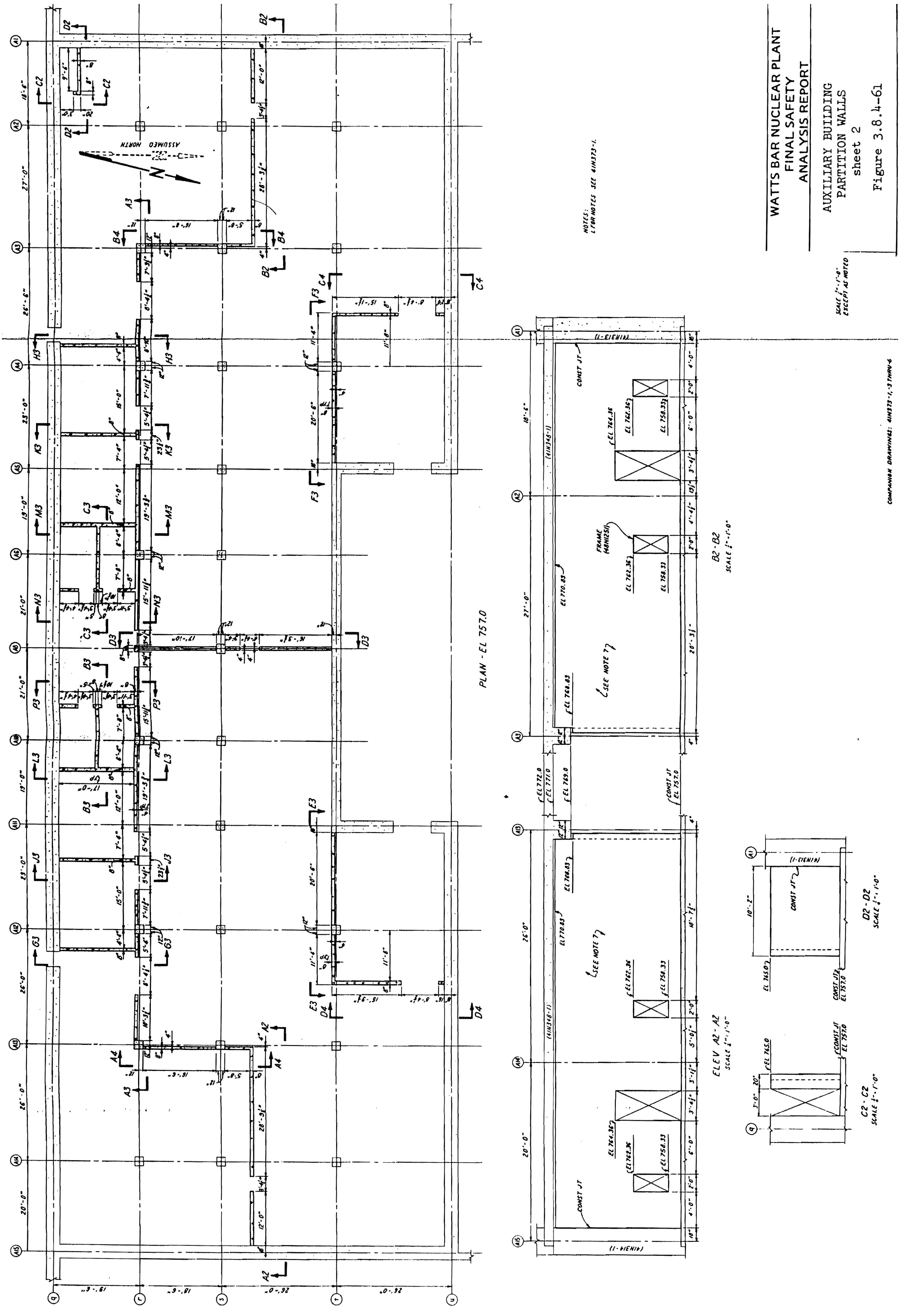


Figure 3.8.4-61 Auxiliary Building Partition Walls (Sheet 2)

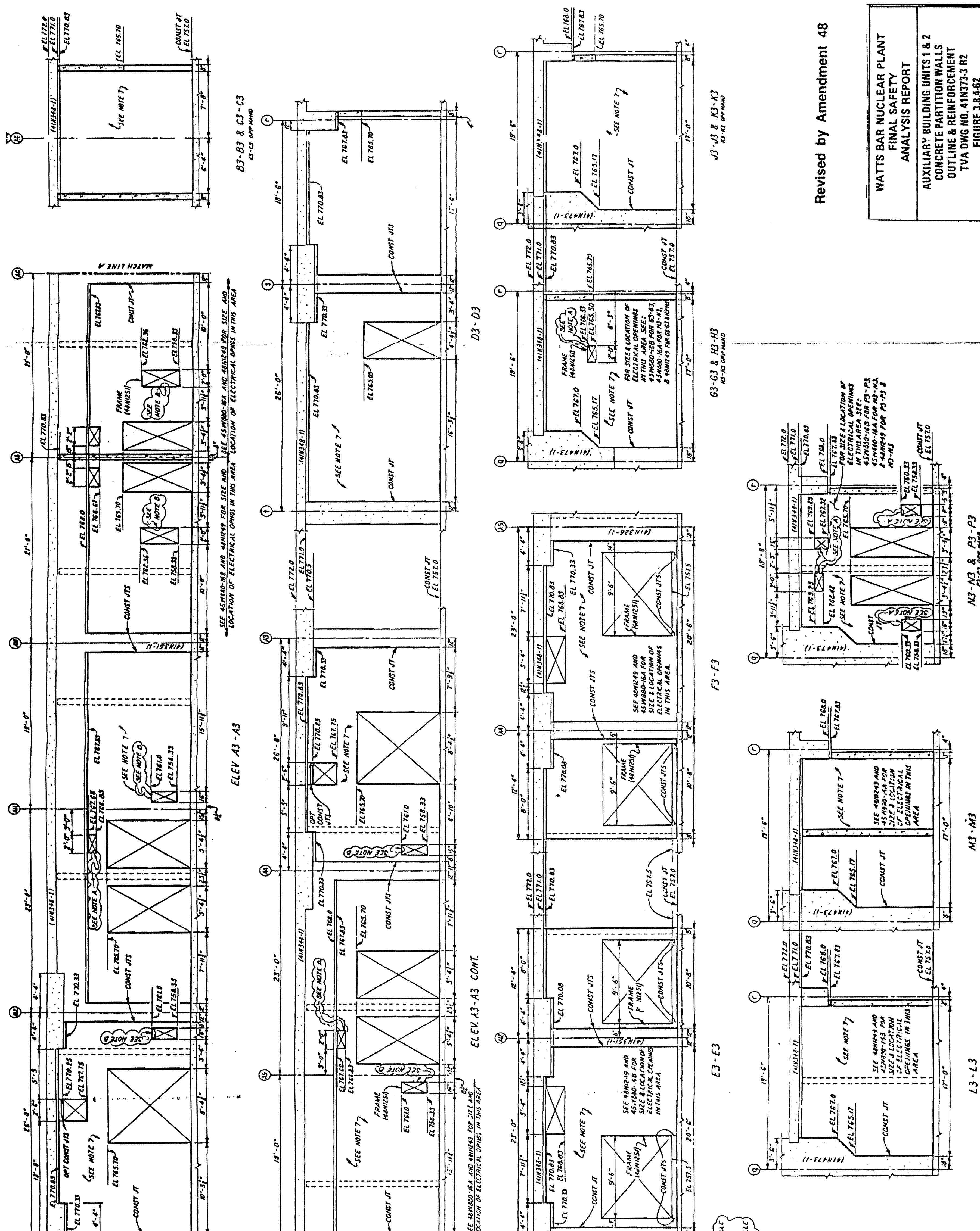
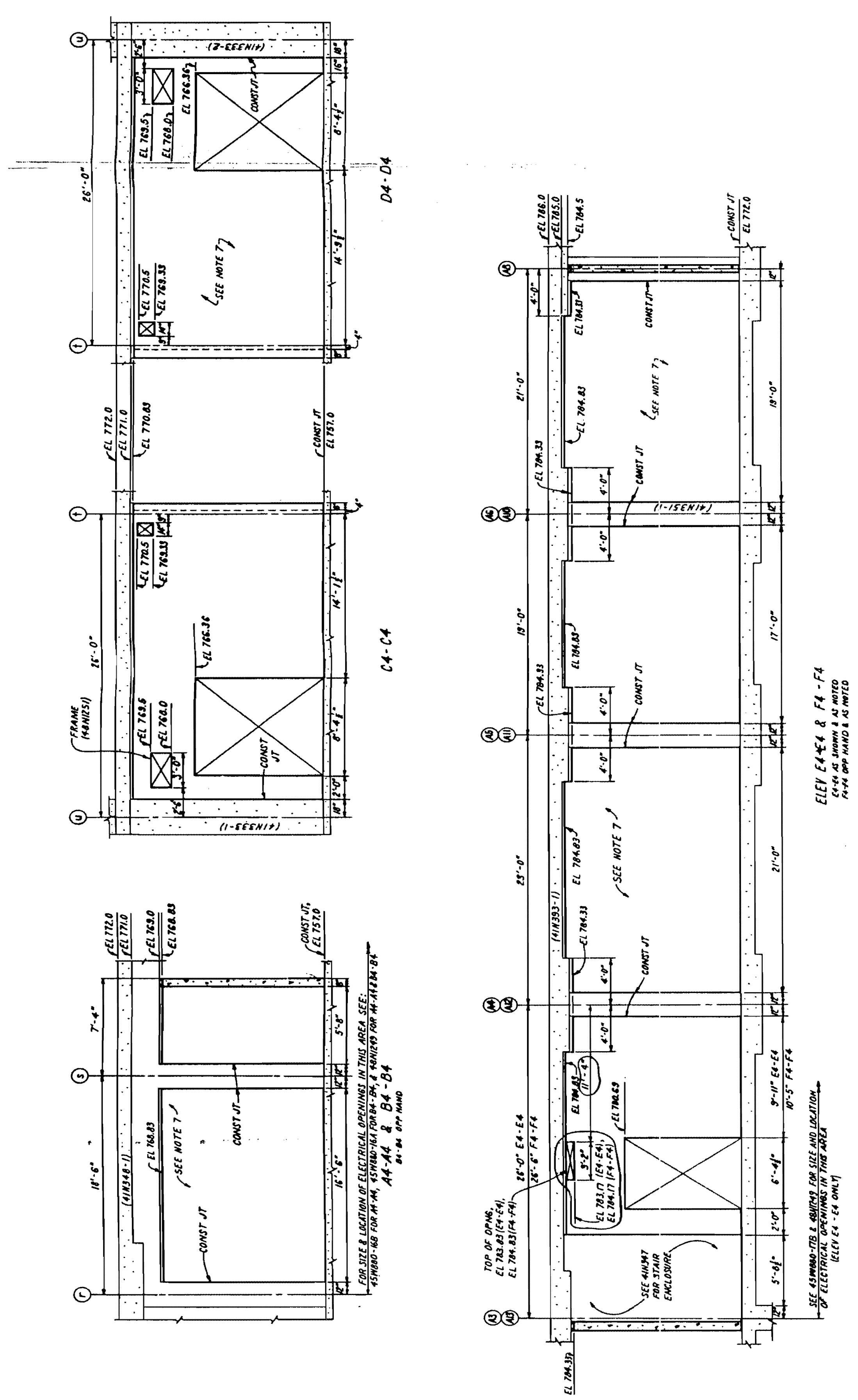


Figure 3.8.4-62 Auxiliary Building Units 1 & 2 Concrete Partition Walls Outline and Reinforcement



Revised by Amendment 33

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

AUXILIARY BUILDING - UNITS 1 & 2
CONCRETE PARTITION WALLS
OUTLINE & REINFORCEMENT
TVA DWG. NO. 41N373-4-R1
FIGURE 3.8.4-63

Figure 3.8.4-63 Auxiliary Building Units 1 & 2 Concrete Partition Walls Outline and Reinforcement

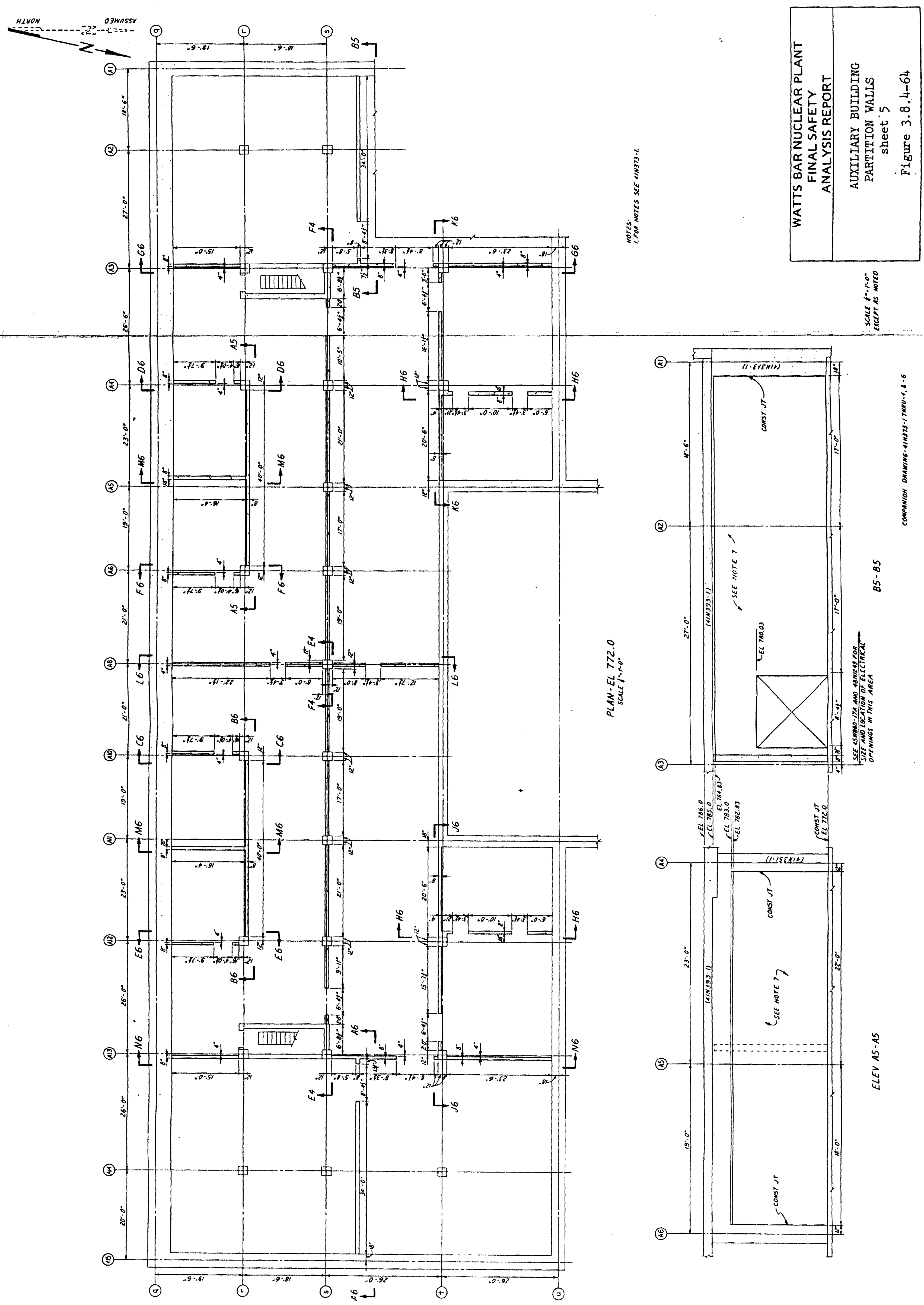
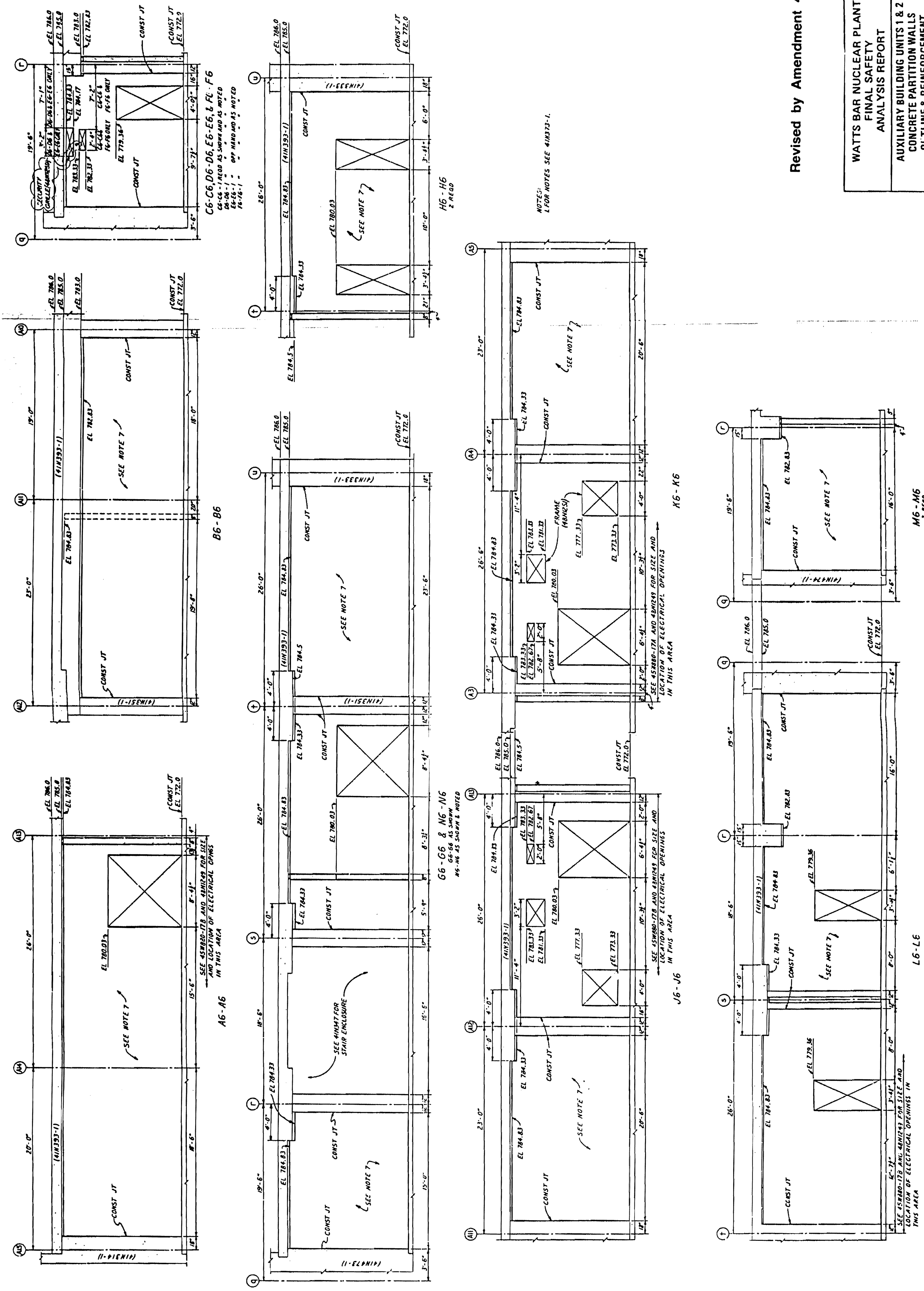


Figure 3.8.4-64 Auxiliary Building Partition Walls (Sheet 5)



Revised by Amendment 48

WATTS BAR NUCLEAR PLANT
FINAL SAFETY
ANALYSIS REPORT

AUXILIARY BUILDING UNITS 1 & 2
CONCRETE PARTITION WALLS
OUTLINE & REINFORCEMENT
TVA DWG NO. 41N373-6 R3
FIGURE 3.8.4-65

Figure 3.8.4-65 Auxiliary Building Units 1 & 2 Concrete Partition Walls Outline and Reinforcement

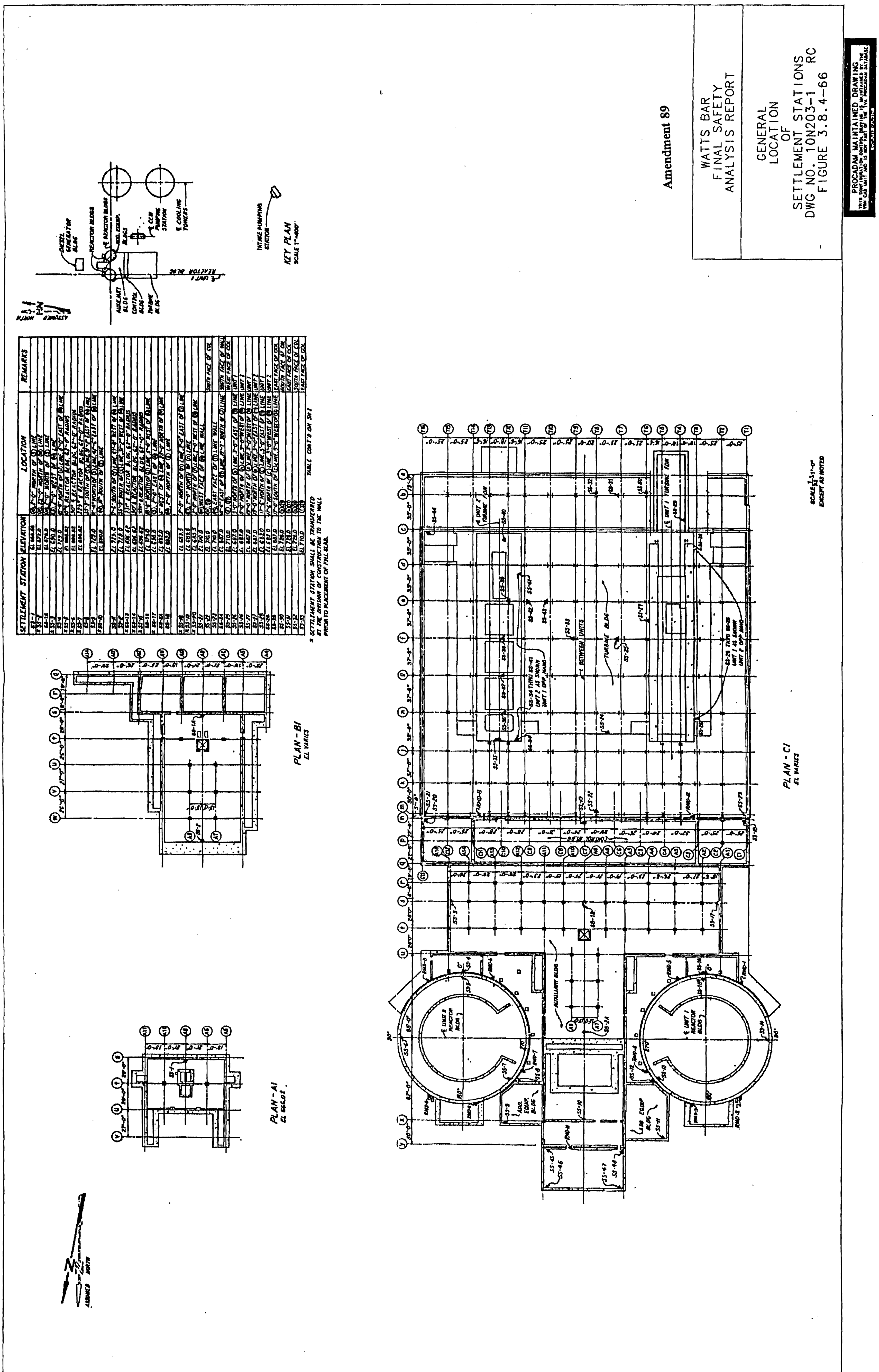


Figure 3.8.4-66 General Location of Settlement Stations

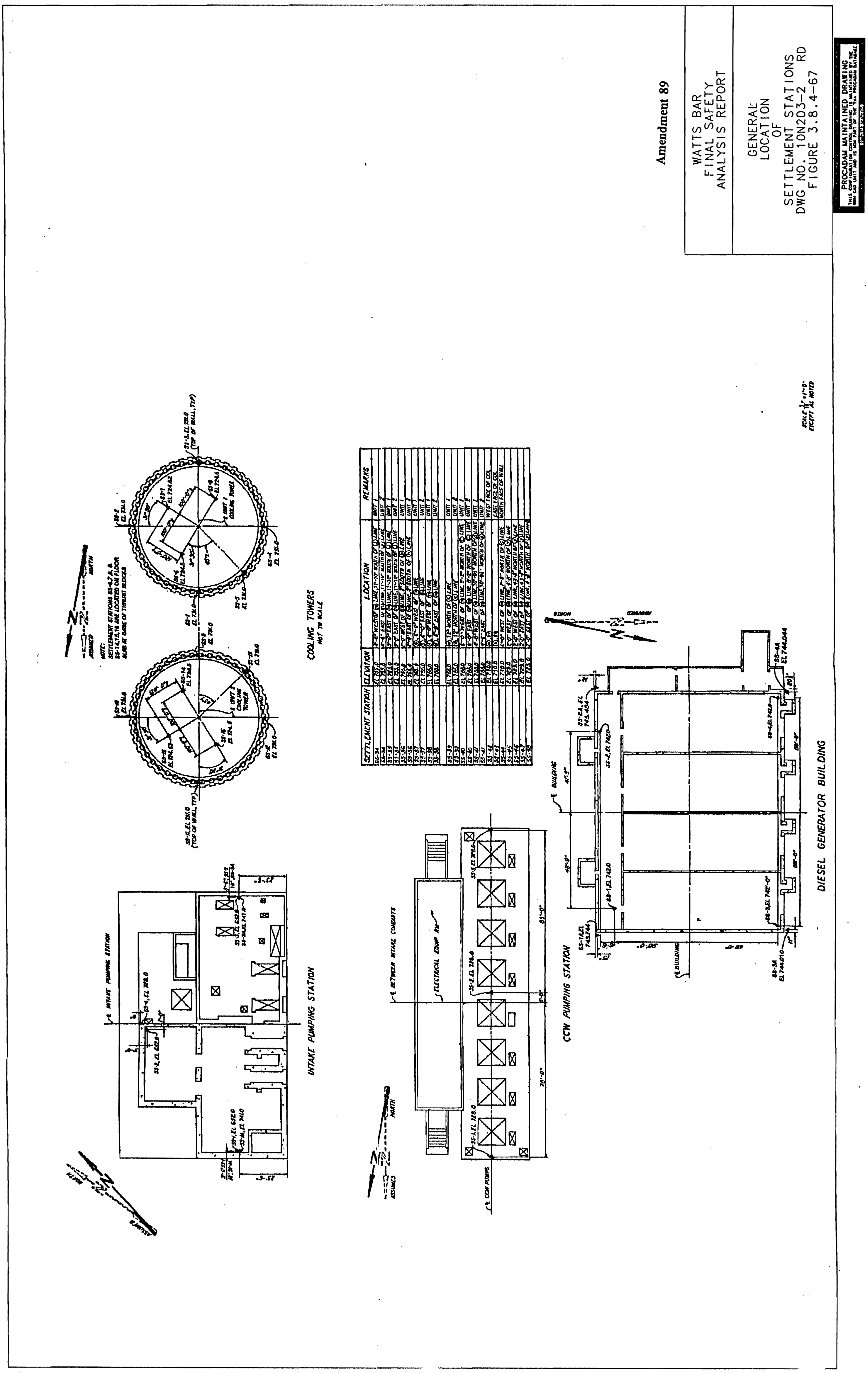


Figure 3.8.4-67 General Location of Settlement Stations

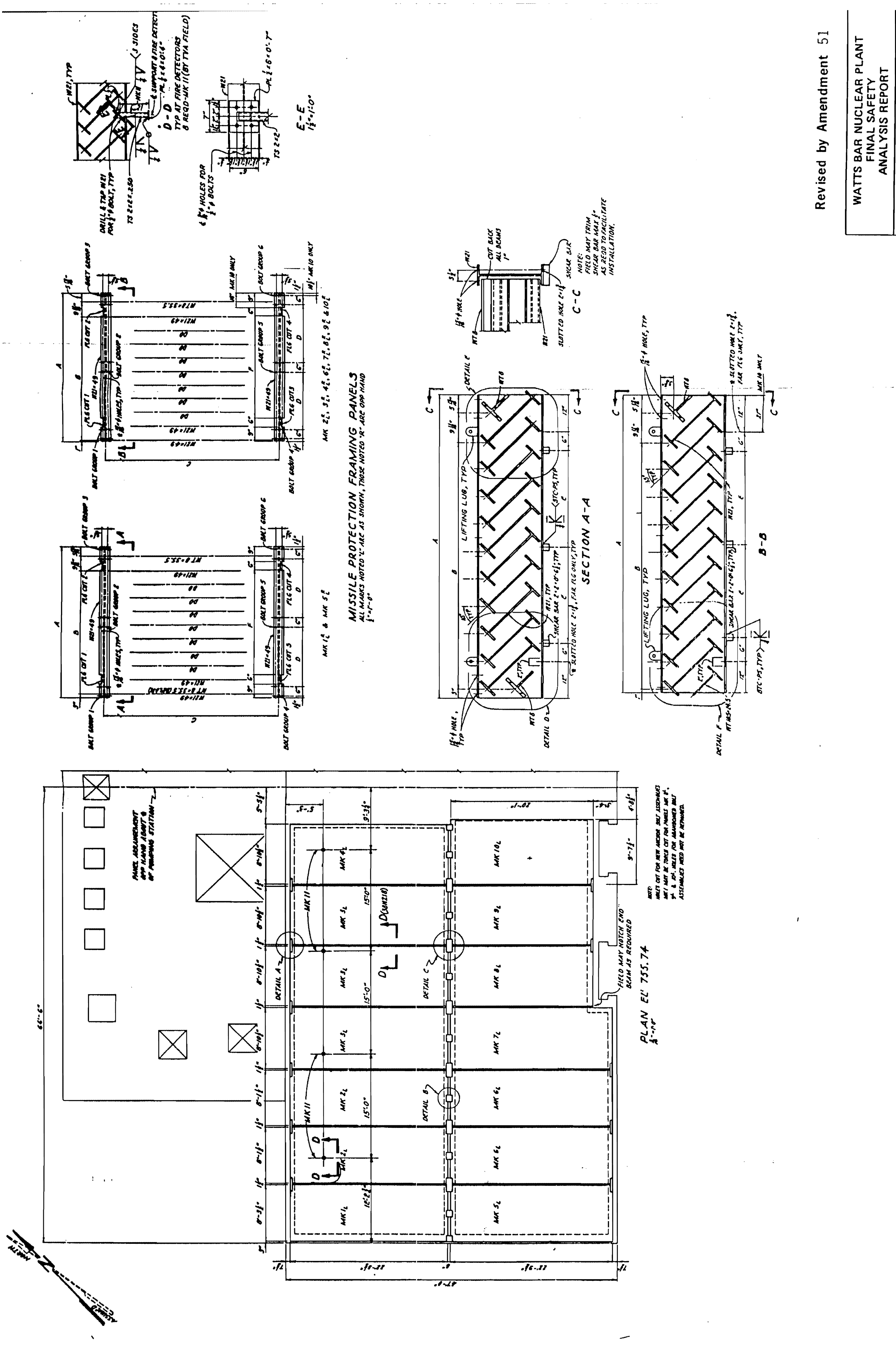


Figure 3.8.4-68 Structural Steel ERCW Intake Pumping Station Missile Protection Framing (Sheet 2)

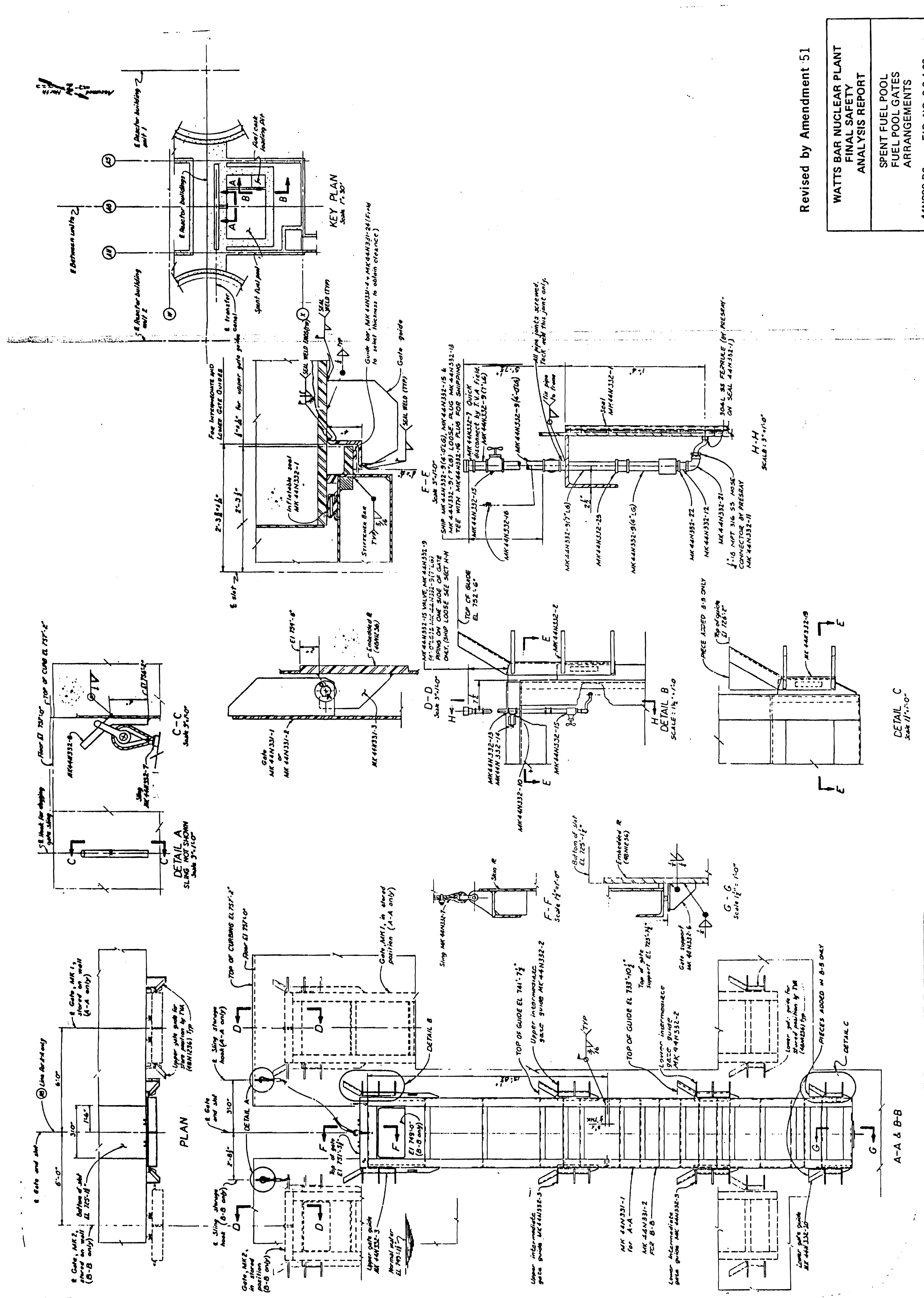


Figure 3.8.4-69 Spent Fuel Pool Fuel Pool Gates Arrangements

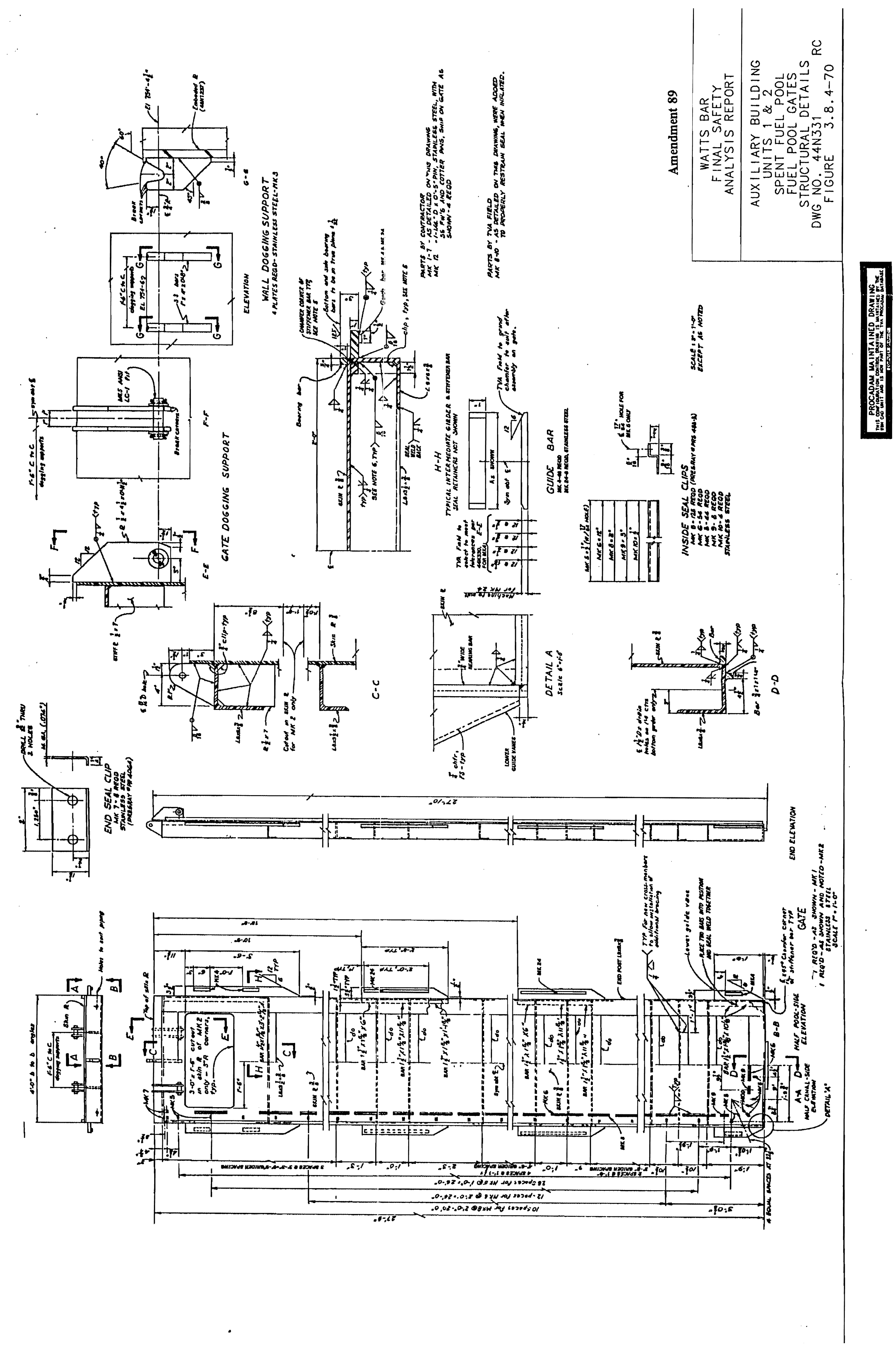


Figure 3.8.4-70 Auxiliary Building Units 1 & 2 Spent Fuel Pool Fuel Pool Gates Structural Details

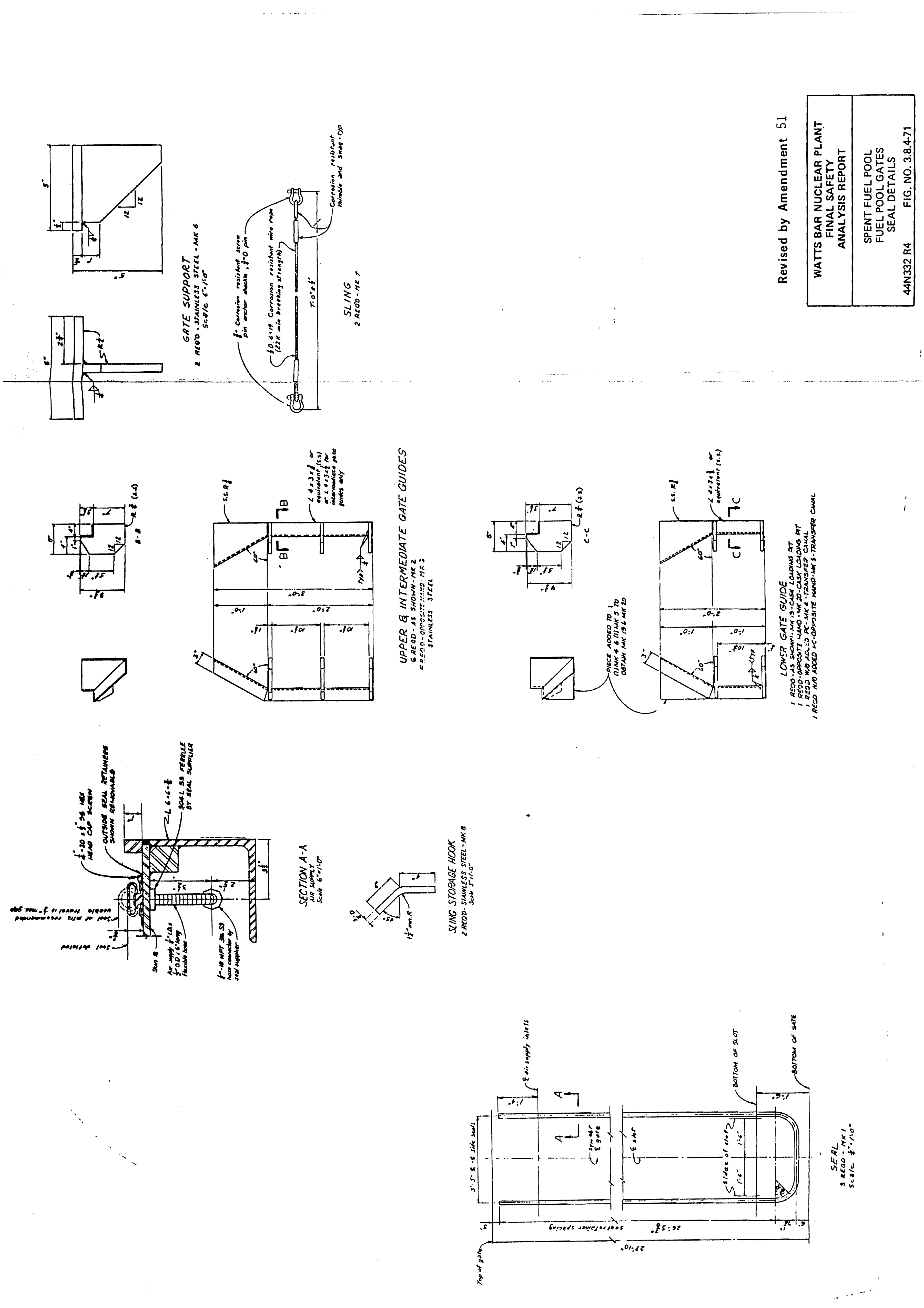


Figure 3.8.4-71 Spent Fuel Pool Fuel Pool Gates Seal Details

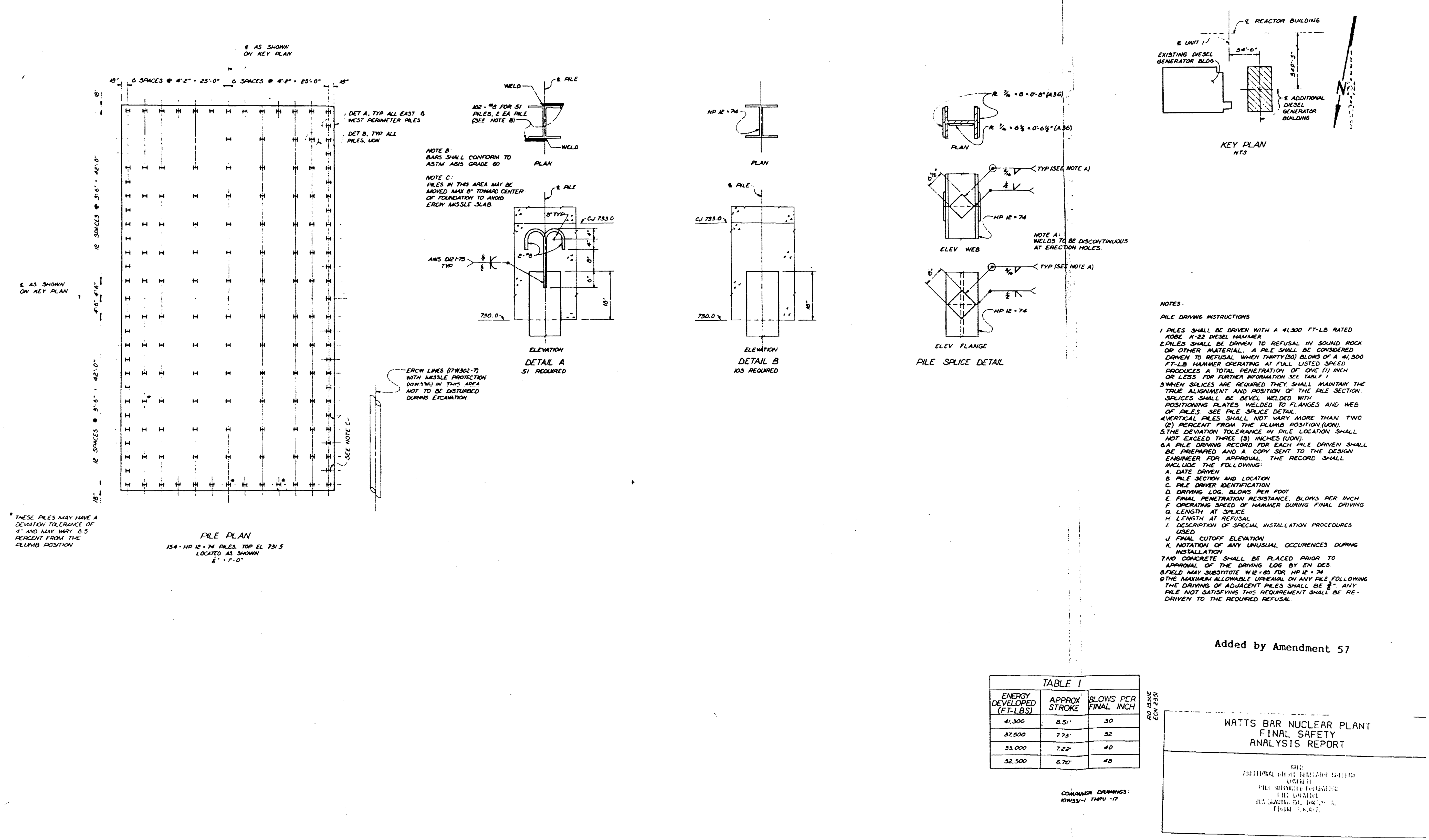


Figure 3.8.4-72 Additional Diesel Generator Building Concrete Pile Supported Foundation

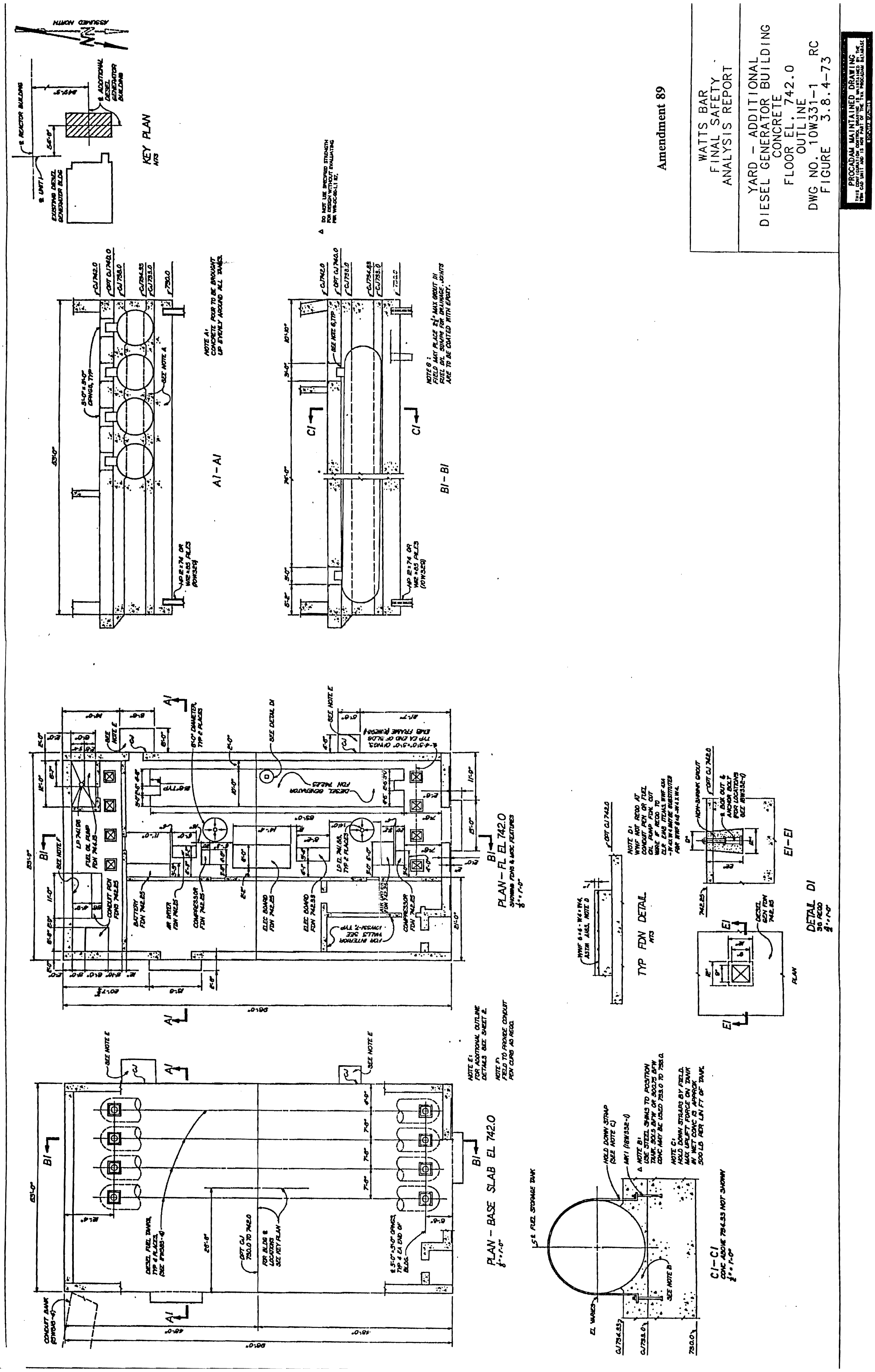


Figure 3.8.4-73 Yard - Additional Diesel Generator Building Concrete Floor EL. 742.0 Outline

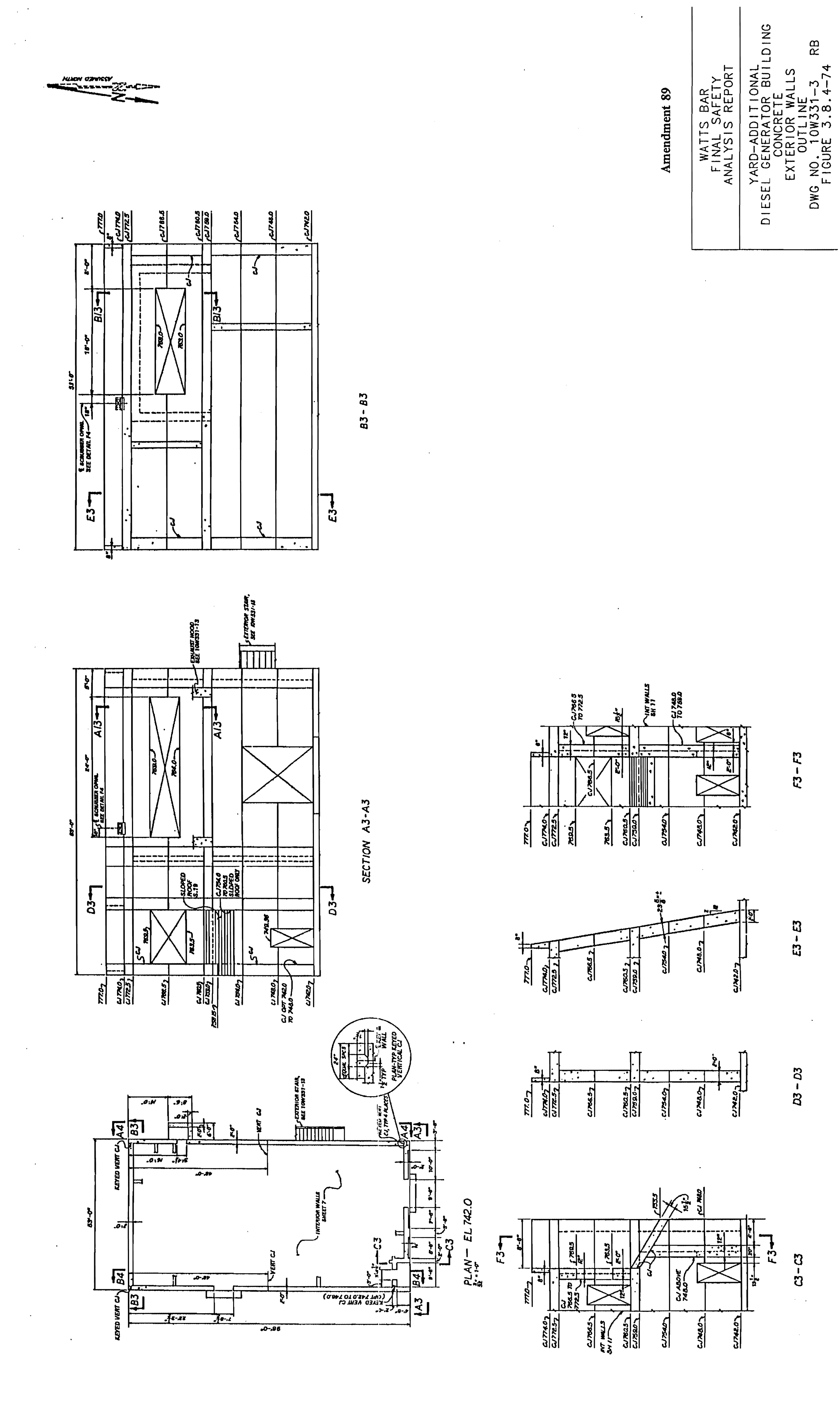
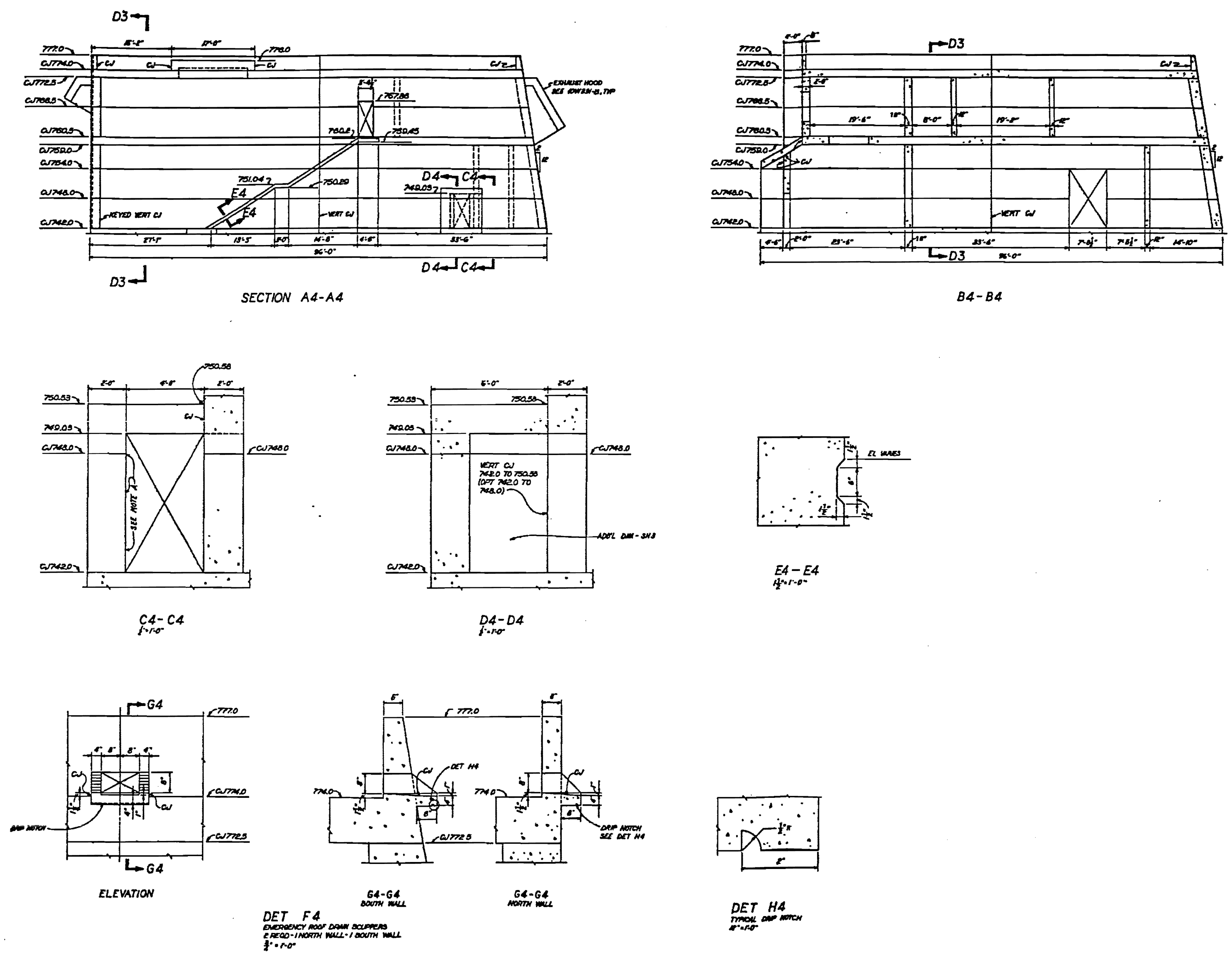


Figure 3.8.4-74 Yard Additional Diesel Generator Building Concrete Exterior Walls Outline



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WATTS BAR
FINAL SAFETY
ANALYSIS REPORT

YARD - ADDITIONAL
DIESEL GENERATOR BUILDING
CONCRETE
EXTERIOR WALLS
OUTLINE
DWG NO. 10W331-4 RC
FIGURE 3.8.4-75

PROCADAM MAINTAINED DRAWING

Figure 3.8.4-75 Yard - Additional Diesel Generator Building Concrete Exterior Walls Outline

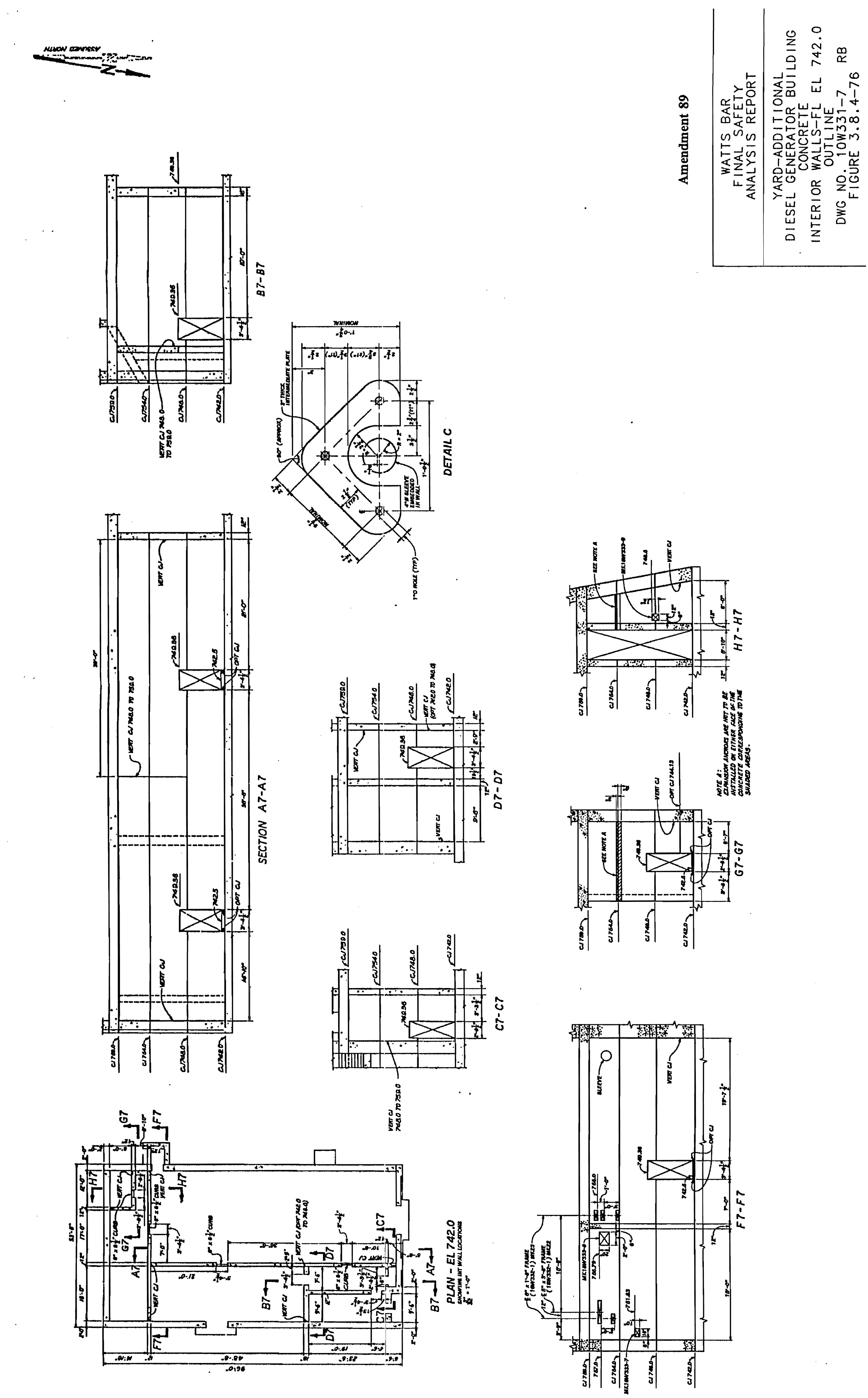

Figure 3.8.4-76 Yard - Additional Diesel Generator Building Concrete Interior Walls - Floor EL. 742.0 Outline

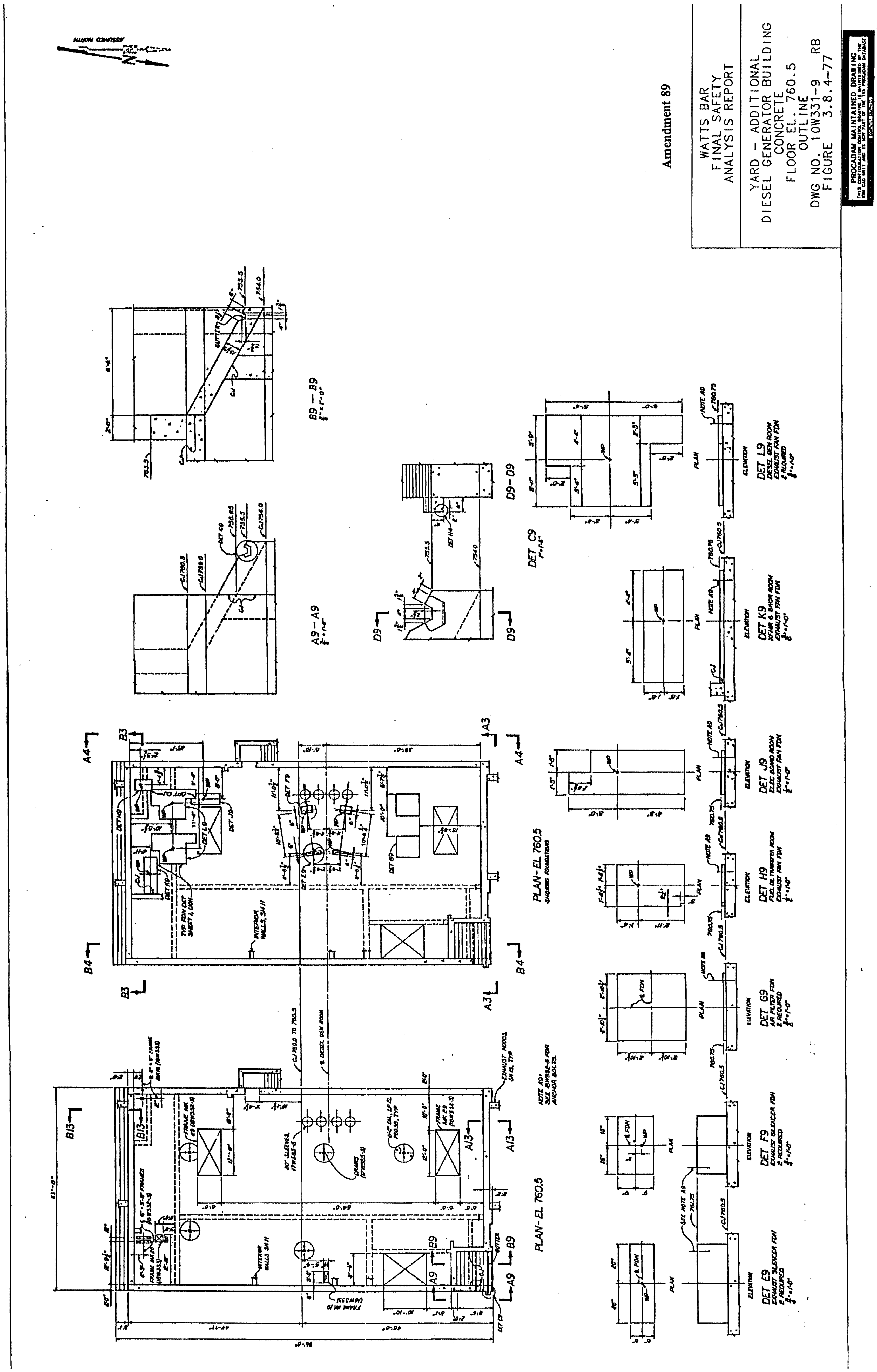

Figure 3.8.4-77 Yard - Additional Diesel Generator Building Concrete Floor EL. 760.5

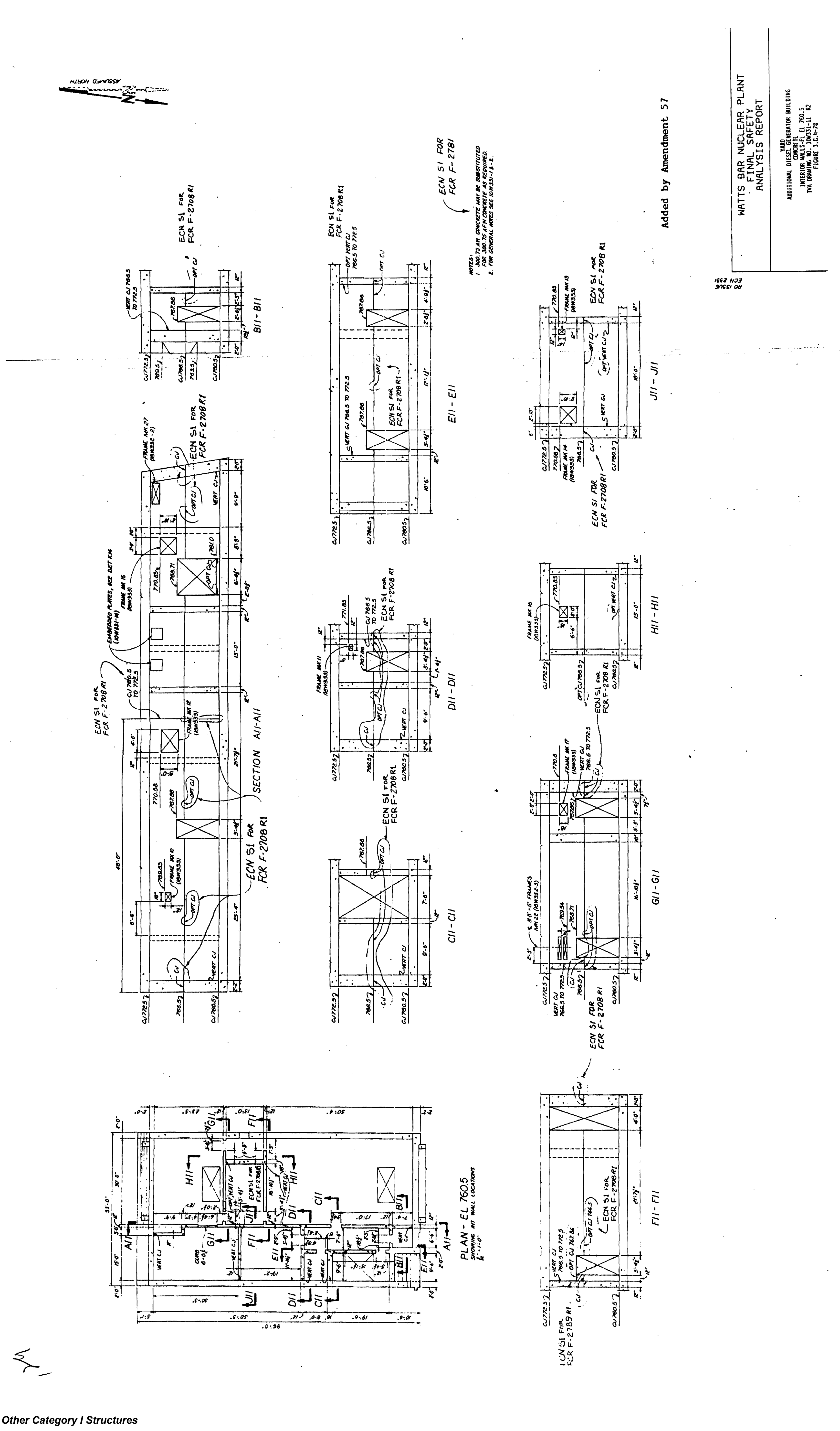

Figure 3.8.4-78 Yard Additional Diesel Generator Building Concrete Interior Walls-FL EL. 760.5

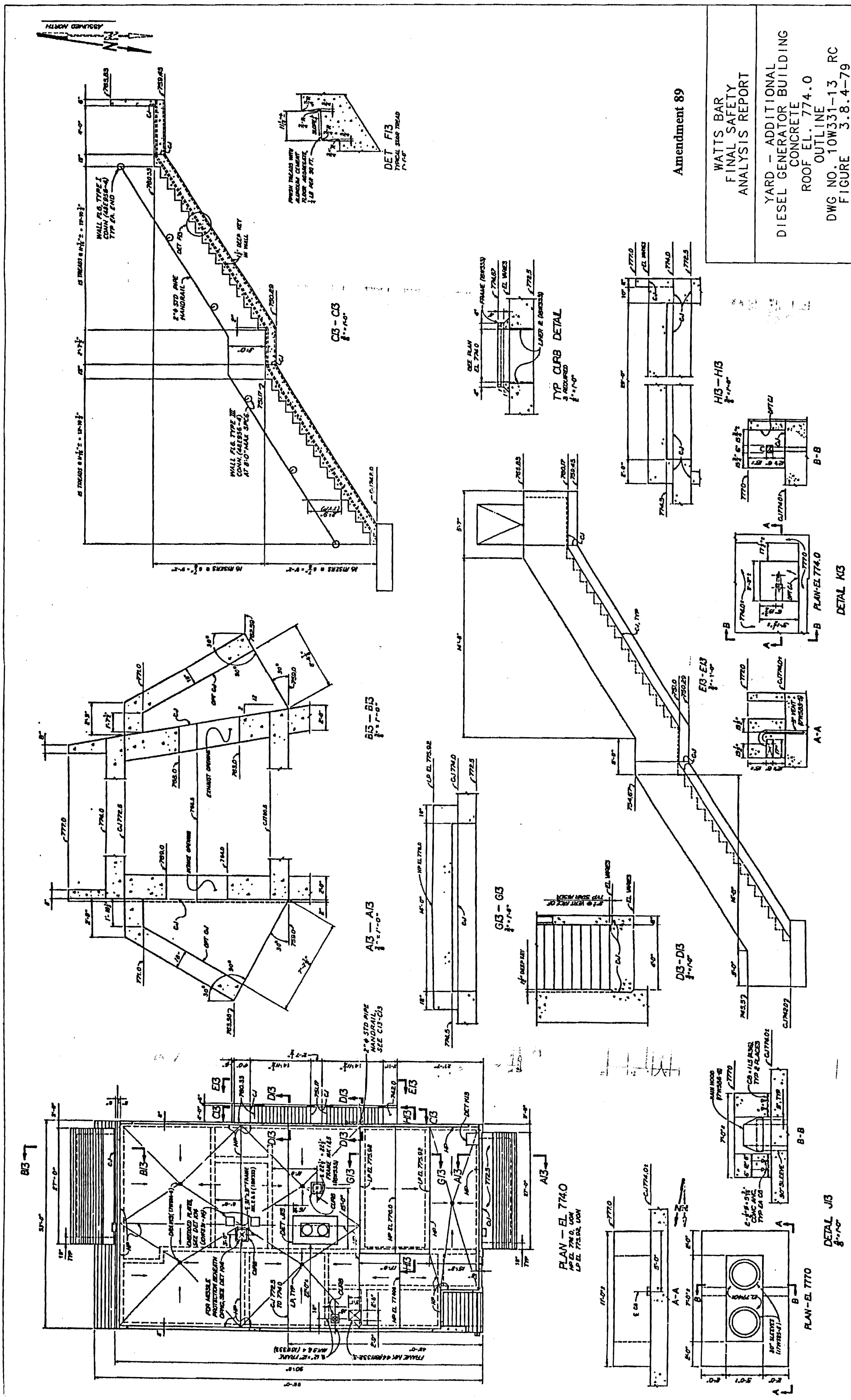

Figure 3.8.4-79 Yard - Additional Diesel Generator Building Concrete Roof EL. 774.0 Outline

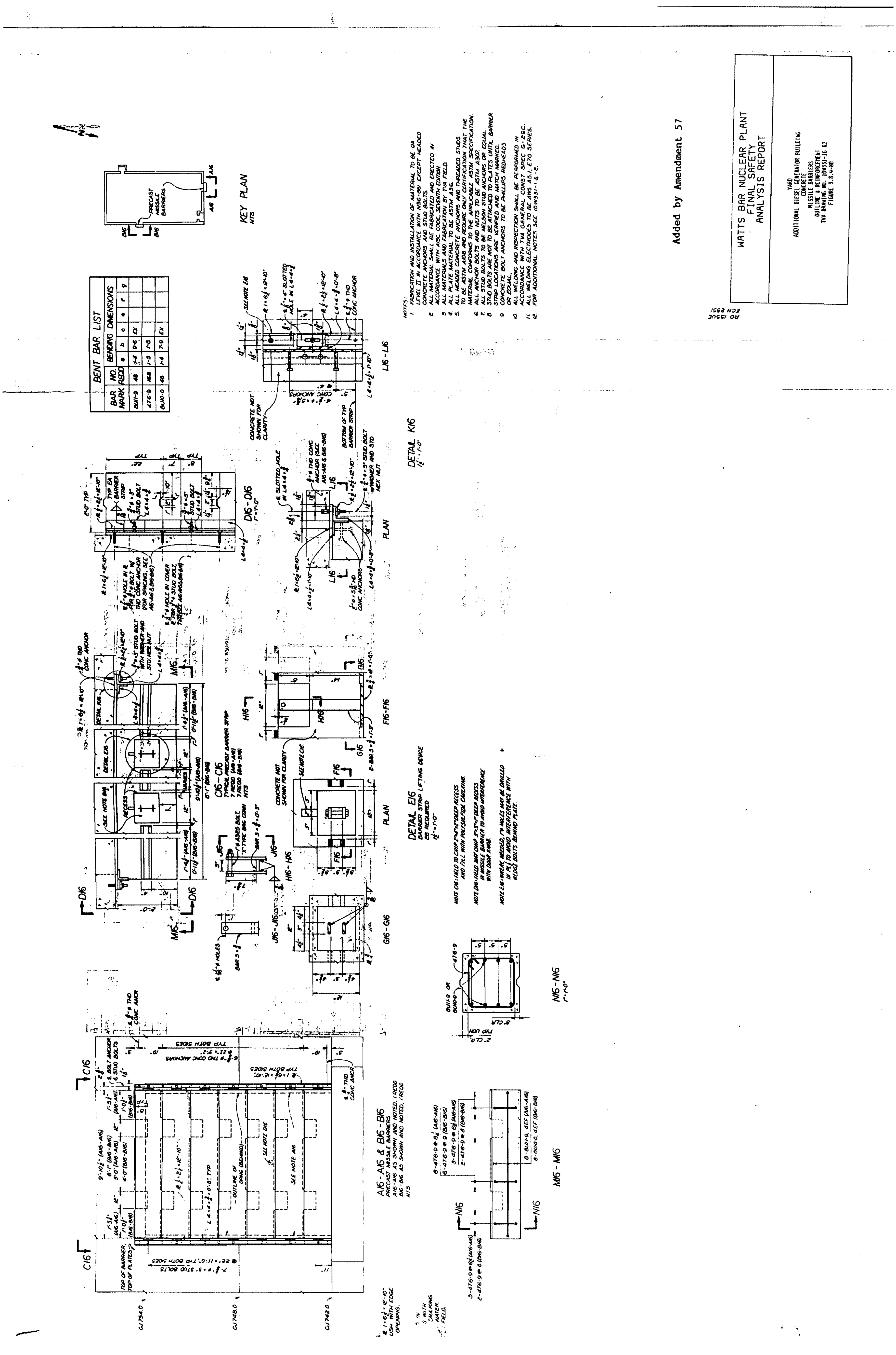

Figure 3.8.4-80 Yard Additional Diesel Generator Building Concrete Missile Barriers Outline & Reinforcement151