VOGTLE ELECTRIC GENERATING PLANT

GEORGIA POWER COMPANY

FUEL HANDLING BUILDING

DESIGN REPORT

Prepared

by

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VEGP-FUEL HANDLING BUILDING DESIGN REPORT

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1.0 INTRODUCTION

The Nuclear Regulatory Commission Standard Review Plan, NUREG-0800, requires the preparation of design reports for Category 1 structures.

This design report represents one of a series of 11 design reports and one seismic analysis report prepared for the Vogtle Electric Generating Plant (VEGP). These reports are listed below:

- Containment Building Design Report
- Containment Internal Structure Design Report
- Auxiliary Building Design Report
- Control Building Design Report
- Fuel Handling Building Design Report
- NSCW Tower and Valve House Design Report
- Diesel Generator Building Design Report
- Auxiliary Feedwater Pumphouse Design Report
- Category 1 Tanks Design Report
- Diesel Fuel Oil Storage Tank Pumphouse Design Report
- Category 1 Tunnels Design Report
- Seismic Analysis Report

The Seismic Analysis Report describes the seismic analysis methodology used to obtain the acceleration responses of Category 1 structures and forms the basis of the seismic loads in all 11 design reports.

The purpose of this design report is to provide the Nuclear Regulatory Commission (NRC) with specific design and construction information for the fuel handling building, in order to assist in planning and conducting a structural audit. Quantitative information is provided regarding the scope of the actual design computations and the final design results.

The report includes a description of the structure and its function, design criteria, loads, materials, analysis and design methodology, samples of governing design forces, a design summary of representative key structural elements.

2.0 DESCRIPTION OF STRUCTURE

2.1 GENERAL DESCRIPTION

The fuel handling building is a five-story reinforced concrete building common to the two-unit plant. It houses the new fuel storage area, cask storage pit and washdown area, and two spent fuel pools. The principal functions of the building are to receive, store, and protect new and spent fuel and to prepare spent fuel for shipment. The building is a shear wall box-type structure with floor and roof slabs acting as rigid diaphragms spanning between the walls. The building is functionally divided into three major areas, a center section that houses the Unit 1 and 2 spent fuel pools, and the east and west wing sections that contain portions of the equipment buildings. Even though the equipment buildings are seismic Category 2, they are designed to Category 1 criteria to eliminate any adverse interaction of the wings with the adjacent Category 1 buildings. The fuel handling building is designed to support the cask handling crane, which is used to transport new and spent fuel casks to and from the building. The interior and exterior walls are solid with occasional openings for doorways, heating, ventilating, and air conditioning (HVAC) ducts, piping and electrical cable trays and a large opening at grade level (elevation 220'-0"), in the center of the south exterior wall, which provides access for the cask handling crane to the auxiliary building.

2.2 LOCATION AND FOUNDATION SUPPORT

All Category 1 structures are founded within the area of the power block excavation. The excavation removed in-situ soils to elevation 130'± where the marl bearing stratum was encountered.

All Category 1 structures are located either directly on the marl bearing stratum or on Category 1 backfill placed above the marl bearing stratum. The backfill consists of densely compacted select sand and silty sand. The nominal finished grade elevation is 220'-0". The high groundwater table is at elevation 165'-0".

The fuel handling building is located south of the control building, north of the auxiliary building and in between the Unit 1 and Unit 2 containments (see figure 1). A 5-1/2-inch seismic gap is provided to separate the fuel handling building from these adjacent structures. The basemat is founded and placed directly on Category 1 backfill at elevation 154'-0" in the equipment building wing sections, and at elevation 173'-0" in the center section. In addition, this Category 1 backfill is placed against the north walls in the equipment building wing sections, west wall of the Unit 1 electrical tunnel, and the east wall of the Unit 2 piping tunnel (from elevation 154'-0" up to the bottom of the adjacent control building and raised center section basemats, which is at elevation 173'-0").

2.3 GEOMETRY AND DIMENSIONS

The fuel handling building is approximately 257 feet long by 75 feet wide and is 134 feet high. The stepped basemat elevations are 154 feet bottom of concrete (BOC) of the wings and 173 feet BOC at the raised center section. There are piping and electrical tunnels that run north-south under the spent fuel pool floor at the transition from the lower wing basemats to the raised center section basemat (see figure 1). Building plan and sections are shown in figures 2 through 4.

2.4 KEY STRUCTURAL ELEMENTS

The key structural elements in the fuel handling building include the roof and floor slabs, shear walls, walls that support the cask handling crane, basemat, and the spent fuel pool walls. Listed below is a brief description of the function and design considerations for these elements.

2.4.1 Roof and Floor Slabs

The fuel handling building has three main roof slabs, level 3 (elevation 263'-8") of both wings, and level 4 (elevation 288'-2") at the center section. The roof slabs are 1 foot 9 inches thick minimum and the roof is flat. The slabs are structurally supported by walls in the center section and walls and steel columns in both wings. Both wing roof slabs comprise part of the equipment building roof, and have openings for HVAC and the containment access and vent shafts. There are no openings in the center section roof slab.

The main floor slabs are level B (elevation 179'-0"), level A (elevation 200'-0"), level 1 (elevation 220'-0"), and level 3 (elevation 263'-8"). The slabs vary from 1 foot 6 inches to 4 feet 3 inches thick, and are structurally supported by walls.

2.4.2 Shear Walls

Lateral loads applied to the fuel handling building are resisted by the four exterior walls, the fuel pool walls, and other shear walls indicated in figures 5 and 6. The fuel pool walls are described in section 2.4.5. The exterior shear walls contain occasional openings for doorways, electrical and piping systems. They vary from 2 to 3 feet thick.

2.4.3 Walls Supporting the Cask Handling Crane

New and spent fuel casks are transported within the fuel handling building by the cask handling crane. The cask handling crane is located at the center bay of the building. The cask handling crane is supported at elevation 264'-7" by a reinforced concrete wall. The crane supporting wall is laterally stiffened by the level 3 and 4 slabs and has the structural characteristics of a deep beam.

2.4.4 Basemat

The fuel handling building basemat is approximately 75 feet wide by 257 feet long and has a uniform thickness of 6 feet. The raised center section and both wings are structurally integrated with one another on a common stepped basemat. Top of the basemat at the east and west wing sections is at elevation 160'-0" and the raised center section is at elevation 179'-0". The basemat contains several shallow sumps in both wings and center section that are approximately 5 feet below the top of their respective basemats. The cask loading pit (elevation 173'-0", top of concrete [TOC]), two transfer tube canals, and two spent fuel pools are located at the center section basemat. These sumps, pit, canals, and pools are lined with 1/4-inch-thick stainless steel plate to serve as a leaktight membrane. Electrical and piping tunnels run north-south under the raised center section of the transition from the lower wings to the raised center section basemat. The basemat is stiffened by the tunnels, fuel pool walls, interior and exterior walls at levels C and B that divide the building into several room compartments. Equipment anchored to or supported by the basemat includes the encapsulation vessels and the spent fuel storage rack system.

2.4.5 Spent Fuel Pool Walls

The fuel handling building contains two spent fuel pools, one for each unit. The fuel pool walls are a minimum of approximately 5 feet thick. The north wall of each pool forms part of the transfer tube canal and contains a gate to provide access for the transfer tube canal. The east wall of Unit 2 and the west wall of Unit 1 form part of the new fuel storage pit and contain a gate to provide access to the cask loading pit. The fuel pool walls are lined with 1/4-inch-thick stainless steel plate to serve as a leak tight membrane.

2.5 MAJOR EQUIPMENT

The primary function of the fuel handling building is to provide storage for new and spent fuel assemblies. The spent fuel assemblies are lifted and transported by a bridge crane at elevation 220'-0" that travels the east-west length of the building. The spent fuel shipping cask is lifted and transported by the cask handling crane at elevation 264'-7" that travels north-south in the center bay of the building. Spent fuel storage racks in the spent fuel pool are used for storage of the spent fuel assemblies. The new fuel storage area is a reinforced concrete pit that provides temporary dry storage for the new fuel assemblies. An equipment and cask cleaning area is located adjacent to the spent fuel pools and new fuel pit. The fuel transfer canal system is used to transport the new and spent fuel assemblies between the fuel handling building and the two containment buildings.

3.0 DESIGN BASES

3.1 CRITERIA

The following documents are applicable to the design of the fuel handling building:

3.1.1 Codes and Standards

- American Concrete Institute (ACI), building code requirements for reinforced concrete, ACI 318-71, including 1974 supplement.
- American Institute of Steel Construction (AISC), specification for the design, fabrication, and erection of structural steel for buildings, adopted February 12, 1969, and Supplements No. 1, 2, and 3.

3.1.2 Regulations

- 10 CFR 50, domestic licensing of production and utilization facilities.

3.1.3 General Design Criteria (GDC)

- GDC 1, 2, 4, and 5 of Appendix A, 10 CFR 50.

3.1.4 Industry Standards

Nationally recognized industry standards, such as American Society for Testing and Materials (ASTM), American Concrete

Institute, and American Iron and Steel Institute (AISI), are used to specify material properties, testing procedures, fabrication, and construction methods.

3.2 LOADS

Definition of each load term considered in the fuel handling building design is provided in Appendix A. The loads applicable to the fuel handling building design are individually discussed below.

3.2.1 Normal Loads

3.2.1.1 Dead Loads (D)

The dead loads considered include the weight of concrete and steel structures; piping, cable tray, conduit, HVAC duct loads, large and small equipment loads, and hydrostatic load in the spent fuel pools.

A minimum of 50 psf uniform load was applied on the applicable area of each roof and floor slab to account for piping, cable tray, conduit, HVAC duct, and small equipment loads.

The major equipment loads are listed below:

Floor Elevation	Center Area		Wing Area	
	Equipment	Weight (lb)	Equipment	Weight (lb)
263'-8"	Exhaust unit	44,500	-	-
220'-0"	Fuel cask	136,000	Exhaust and filter unit	60,000
200'-0"	New fuel racks (both units)	324,000	Spent fuel pit heat exchanger	52,000
179'-½"	Fuel rack (one pool)	3,930,000	-	-

3.2.1.2 Live Loads (L)

The live loads include occupancy loads, soil pressures, hydrostatic pressures due to groundwater, movable equipment loads, and precipitation loads.

A uniform load of 100 psf was used as the floor design live load applicable in areas not occupied by equipment. A uniform load of 30 psf was used as the roof slab live load, which envelops the effects of occupancy, snow, and 100-year rainwater ponding loads.

Static soil lateral pressure is also considered as live load.

The lift capacity of the hoist plus the impact loads were considered as the bridge crane/monorail live loads.

3.2.1.3 Operating Thermal Loads (T_o)

The thermal loads on the spent fuel walls and floor under normal operating conditions are considered in the pool wall and basemat design. The temperature data are listed below:

- Normal operating temperature in pool — 120°F
- Normal inside temperature in summer — 90°F
- Normal inside temperature in winter — 60°F

3.2.1.4 Operating Pipe and Equipment Load (R_o)

The pipe and equipment reactions during normal or shutdown condition are accounted for as part of the 50 psf of the design dead loads, (D).

3.2.2 <u>Severe Environmental Loads</u>

3.2.2.1 Operating Basis Earthquake, OBE (E)

Based on the plant site geologic and seismologic investigations, the peak ground acceleration for OBE is established as 0.12g. The free-field response spectra and the development of horizontal and vertical floor accelerations and in-structure response spectra at the basemat, floor, and roof slab elevations are discussed in the seismic analysis report.

The horizontal and vertical floor accelerations are provided in table 1.

The OBE damping values as percentages of critical applicable to the fuel handling building design are as follows:

Reinforced concrete structures	4
Welded steel structures	2
Bolted steel structures	4

The dynamic lateral earth pressures due to the OBE are computed by the Mononobe-Okabe method of analysis for dynamic earth pressures in dry cohesionless materials. Figure 7 shows the dynamic incremental soil pressure profile.

Consideration is given to hydrodynamic pressures acting on the fuel pool walls and basemat, (reference 1). Representative hydrodynamic pressure profiles are provided in figures 8 and 9.

3.2.2.2 Design Wind (W)

The fuel handling building is completely surrounded by other Category 1 structures, and is designed for a wind velocity of 110 mph, which is based on a wind speed 30 feet above ground. Exposure C, applicable to flat open country is used. The effective velocity pressure profile for the 110 mph wind used in the design (see figure 12) is in accordance with reference 2.

3.2.3 <u>Extreme Environmental Loads</u>

3.2.3.1 Safe Shutdown Earthquake, SSE (E')

Based on the plant site geologic and seismologic investigations, the peak ground acceleration for SSE is established as 0.20g. The free-field response spectra and the development of horizontal and vertical floor accelerations and in-structure response spectra at the basemat, floor, and roof slab elevations are discussed in the seismic analysis report.

The horizontal and vertical floor accelerations are provided in table 1.

The SSE damping values as percentages of critical applicable to the fuel handling building design are as follows:

Reinforced concrete structures	7
Welded steel structures	4
Bolted steel structures	7

The dynamic lateral earth pressures due to the SSE are computed by the Mononobe-Okabe method of analysis for dynamic earth pressures in dry cohesionless materials. Figure 7 shows the dynamic incremental soil pressure profile.

Consideration is given to hydrodynamic pressures acting on the fuel pool walls and basemat (reference 1). Representative hydrodynamic pressure profiles are provided in figures 10 and 11.

3.2.3.2 Tornado Loads (W_t)

Loads due to the design tornado include wind pressures, atmospheric pressure differentials, and tornado missile strikes. The design tornado parameters, which are in conformance with the Region I parameters defined in Regulatory Guide 1.76, are as follows:

• Rotational tornado speed	290 mph
• Translational tornado speed	70 mph maximum 5 mph minimum
• Maximum wind speed	360 mph
• Radius of tornado at maximum rotational speed	150 feet
• Atmospheric pressure differential	-3 psi
• Rate of pressure differential change	2 psi/sec

The resultant tornado effective velocity pressure profile used in the design (see figure 12) is in accordance with reference 3. The effective velocity pressure includes the size coefficient, and is used in conjunction with the external pressure coefficient to determine the net positive and negative pressures.

The fuel handling building is a partially vented structure. Conservatively, all walls and slabs are designed for a tornado pressurization effect of ± 3 psi.

The fuel handling building is also designed to withstand tornado missile input effects from airborne objects transported by the tornado. The tornado missile design parameters are listed in table 2. Missile trajectories up to and including 45 degrees off the horizontal use the listed horizontal velocities. Those trajectories greater than 45 degrees use the listed vertical velocities.

Tornado loading (W_t) is defined as the worst case of the following combinations of tornado load effects:

$W_t = W_{tq}$ (Velocity pressure effects)
$W_t = W_{tp}$ (Atmospheric pressure drop effects)
$W_t = W_{tm}$ (Missile impact effects)
$W_t = W_{tq} + 0.5\ W_{tp}$
$W_t = W_{tq} + W_{tm}$
$W_t = W_{tq} + 0.5\ W_{tp} + W_{tm}$

3.2.3.3 Probable Maximum Precipitation Load, PMP (N)

The load due to probable maximum precipitation is applied to the fuel handling building roof areas.

Special roof scuppers are provided with sufficient capacity to ensure that the depth of ponding water due to the PMP rainfall does not exceed 18 inches. This results in an applied PMP load of 94 psf.

3.2.3.4 Blast Load (B)

The blast load accounts for a postulated site-proximity explosion. The blast load is conservatively taken as a peak positive incident overpressure of 2 psi (acting inward or outward) applied as a uniform static load.

3.2.4 Abnormal Loads

3.2.4.1 Thermal Load (T_a)

The thermal loads on the spent fuel wall and floor under abnormal conditions are considered in the spent fuel pool wall and basemat design. The design temperature in the pool is 195°F.

3.3 LOAD COMBINATIONS AND STRESS/STRENGTH LIMITS

The load combinations and stress/strength limits for structural steel and concrete are provided in Appendix B.

3.4 MATERIALS

The following materials and material properties are used in the design of the fuel handling building:

3.4.1 Concrete

• Compressive strength	$f_c' = 4$ ksi
• Modulus of elasticity	$E_c = 3,830$ ksi
• Shear modulus	$G = 1,530$ ksi
• Poisson's ratio	$\nu = 0.17 - 0.25$

3.4.2 Reinforcement - ASTM A615, Grade 60

• Minimum yield stress	$F_y = 60$ ksi
• Minimum tensile strength	$F_{ult} = 90$ ksi
• Minimum elongation	7-9% in 8 inches

3.4.3 Structural Steel

3.4.3.1 ASTM A36

- Minimum yield stress F_y = 36 ksi
- Minimum tensile strength F_{ult} = 58 ksi
- Modulus of elasticity E_s = 29,000 ksi

3.4.3.2 ASTM A500, Grade B: Structural Tubing

- Minimum yield stress F_y = 46 ksi
- Minimum tensile strength F_{ult} = 58 ksi
- Modulus of elasticity E_s = 29,000 ksi

3.4.4 Structural Bolts

3.4.4.1 ASTM A325 (1/2 inch to 1 inch diameter inclusive)

- Minimum yield stress F_y = 92 ksi
- Minimum tensile strength F_{ult} = 120 ksi

3.4.4.2 ASTM A325 (1-1/8 inch to 1-1/2 inch inclusive)

- Minimum yield stress F_y = 81 ksi
- Minimum tensile strength F_{ult} = 105 ksi

3.4.4.3 ASTM A307

- Minimum yield stress F_y is not applicable
- Minimum tensile strength F_{ult} = 60 ksi

3.4.5 Steel Liner Plate - ASTM A240, Type 304L

- Minimum yield stress F_y = 25 ksi
- Minimum tensile strength F_{ult} = 70 ksi
- Modulus of elasticity E_s = 29,000 ksi

3.4.6 Anchor Bolts and Headed Anchor Studs

3.4.6.1 ASTM A36

- Minimum yield stress — F_y = 36 ksi
- Minimum tensile strength — F_{ult} = 58 ksi

3.4.6.2 ASTM A108

- Minimum yield stress — F_y = 50 ksi
- Minimum tensile strength — F_{ult} = 60 ksi

3.4.6.3 ASTM A307

- Minimum yield stress — F_y is not applicable
- Minimum tensile strength — F_{ult} = 60 ksi

3.4.6.4 ASTM A320, Grade B8

- Minimum yield stress — F_y = 30 ksi
- Minimum tensile strength — F_{ult} = 75 ksi

3.4.7 Foundation Media

3.4.7.1 General Description

See section 2.2

3.4.7.2 Category 1 Backfill

- Moist unit weight — γ_m = 126 pcf
- Saturated unit weight — γ_t = 132 pcf
- Shear modulus

G	Depth (feet)
1530 ksf	0-10
2650 ksf	10-20
3740 ksf	20-40
5510 ksf	40-marl bearing stratum

- Angle of internal friction — $\phi = 34°$
- Cohesion — C = 0

3.4.7.3 Modulus of Subgrade Reaction

- Static 85 kcf
- Dynamic 250 kcf

3.4.7.4 Net Bearing Capacities

- Ultimate 64.0 ksf
- Allowable static 21.3 ksf
- Allowable dynamic 32.0 ksf

4.0 STRUCTURAL ANALYSIS AND DESIGN

This section provides the methodologies employed to analyze the fuel handling building and to design its key structural elements, using the applicable loads and load combinations specified in section 3.0.

A preliminary proportioning of key structural elements is based on plant layout and separation requirements, and, where applicable, the minimum thickness requirements for radiation shielding and for the prevention of concrete scabbing or perforation due to tornado missile impact. The proportioning of these elements is finalized by confirming that strength requirements and where applicable, ductility and/or stiffness requirements are satisfied.

The structural analysis is performed either by manual analysis or computer analysis. In the manual analysis, the building structure or substructure is considered as an assemblage of slabs, walls, and columns, and the analysis is performed using standard structural analysis techniques. In the computer analysis, the building structure or substructure is modeled as an assemblage of finite elements and the analysis is performed using the standard finite element method utilizing a computer program.

For manual analyses, the analysis techniques, boundary conditions, and application of loads are provided to illustrate the method of analysis.

For computer analyses, the modeling techniques, boundary conditions, application of loads, and description of the computer model are provided to illustrate the overall method of analysis.

In addition, for both manual and computer analyses and design, representative analysis and design results are provided to illustrate the response of the key structural elements for governing load combinations.

4.1 SELECTION OF GOVERNING LOAD COMBINATION

An evaluation of load magnitudes, load factors and load combinations is performed to determine the load combination that governs the overall response of the structure. It is determined that load combination equation 2, for steel design (Appendix B, Table B.1) and equation 3 for concrete design (Appendix B, Table B.2) containing OBE, govern over all other load combinations, and hence forms the basis for the overall structural analysis and design of the fuel handling building.

All other load combinations, including the effects of abnormal loads and tornado loads, are evaluated where applicable on a local area basis (i.e., sections 5.2 and 5.3). The localized response is combined with the analysis results of the overall structural response, as applicable, to confirm that design integrity is maintained.

4.2 VERTICAL LOAD ANALYSIS

The vertical load carrying elements of the fuel handling building consist of concrete slabs that support the applied vertical loads, walls and columns that support the slabs, and the basemat which transmits the loads from the walls and columns to the foundation medium. Representative vertical load carrying elements are identified in figures 5 and 6.

The analysis of the building for vertical loads begins at the roof slab and proceeds progressively down through each level of the building to the basemat. Slabs and girders are analyzed for the vertical loads applied to them. The total vertical load on a wall or column at a given level is computed based on its self weight, the vertical loads at that level from the slab tributary areas, and the cumulative vertical loads from the levels above.

4.3 LATERAL LOAD ANALYSIS

The lateral load carrying elements of the fuel handling building consist of concrete slabs acting as rigid diaphragms to resist applied lateral loads, the shear walls which transmit the loads from the slab diaphragm to the basemat, and the basemat which transmits the loads from the walls and columns to the foundation medium. Representative lateral load-carrying elements are identified in figures 5 and 6.

Since the building structure utilizes the slab diaphragms for horizontal shear distribution, the lateral load analysis is performed by a conventional rigidity and mass analysis. In this analysis, the maximum horizontal design forces for earthquake loads and soil pressure loads are applied at each slab level, as appropriate. The design horizontal earthquake load at each level of the building is obtained by multiplying the lumped story mass at that level by the maximum floor acceleration applicable to that level. The design horizontal soil pressure load of the building is obtained from the lateral earth pressure with due consideration to the seismic effects and the surcharge effects from the raised center section basemat. In the analysis, the horizontal shear loads are carried progressively down from the roof diaphragm through each level of the building to the basemat, to obtain the story shear at each level. The story shear load at each level is distributed to the shear walls at that level in proportion to their relative rigidities.

To account for the torsion caused by the seismic wave propagation effects, the inherent building eccentricity between the center of mass and center of rigidity at each level is increased by 5 percent of the maximum plan dimension in the computation of the torsional moment. The torsional moment is obtained as the product of this augmented eccentricity and the story shear at that level. The shear in the walls resulting from this torsional moment is computed based on the relative torsional rigidities of the walls.

For a given shear wall, the shear due to story shear (direct shear) and shear due to torsional moment (torsional shear) are combined at a given level to obtain the total design shear load. The torsional shear is neglected when it acts in a direction opposite to the direct shear.

4.4 COMBINED EFFECTS OF THREE COMPONENT EARTHQUAKE LOADS

The combination of co-directional responses due to three component earthquake effects is performed using the Square Root of the Sum of the Squares (SRSS) method, i.e., $R = \left(R_i^2 + R_j^2 + R_k^2\right)^{1/2}$ or the Component Factor method, i.e.,

$$R = R_i + 0.4\ R_j + 0.4\ R_k$$
$$R = 0.4\ R_i + R_j + 0.4\ R_k$$
$$R = 0.4\ R_i + 0.4\ R_j + R_k,$$

wherein 100 percent of the design forces from any one of the three components of the earthquake is considered in combination with 40 percent of the design forces from each of the other two components of the earthquake.

4.5 ROOF AND FLOOR SLABS

4.5.1 Analysis and Design Methodology

A representative slab panel plan (elevation 200'-0") of the fuel handling building is presented in figure 5, showing the structural elements provided for vertical and lateral support of the slab panels, which consist of load bearing walls and load bearing shear walls. Based on the panel configuration, the relative stiffness of the supporting members and the type of fixity provided, slab panels are analyzed for one-way or two-way slab action using appropriate boundary conditions and standard beam and plate formulae.

Equivalent uniformly distributed loads are applied to slab panels. The design vertical earthquake loads for slab panels in a level are obtained by multiplying the effective mass from the applied loading (including its own mass) by the maximum floor acceleration at that level.

Based on the floor flexibility study, it is concluded that the effects of vertical flexibility on the fuel handling building floor accelerations and response spectra are insignificant, as long as the fundamental floor system frequency is equal to or higher than 10 cps. The evaluation of the floor systems in the fuel handling building demonstrates that their frequencies are higher than this value. The details of the floor flexibility study are provided in the seismic analysis report.

Slab panels are selected for design on the basis of the controlling combination of design load intensity, span, panel configuration, and support conditions.

The structural design is based on strength considerations and consists of sizing and detailing the reinforcing steel to meet the ACI 318 Code requirements. In general, the reinforcing requirements are determined for the governing face of the slab and conservatively provided on both faces.

As appropriate, additional reinforcement is provided in the slab adjacent to large floor openings.

4.5.2 Design Results

The design results for governing load combinations are presented in table 3 for representative slab panels. See figure 13 for representative design details.

4.6 SHEAR WALLS

4.6.1 Analysis and Design Methodology

The location of shear walls are identified in figures 5 and 6 for representative elevations.

The details of the analysis methodology used to compute the total in-plane design loads at various levels of a shear wall are described under vertical and lateral load analyses in sections 4.2 and 4.3. The in-plane design loads include axial loads resulting from the overturning moment.

The out-of-plane design loads that are considered include the inertia loads on the walls due to the structural acceleration caused by the design earthquake.

The design in-plane shear force and the overturning moment acting on a shear wall at a given level is computed by considering the shear loads acting at all levels above, and the resulting overturning moments. Conventional beam analysis is used to compute the bending moment and out-of-plane shear forces resulting from the out-of-plane design loads. At controlling sections, the combined effects of in-plane overturning moment and axial loads, and the out-of-plane loads are evaluated.

The shear wall design is performed in accordance with the ACI 318 Code using the following methodology:

A. The horizontal and vertical reinforcement required to resist the design shear loads is determined.

B. The flexural capacity of the shear wall using the reinforcement determined is obtained using the Cardenas equation (reference 4).

C. If the flexural capacity computed is less than the design overturning moment, then the reinforcement required is determined in one of the following two ways:

   1. The total vertical reinforcement required for the design moment is computed using the Cardenas equation and is distributed uniformly along the length of the wall.

   2. The reinforcement required in the end sections of the wall to resist the overturning moment is computed.

D. The reinforcement provided for the in-plane loads is evaluated for the combined effects of in-plane and out-of-plane loads, and additional reinforcing steel is added if necessary.

4.6.2 Design Results

The design results for governing load combinations are presented in table 4 for representative shear walls. See figure 14 for representative design details.

4.7 WALLS SUPPORTING THE CASK HANDLING CRANE

4.7.1 Analysis and Design Methodology

The structure supporting the cask handling crane is designed as a simply supported deep beam, consisting of the wall in web action, and the effective areas of the roof slab and the level 3 floor slab in flange action. The deep beam moments and shears are determined using standard beam formulas.

Uniformly distributed roof and floor loads are converted to an equivalent uniform linear load using the tributary load method. Concentrated cask handling crane truck loads are applied eccentrically to the bottom of the wall at the rail centerline. The design vertical earthquake load for the supporting wall is obtained by multiplying the tributary mass from the applied loading (including the deep beam wall's own mass) by the maximum floor acceleration at the level 4 roof.

The structural design of the walls supporting the cask handling crane is governed by strength considerations, and consists of sizing and detailing the reinforcing steel in accordance with the provisions of the ACI 318 Code. Appropriate consideration is given to the corbel-like torsion action on the wall ledge.

4.7.2 Design Results

The design results for governing load combinations are presented in table 5 for representative walls supporting the cask handling crane. See figures 15 and 16 for design details.

4.8 BASEMAT

4.8.1 Analysis Methodology and Computer Model

The basemat is analyzed utilizing a finite element model with the Bechtel Structural Analysis Program (BSAP), which is a general purpose computer program for finite element analyses. This program uses the direct stiffness approach to perform linear elastic analysis of a three-dimensional finite element model.

The finite element model includes the structural elements in the building through elevation 220'-0" and the basemat, and is prepared using conventional modeling techniques. Plate elements are used to model the basemat, the spent fuel pool walls, and all other structural walls and slabs below elevation 220'-0". Boundary (spring-type) elements are used as follows:

A. To characterize the stiffness effects of soil beneath the basemat.

B. To eliminate singularity conditions by providing boundary conditions that prevent in-plane rotation of walls that are oriented in a manner which precludes the use of global boundary conditions to eliminate the inplane rotational degrees of freedom.

The vertical stiffness of each soil spring is determined by multiplying the nodal tributary area by the modulus of subgrade reaction. The horizontal spring stiffnesses are computed to model the stiffness effect of the soil in the horizontal direction.

The structural shear walls to elevation 220'-0" are modeled to represent the stiffness interaction effects at the wall/basemat junction. There are a total of 1002 boundary elements which represent soil stiffness, 1489 plate elements to model the basemat and walls, and 8 beam elements along the periphery of the basemat in the penetration area to model the thickened portion of the mat around the containment building.

Computer plots of the fuel handling building basemat model, including node numbers and element numbers, are shown in figure 17. Only one half of the fuel handling building is modeled to take advantage of the symmetry of the building in the east-west direction about the centerline of the two-unit plant.

The boundary conditions for the basemat model are as follows: boundary elements, representing the translational soil stiffness, are applied at each basemat node in the three global translational directions; boundary elements with large rotational rigidity and no translational rigidity are applied at the plate element nodes to eliminate singularity conditions by restraining in-plane rotation; along the axis of building symmetry, symmetrical boundary conditions are used for vertical and north-south loads, and anti-symmetrical boundary conditions are used for east-west loads.

4.8.2 Application of Loads

The magnitude and distribution of loads applied to the basemat model are consistent with the cumulative results of the vertical and horizontal load analyses of the overall building structure. As described in the other sections of this report, the loads include dead load, live load, hydrostatic, and hydrodynamic loads, vertical and horizontal seismic loads, and lateral soil pressure loads.

Dead load, live load, and vertical and horizontal seismic loads for the elements in the model are accounted for internally by the computer program by assigning a mass density to the plate elements and applying the appropriate static acceleration. Dead load, live load, and vertical and horizontal seismic loads associated with the portion of the structure above elevation 220'-0" are applied as nodal forces at elevation 220'-0".

Hydrostatic loads due to water in the fuel pool are applied to the appropriate plate elements as equivalent pressure loads. The hydrodynamic effects of the water, including both impulsive and convective forces (reference 1) are applied to the appropriate plate elements as equivalent pressure loads.

Lateral soil pressure loads and surcharge from the center portion of the fuel handling building are applied to the electrical tunnel wall (refer to section 2.4.4).

4.8.3 Design Methodology

The design of the basemat, including the sizing and detailing the reinforcing steel, is done in accordance with the requirements of the ACI 318 Code.

The required flexural reinforcement in the basemat is calculated using the OPTCON module of program BSAP-POST. BSAP-POST (which consists of a collection of modules that perform specific independent tasks) is a general purpose, post-processor program for the BSAP finite element analysis program. BSAP-POST reads computed BSAP results into an internal common data storage base and optionally performs one or several additional operations (i.e., plotting) or calculations (i.e., creating load combinations or designing reinforced concrete members).

In general, the OPTCON processor is a reinforced concrete analysis and design program for doubly reinforced concrete sections which creates reinforced concrete interaction diagrams based on the maximum allowable resistance of a section for given stress and strain limitations (code allowables). Any load combination whose design axial force and corresponding moment (load set) falls within the interaction diagram indicates all stress and strain code criteria are satisfied.

The thermal effects on the basemat under operating conditions are evaluated using the methodology described in section 5.3.

Basemat shear is computed using the design moments from the finite element analysis and determining the moment gradient between adjacent elements.

4.8.4 Design Results

Representative results of the basemat analysis is provided in figure 18. Representative results of the basemat design is provided in figure 19. Representative design details are shown in figures 20 and 21.

4.9 SPENT FUEL POOL WALLS

4.9.1 Analysis Methodology and Computer Model

The spent fuel pool walls are analyzed utilizing the basemat finite element computer model. The analysis methodology and computer model are described in section 4.8.1.

4.9.2 Application of Loads

The load application procedures for the analysis of the spent fuel pool walls are described in section 4.8.2.

4.9.3 Design Methodology

The design of the spent fuel pool walls, including the sizing and detailing of reinforcing steel, is done in accordance with the strength design provisions of the ACI 318 code.

The required flexural reinforcement in the spent fuel pool walls is determined based on the design forces obtained from the BSAP analysis (refer to sections 4.8.1 and 4.8.2), with the use of a OPTCON computer program. For a description of computer design using the OPTCON module of BSAP-POST refer to section 4.8.3.

4.9.4 Design Results

Representative results of the spent fuel pool analysis is provided in figure 22. Representative results of the spent fuel pool wall design is provided in figure 23. Representative design details are shown in figure 24.

5.0 MISCELLANEOUS ANALYSIS AND DESIGN

Once the basic design of the fuel handling building has been completed (refer to section 4), the structure is evaluated for the effects of abnormal loads and tornado loads. This is done on a local area basis where applicable. In addition, the overall stability of the fuel handling building is evaluated to ensure an adequate safety factor against instability is provided. This section describes these analyses and significant special provisions employed in the fuel handling building design.

5.1 STABILITY ANALYSIS

The overall stability of the fuel handling building is evaluated by determining the factor of safety against overturning, sliding, and flotation.

5.1.1 Overturning

The factor of safety against overturning is determined using the equivalent static method and the energy balance method.

The equivalent static method does not account for the dynamic characteristics of the loading and therefore results in a factor of safety lower than the energy balance method. The factor of safety obtained from the energy balance method reflects the actual design conditions and therefore provides a more appropriate measure of the design margin.

The factor of safety against overturning using the equivalent static method is defined as the ratio of the resisting moment due to net gravity forces to the overturning moment caused by the maximum lateral forces acting on the structure. The gravity forces are reduced to account for the effects of buoyancy and the vertical component of earthquake.

The factor of safety against overturning using the energy balance method is defined as the ratio of the increase in the potential energy at the point of overturning about the critical edge of the structure to the maximum kinetic energy that could be imparted to the structure as a result of earthquake loadings. The energy balance analysis methodology is described in reference 5.

5.1.2 Sliding

The factor of safety against sliding is defined as the ratio of combined frictional and passive sliding resistance of the foundation to the maximum calculated lateral force.

5.1.3 Flotation

The factor of safety against flotation is defined as the ratio of the total weight of the structure and its foundation to the buoyant force, defined as the volume of the ground water displaced by the submerged portion of the structure multiplied by the unit weight of water.

5.1.4 Analysis Results

The minimum required factors of safety and the calculated factors of safety for stability are provided in table 6.

5.2 TORNADO LOAD EFFECTS

Tornado load effects result from wind pressures, atmospheric pressure differentials, and tornado missile strikes. The magnitude and combinations of tornado load effects considered are described in section 3.2. The load combination involving tornado load effects is specified by equation 8 of Table B.2 in Appendix B.

Controlling roof and exterior wall panels are evaluated for tornado load effects, and the localized response is combined with the analysis results of the overall structural response, as applicable, to confirm that design integrity is maintained. Additional reinforcing steel is provided in accordance with the ACI 318 Code, if necessary, to satisfy design requirements. In addition, barriers are provided for the openings in the exterior walls or roofs unless the systems or components located in the exterior rooms are nonsafety-related. In this case, the interior walls and slabs are treated as barriers for the safety-related systems or components located in the interior rooms. Any openings in the exterior walls or slabs and the interior walls or slabs that may be susceptible to missile entry are evaluated to ensure that no safety-related systems or components are located in a potential path of the missile.

The methodology used to analyze and design the structural elements to withstand the tornado load effects is described in reference 3. Specific procedures used for analysis of missile impact effects are described in Appendix C.

Representative results of the tornado missile analysis are provided in table 7.

All wall and roof panels providing protection against tornado load effects have a minimum thickness of 24 and 21 inches respectively, to preclude missile perforation and concrete scabbing.

5.3 ABNORMAL LOADS EFFECTS

For this structure the only applicable abnormal loads are generated by a postulated accident which occurs only in the spent fuel pool.

The spent fuel pools are located between column lines $F_{A.3}$ and $F_{A.8}$, and F_4 and F_6 for Unit 1, and F_1 and F_3 for Unit 2.

The spent fuel pool walls and floor are analyzed using the BSAP computer program, utilizing a finite element model as described in sections 4.8 and 4.9. The loads applied to the model include dead loads, live loads, vertical and horizontal OBE/SSE loads, hydrodynamic and hydrostatic, and thermal loads. Load combination equations 9, 10 and 11 of Appendix B, Table B.2 are considered in determining the design forces.

The reinforcing steel provided on the basis of overall structural response, as per the design methodology described in section 4, is evaluated for the governing design forces resulting from the effects of abnormal loads, to ensure that the requirements of the ACI 318 Code are satisfied. This is accomplished using OPTCON computed program described in section 4.8.3.

OPTCON calculates the thermal moment induced by the thermal gradient, by considering the relaxation effects of concrete cracking and reinforcement-yielding. For each load combination analyzed, the state of stress and strain is determined before the thermal load is applied. The thermal moment is approximated based upon an iterative approach which considers equilibrium and compatibility conditions. The final force-moment set (which includes the cracked section final thermal moment) is checked to verify that it falls within the code allowable interaction diagram.

5.4 FOUNDATION BEARING PRESSURE

The maximum calculated bearing pressures under the governing design load conditions are provided in table 8.

6.0 CONCLUSION

The analysis and design of the fuel handling building includes all credible loading conditions and complies with all applicable design requirements.

7.0 REFERENCES

1. U.S. Atomic Energy Commission, Nuclear Reactors and Earthquakes, Division of Technical Information, Report TID-7024, August 1963.

2. "Building Code Requirements for Minimum Design Loads in Buildings and Other Structures," ANSI A58.1-1972, American National Standards Institute, New York, N.Y., 1972.

3. BC-TOP-3-A, Revision 3, Tornado and Extreme Wind Design Criteria for Nuclear Power Plants, Bechtel Power Corp., August 1974.

4. Design Provisions for Shear Walls, Portland Cement Association, 1973.

5. BC-TOP-4-A, Revision 3, Seismic Analyses of Structures and Equipment for Nuclear Power Plants, Bechtel Power Corp., November 1974.

TABLE 1

FUEL HANDLING BUILDING SEISMIC ACCELERATION VALUES

	FLOOR ACCELERATIONS (g's)(1)					
	SSE			OBE		
Elevation	E-W	N-S	Vert.	E-W	N-S	Vert.
160'-0"	0.24	0.21	0.39	0.16	0.14	0.24
179'-0½"	0.34	0.25	0.41	0.22	0.17	0.27
200'-0"	0.37	0.27	0.42	0.24	0.19	0.28
220'-0" (grade level)	0.39	0.30	0.43	0.25	0.20	0.29
263'-8"	0.54	0.41	0.46	0.35	0.28	0.31
263'-8"	0.54	0.42	0.46	0.35	0.28	0.31
288'-2"	0.61	0.49	0.49	0.42	0.33	0.33

(1) The actual acceleration values used in the design of the structure may be higher than the values shown.

TABLE 2

TORNADO MISSILE DATA

Missile	Weight W (lb)	End-On Height Limit (ft)	End-On Horizontal Velocity (ft/sec)	Vertical Velocity (ft/sec)
4" x 12" x 12' Plank	200	216	200	160
3" Ø std x 10' Pipe	78.5	212	200	160
1" Ø x 3' Steel Rod	8	Unlimited	317	254
6" Ø std x 15' Pipe	285	101	160	128
12" Ø std x 15' Pipe	744	46	150	120
13-1/2" Ø x 35' Utility Pole	1490	30(1)	211	169
Automobile (20 ft^2 project area)	4000	0	75	60

(1) To 30' above all grade levels within 1/2 mile of facility structures.

TABLE 3

DESIGN RESULTS OF REPRESENTATIVE SLAB PANELS

Element	Governing[1] Load Combination Equation	Design Force Moment k-ft/ft	A_s Required (in.^{2}/ft)				A_s Provided (in.^{2}/ft)			
			N-S		E-W		N-S		E-W	
			Top	Bot.	Top	Bot.	Top	Bot.	Top	Bot.
Level A Center Section Corridor Slab	3	N-S $+M_u$ = 32.2 N-S $-M_u$ = 24.3	0.39 (2)	0.51	0.39 (2)	0.39 (2)	0.60	0.60	0.44	0.44
Level A New Fuel Storage Pit Slab	3	E-W M_u = ± 54.0	0.56 (2)	0.56 (2)	0.56 (2)	0.56 (2)	1.00	1.00	1.00	1.00
Level 1 Center Section Corridor Slab	3	N-S $+M_u$ = 21.8 N-S $-M_u$ = 30.5	0.54	0.38	0.28 (2)	0.28 (2)	0.60	0.60	0.44	0.44
Level 1 Cask Washdown Area Slab	3	N-S $+M_u$ = 43.3 N-S $-M_u$ = 79.0	0.93 (2)	0.93 (2)	0.93 (2)	0.93 (2)	1.00	1.00	1.00	1.00
Level 4 Roof Slab Between F_3 & F_4	3	N-S $+M_u$ = 65.9 N-S $-M_u$ = 140.5 E-W $+M_u$ = 120.1 E-W $-M_u$ = 266.9	1.67	0.74	2.90	1.28	3.12	1.56	3.12	3.12

(1) Load combination equations correspond to equations in Appendix B.
(2) Governed by minimum code reinforcement requirements.

TABLE 4

DESIGN RESULTS OF REPRESENTATIVE SHEAR WALLS

Element	Governing[1] Load Combination Equation	Design Forces (In-Plane)			A_s Required			A_s Provided		
					(in.^{2}/ft)		(in.^{2})	(in.^{2}/ft)		(in.^{2})
		N_u (k)	V_u (k)	M_u (k-ft)	Horiz.	Vert.	ER[3]	Horiz.	Vert.	ER[3]
Level 3 North Wall Between F_1 and F_6	3	747	4,575	112,047	0.72 (2)	0.72 (2)	-	2.00	2.00	-
Level 3 South Wall Between F_4 and F_6	3	89	1,406	34,433	1.08 (2)	1.08 (2)	-	2.00	2.00	-
Level 2 East or West Wall Between F_A and F_B	3	473	9,735	516,989	2.28	2.00	133.4	3.12	2.54	220.00
Level B North Wall Between F_1 and F_6	3	13,717	25,271	1,013,740	2.35 (2)	2.35 (2)	-	2.54	2.54	-
Level B South Wall Between F_1 and F_6	3	8,869	15,628	1,126,030	1.08 (2)	1.08 (2)	-	2.00	2.00	-

(1) Load combination equations correspond to equations in Appendix B.
(2) Governed by minimum code reinforcement requirements.
(3) ER - End reinforcement

TABLE 5

DESIGN RESULTS OF WALLS SUPPORTING THE CASK HANDLING CRANE

Element	Governing(1) Load Combination Equation	Design Force	A_s Required	A_s Provided
Deep Beam Wall	3	80,000 k-ft	70.33 in.^{2}	83.25 in.^{2}
Corbel	3	Shear-friction 144 k/ft	2.02 in.^{2}/ft	3.00 in.^{2}/ft
		Direct tension 46k/ft moment 215 k-ft/ft	2.21 in.^{2}/ft (2)	2.53 in.^{2}/ft
Crane Beam	3	Torsion 2,280 k-ft	Torsional stirrup 0.74 in.^{2}/ft Longitudinal reinf. 18.3 in.^{2}	1.00 in.^{2}/ft 34.3 in.^{2}

(1) Load combination equations correspond to equations in Appendix B.
(2) Governed by minimum code reinforcement requirements.

TABLE 6

FACTORS OF SAFETY FOR STRUCTURAL STABILITY

Load[(1)(3)] Combination	Overturning Factor of Safety			Sliding Factor of Safety		Flotation Factor of Safety	
	Minimum Required	Calculated: Equivalent Static	Calculated: Energy Balance	Minimum Required	Calculated	Minimum Required	Calculated
D + H + E	1.5	1.6	See note (2)	1.5	1.67	-	-
D + H + E'	1.1	1.3	184	1.1	1.34	-	-
D + F'	-	-	-	-	-	1.1	15

(1) D = Dead weight of structure
H = Lateral earth pressure
E = OBE
E' = SSE
F' = Buoyant force

(2) The factor of safety for the SSE load case also satisfies the minimum required factor of safety for the OBE case.

(3) Lateral loads caused by design wind, tornado, and blast are less in magnitude than lateral loads caused by design OBE and SSE.

TABLE 7

TORNADO MISSILE ANALYSIS RESULTS[1]

Panel Description and Location	Panel Size Length (ft)	Width (ft)	Thickness (ft)	Computed Ductility Ratio	Allowable Ductility Ratio
Level 4 Roof Area Between Lines F_1 and F_2 and F_A and F_B	73	26	2	2.0	10
Level 4 Roof Area Between Lines F_3 and F_4 and F_A and F_B	73	47	2	1.3	10
Level 3 Exterior Wall Along F_B Line	26	24.5	3	1.0	10
Level 3 Exterior Wall Along F_1 Line	73	24.5	3	1.2	10

(1) Governing combination of tornado load effects is $W_t = W_{tq} + 0.5\ W_{tp} + W_{tm}$

TABLE 8

MAXIMUM FOUNDATION BEARING PRESSURES[1]

Gross Static (ksf)	Net Static (ksf)	Gross Dynamic (ksf)	Net Dynamic (ksf)	Allowable Net[2] Bearing Capacity		Computed Factor[3] of Safety	
				Static (ksf)	Dynamic (ksf)	Static	Dynamic
8.1	0.1	23.4	15.4	21.3	32.0	640	4.2

(1) Maximum foundation bearing pressures are defined as follows:

Gross Static = Total structure dead load plus operating live load divided by total basemat area.

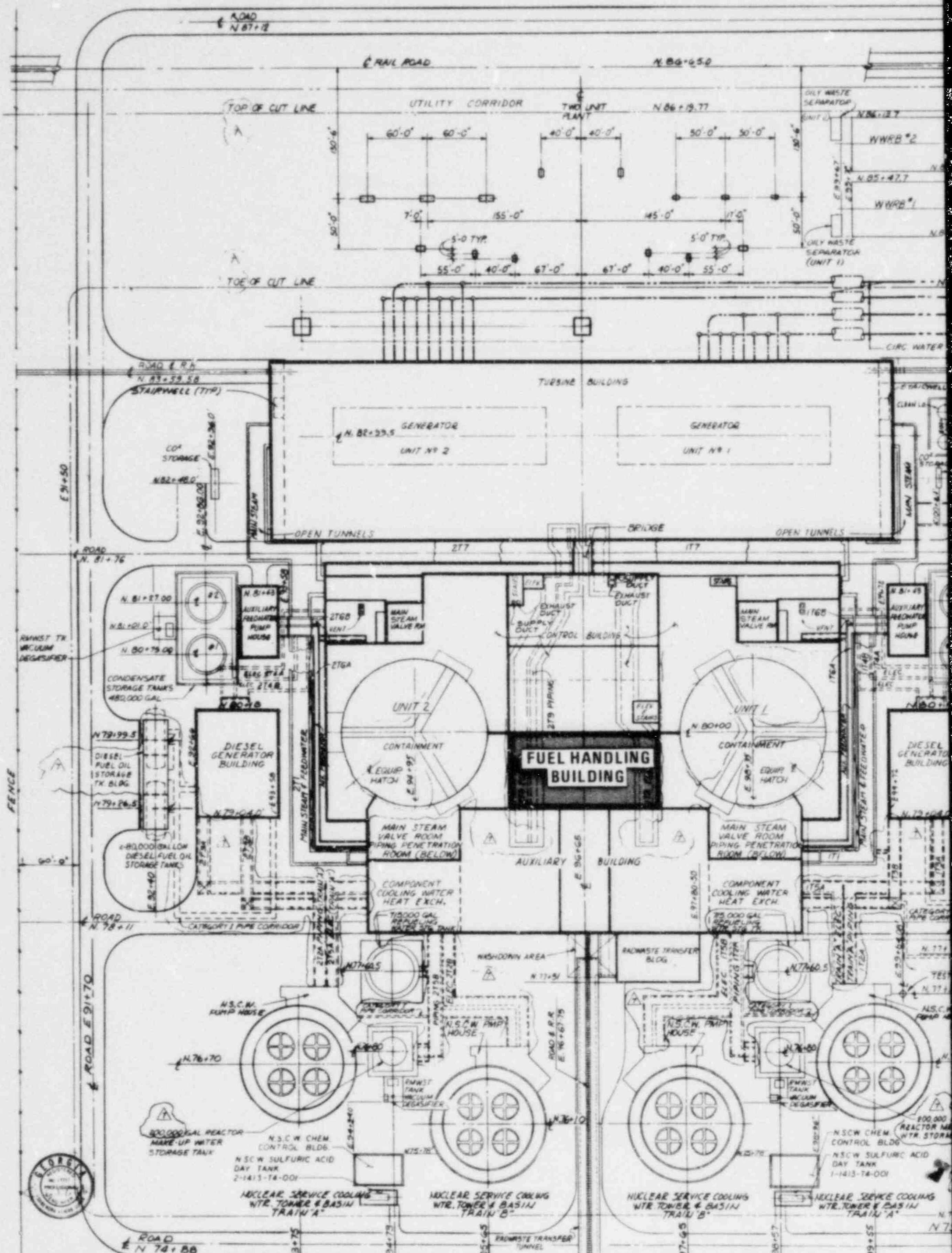
Net Static = The static pressure in excess of the overburden pressure at the base of the structure.

Gross Dynamic = Maximum soil pressure under dynamic loading conditions (i.e., unfactored SSE).

Net Dynamic = The dynamic pressure in excess of the overburden pressure at the base of the structure.

(2) The allowable net static and dynamic bearing capacities are obtained by dividing the ultimate net bearing capacity by factors of 3 and 2 respectively. The ultimate net bearing capacity is the pressure in excess of the overburden pressure at the foundation level at which shear failure may occur in the foundation stratum.

(3) The computed factor of safety is the ultimate net bearing capacity divided by the net static or net dynamic bearing pressure.



ROAD N 87+12
℄ RAIL ROAD
N 86+65.0
TOP OF CUT LINE
UTILITY CORRIDOR
TWO UNIT PLANT
N 86+19.77
OILY WASTE SEPARATOR
WWRB #2
N 85+47.7
WWRB #1
OILY WASTE SEPARATOR (UNIT 1)
TOE OF CUT LINE
CIRC WATER
ROAD & R.R. N 83+53.56
STAIRWELL (TYP)
TURBINE BUILDING
GENERATOR
UNIT Nº 2
UNIT Nº 1
CO2 STORAGE
E 94+50
MAIN STEAM
OPEN TUNNELS
BRIDGE
ROAD N. 81+76
2T7
1T7
AUXILIARY FEEDWATER PUMP HOUSE
MAIN STEAM VALVE RM
SUPPLY DUCT
EXHAUST DUCT
CONTROL BUILDING
RWWST TK VACUUM DEGASIFIER
CONDENSATE STORAGE TANKS 480,000 GAL
UNIT 2
UNIT 1
CONTAINMENT
EQUIP HATCH
FUEL HANDLING BUILDING
DIESEL GENERATOR BUILDING
DIESEL FUEL OIL STORAGE TK BLDG
FENCE
MAIN STEAM VALVE ROOM PIPING PENETRATION ROOM (BELOW)
AUXILIARY BUILDING
480,000 GALLON DIESEL FUEL OIL STORAGE TANKS
COMPONENT COOLING WATER HEAT EXCH.
CATEGORY 1 PIPE CORRIDOR
ROAD N. 78+11
WASHDOWN AREA
RADWASTE TRANSFER BLDG
N.S.C.W. PUMP HOUSE
N.S.C.W. PMP HOUSE
ROAD E 91+70
RWWST TANK VACUUM DEGASIFIER
500,000 GAL REACTOR MAKE-UP WATER STORAGE TANK
N.S.C.W. CHEM. CONTROL BLDG.
N.S.C.W. SULFURIC ACID DAY TANK 2-1413-T4-001
N.S.C.W. SULFURIC ACID DAY TANK 1-1413-T4-001
NUCLEAR SERVICE COOLING WTR. TOWER & BASIN TRAIN "A"
NUCLEAR SERVICE COOLING WTR. TOWER & BASIN TRAIN "B"
ROAD N 74+88
RADWASTE TRANSFER TUNNEL
E 93+75
E 94+79
E 95+65
E 97+65
E 98+57
E 99+55

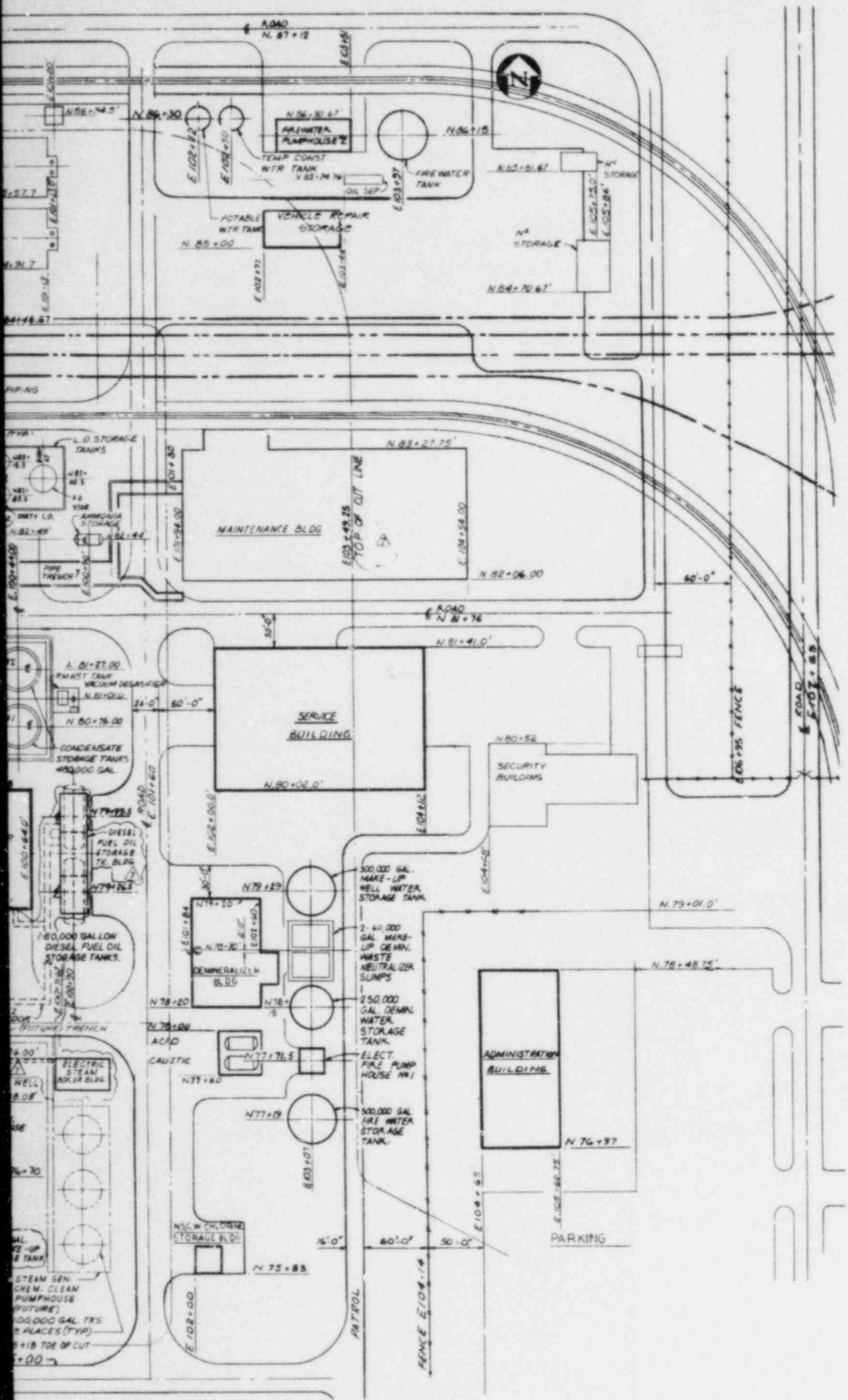


Figure 1
LOCATION OF FUEL
HANDLING BUILDING

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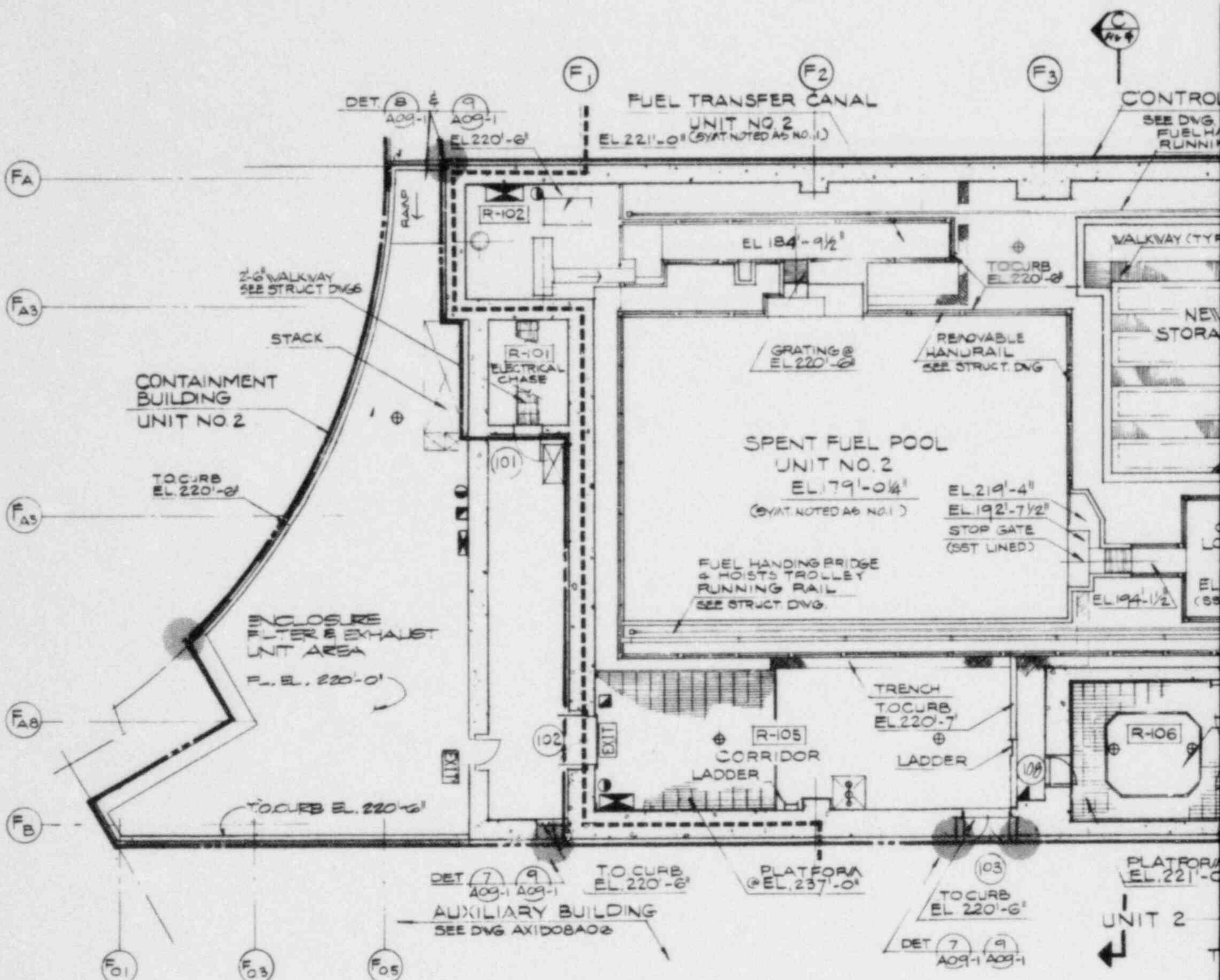


FUEL TRANSFER CANAL
UNIT NO. 2
EL. 221'-0" (SYM. NOTED AS NO. 1)
DET. 8 & 9 A09-1 A09-1
EL. 220'-6"
CONTRO
SEE DWG.
FUEL HA
RUNNI
R-102
RAMP
EL 184'-9½"
T.O.CURB EL. 220'-6"
WALKWAY (TYP
2'-6" WALKWAY SEE STRUCT DWGS
STACK
R-101 ELECTRICAL CHASE
GRATING @ EL. 220'-6"
REMOVABLE HANDRAIL SEE STRUCT. DWG
NEW STORA
CONTAINMENT BUILDING UNIT NO. 2
101
SPENT FUEL POOL UNIT NO. 2
EL. 179'-0¼"
(SYM. NOTED AS NO. 1)
T.O.CURB EL. 220'-6"
EL. 219'-4"
EL. 192'-7½"
STOP GATE (SST LINED)
FUEL HANDING BRIDGE & HOISTS TROLLEY RUNNING RAIL SEE STRUCT. DWG.
EL. 194'-1½
ENCLOSURE FILTER & EXHAUST UNIT AREA
FL. EL. 220'-0"
TRENCH
T.O.CURB EL. 220'-7"
R-105
CORRIDOR
LADDER
LADDER
R-106
102
EXIT
EXIT
T.O.CURB EL. 220'-6"
DET 7 9 A09-1 A09-1
T.O.CURB EL. 220'-6"
PLATFORM @ EL. 237'-0"
103
T.O.CURB EL. 220'-6"
DET 7 9 A09-1 A09-1
PLATFORM EL. 221'-0
UNIT 2
AUXILIARY BUILDING SEE DWG AX1D08A02

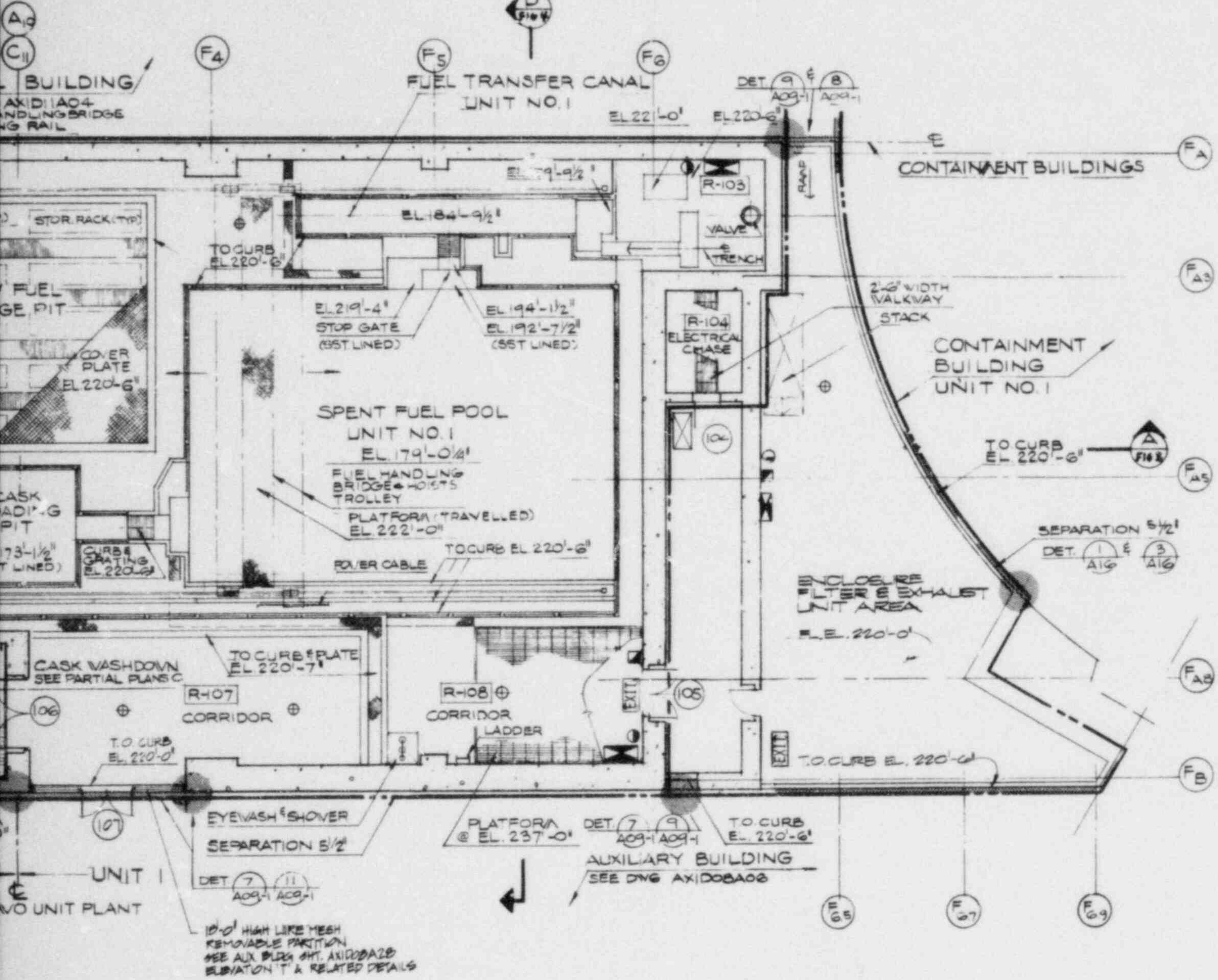


Figure 2
FUEL HANDLING BUILDING
FLOOR PLAN, EL. 220'-0"

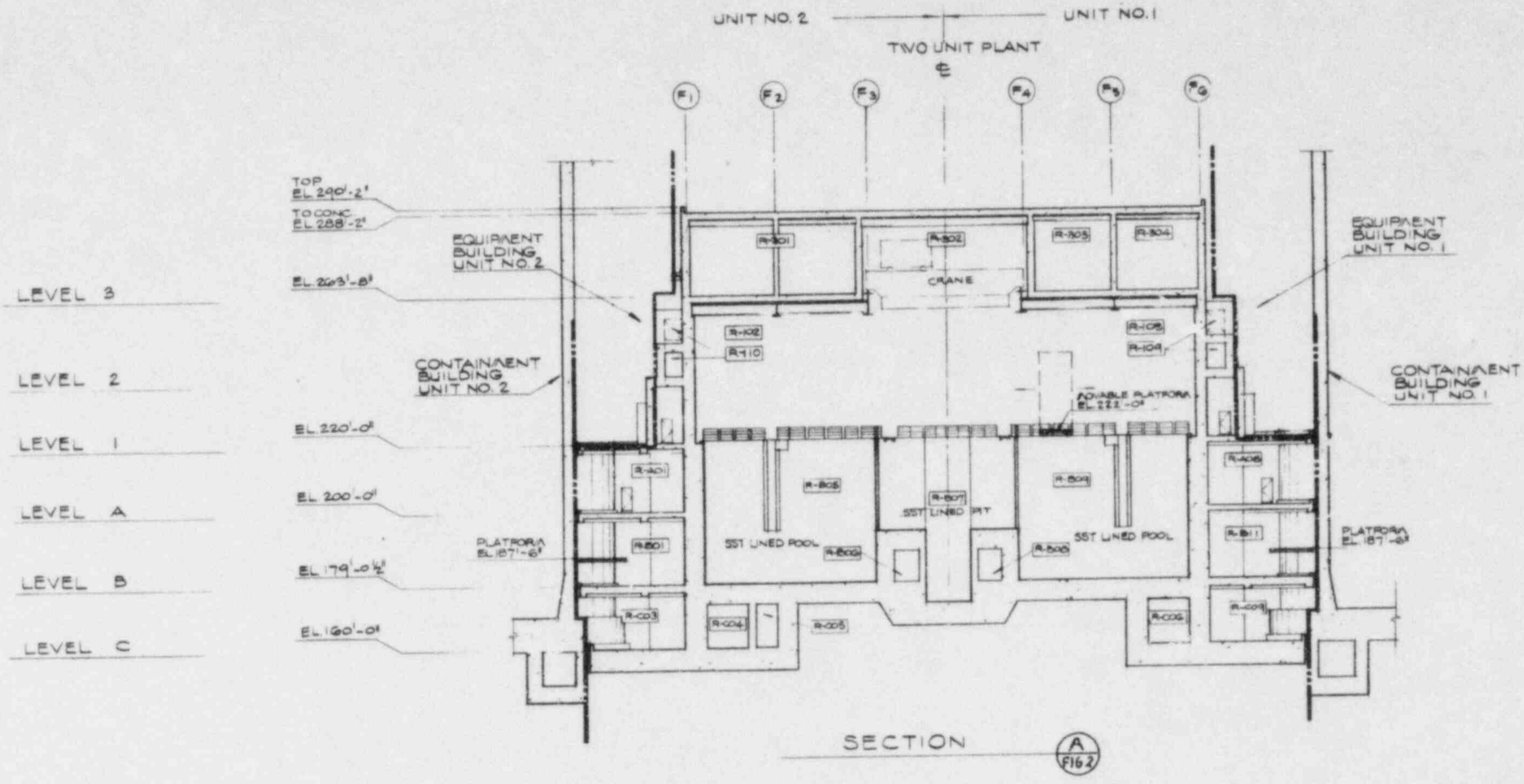


Figure 3
FUEL HANDLING BUILDING SECTION
LOOKING NORTH

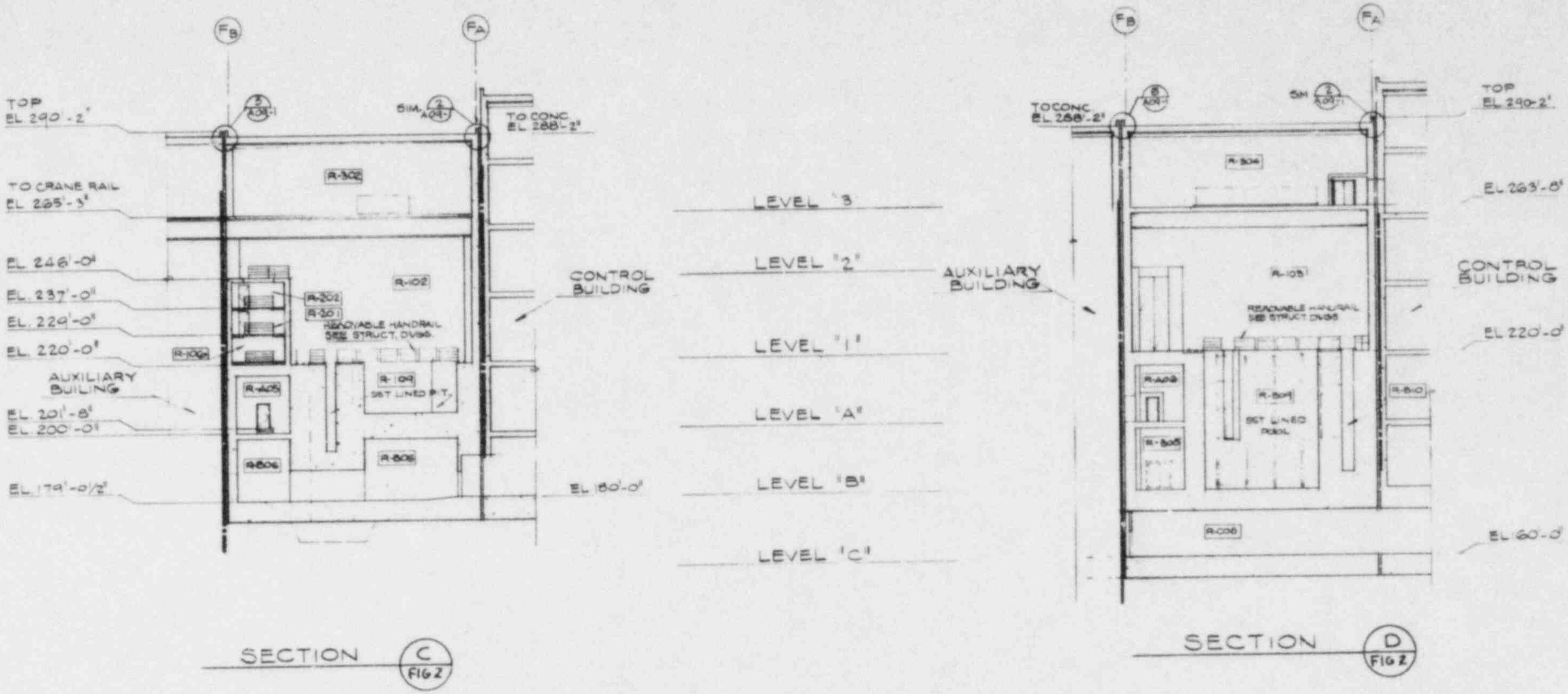
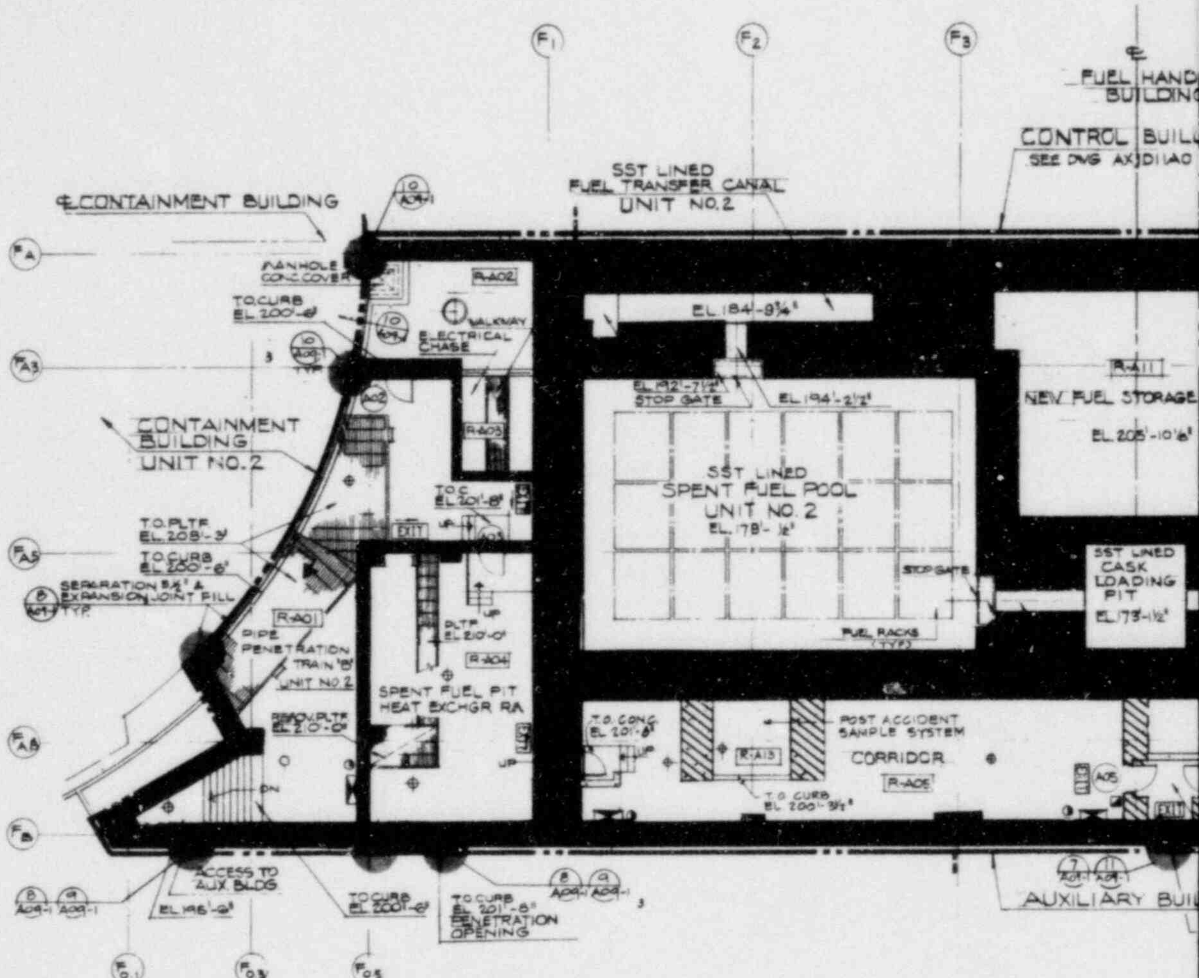


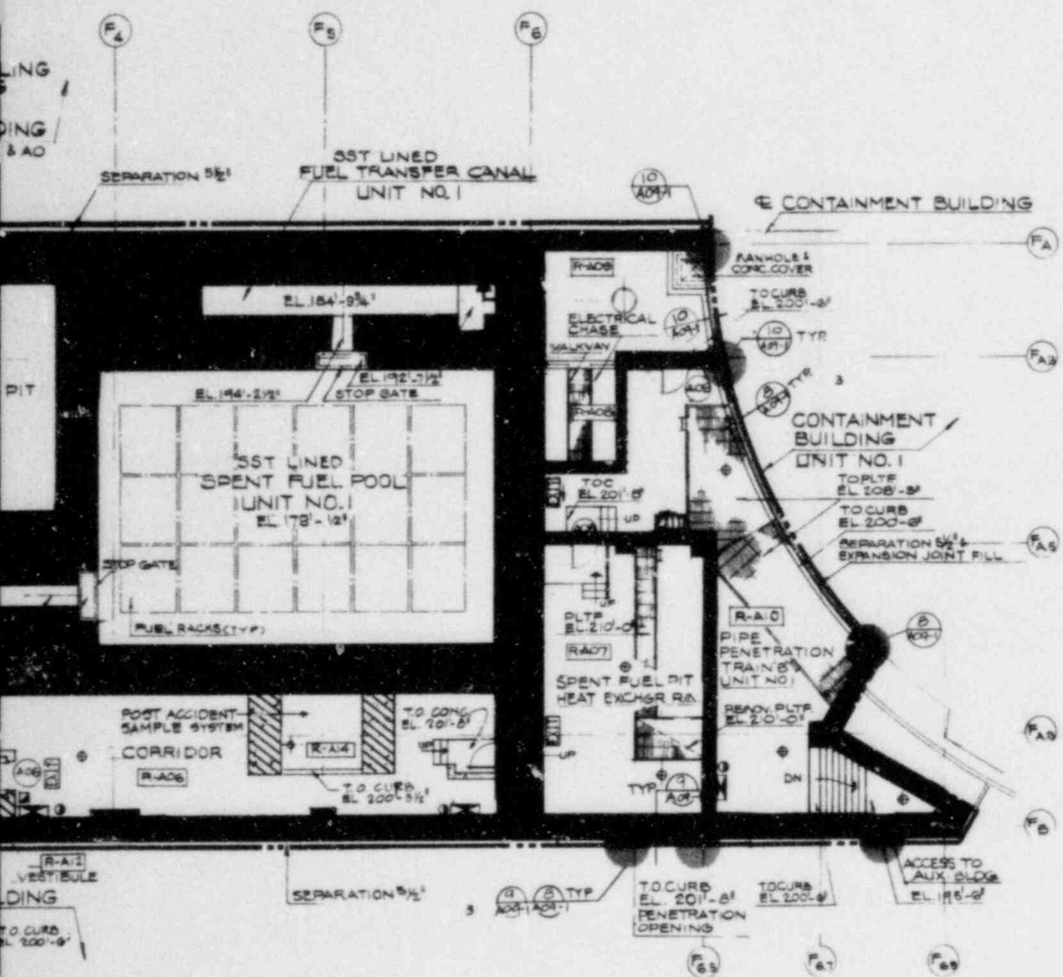
Figure 4
FUEL HANDLING BUILDING SECTIONS
LOOKING EAST



SLAB PANEL FRAMING PLAN E

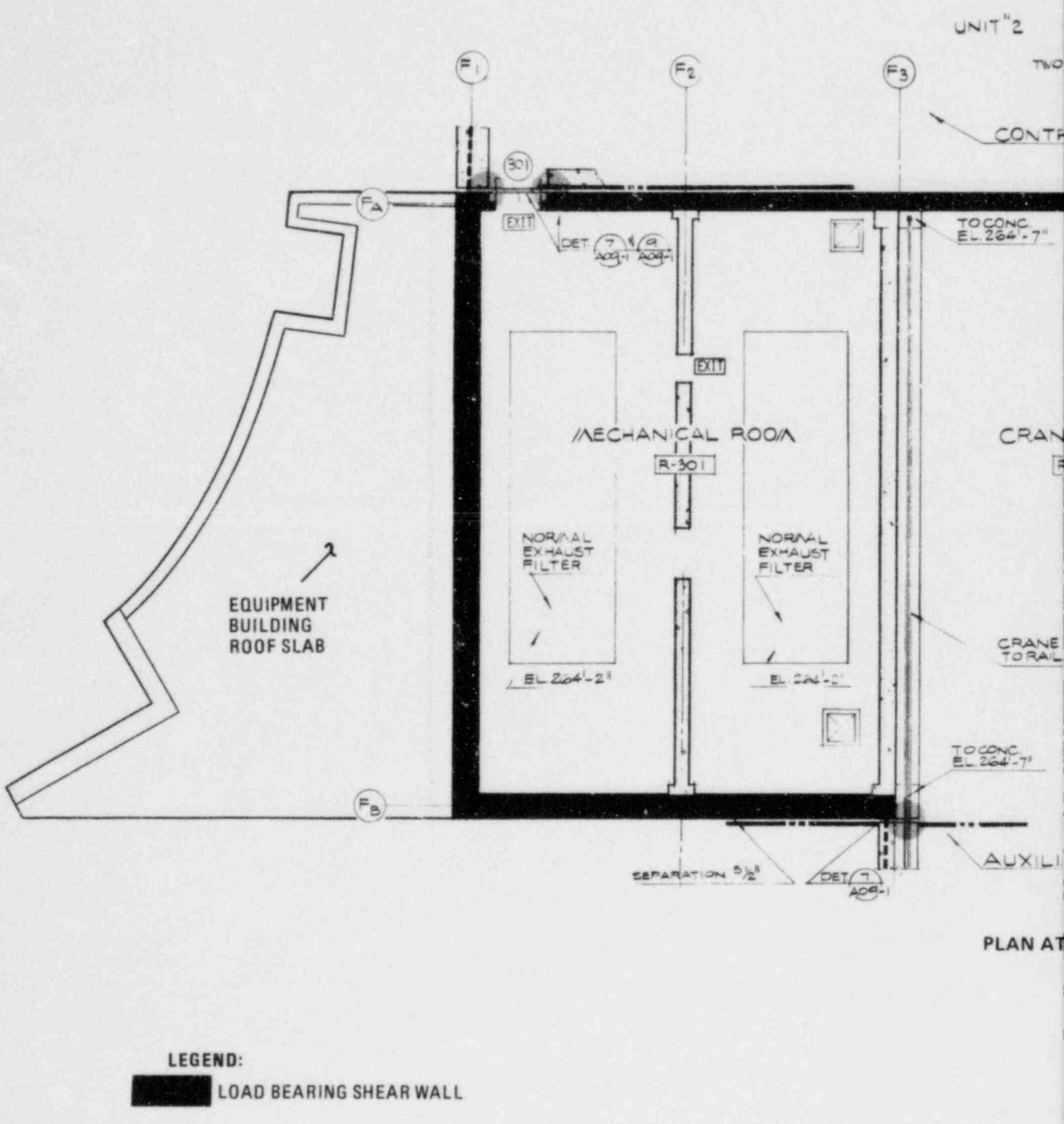
LEGEND:

- LOAD BEARING WALL
- LOAD BEARING SHEAR WALL



L. 200'-0" (UNO)

Figure 5
LVL A FLOOR PLAN SHOWING
LOCATION OF SHEAR WALLS



PLAN AT

LEGEND:

LOAD BEARING SHEAR WALL

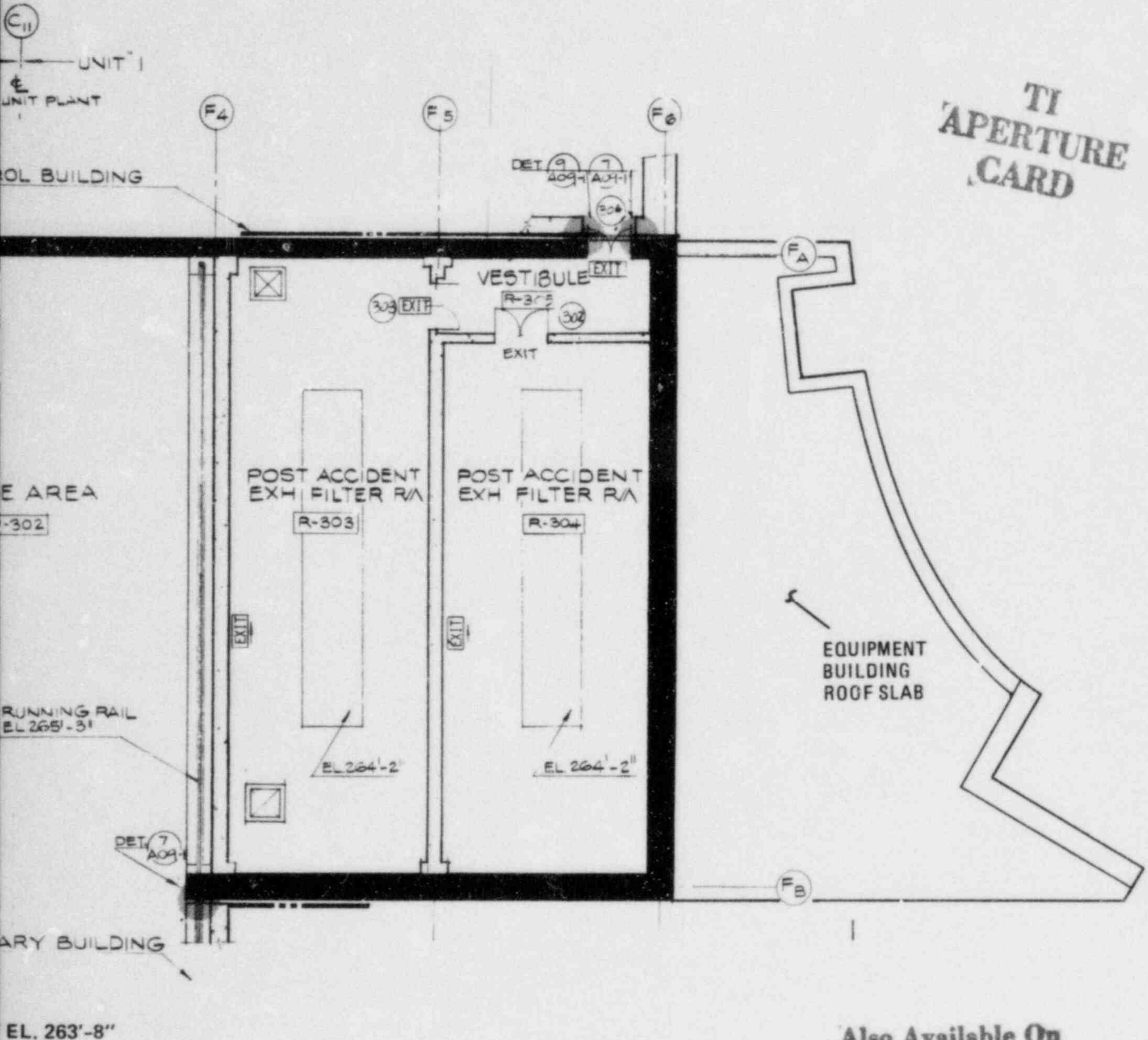


Figure 6
LVL 3 FLOOR PLAN SHOWING
LOCATION OF SHEAR WALLS

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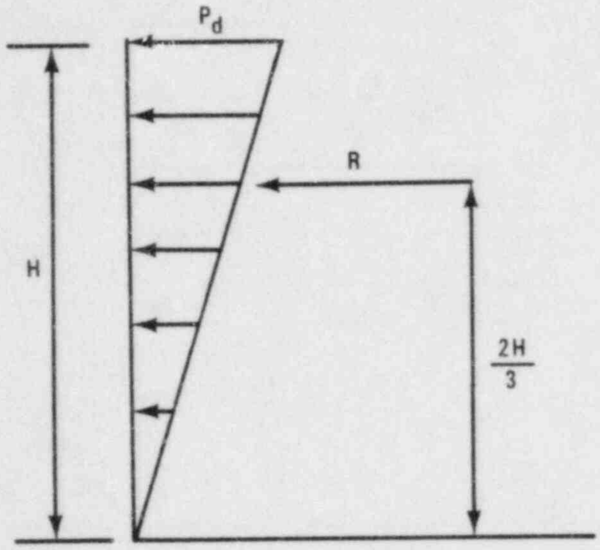


H: HEIGHT FROM BASE OF STRUCTURE TO SOIL SURFACE

P_d = DYNAMIC INCREMENTAL SOIL PRESSURE

R: RESULTANT FORCE

$R = .075\ V_m H^2$ (SSE)*

$= .045\ V_m H^2$ (OBE)*

$P_d = \frac{2R}{H}$

γ_m = SOIL MOIST UNIT WEIGHT, PCF

*DERIVED USING THE PEAK GROUND ACCELERATIONS OF 0.12g AND 0.20g FOR OBE AND SSE RESPECTIVELY.

Figure 7
DYNAMIC INCREMENTAL
SOIL PRESSURE PROFILE

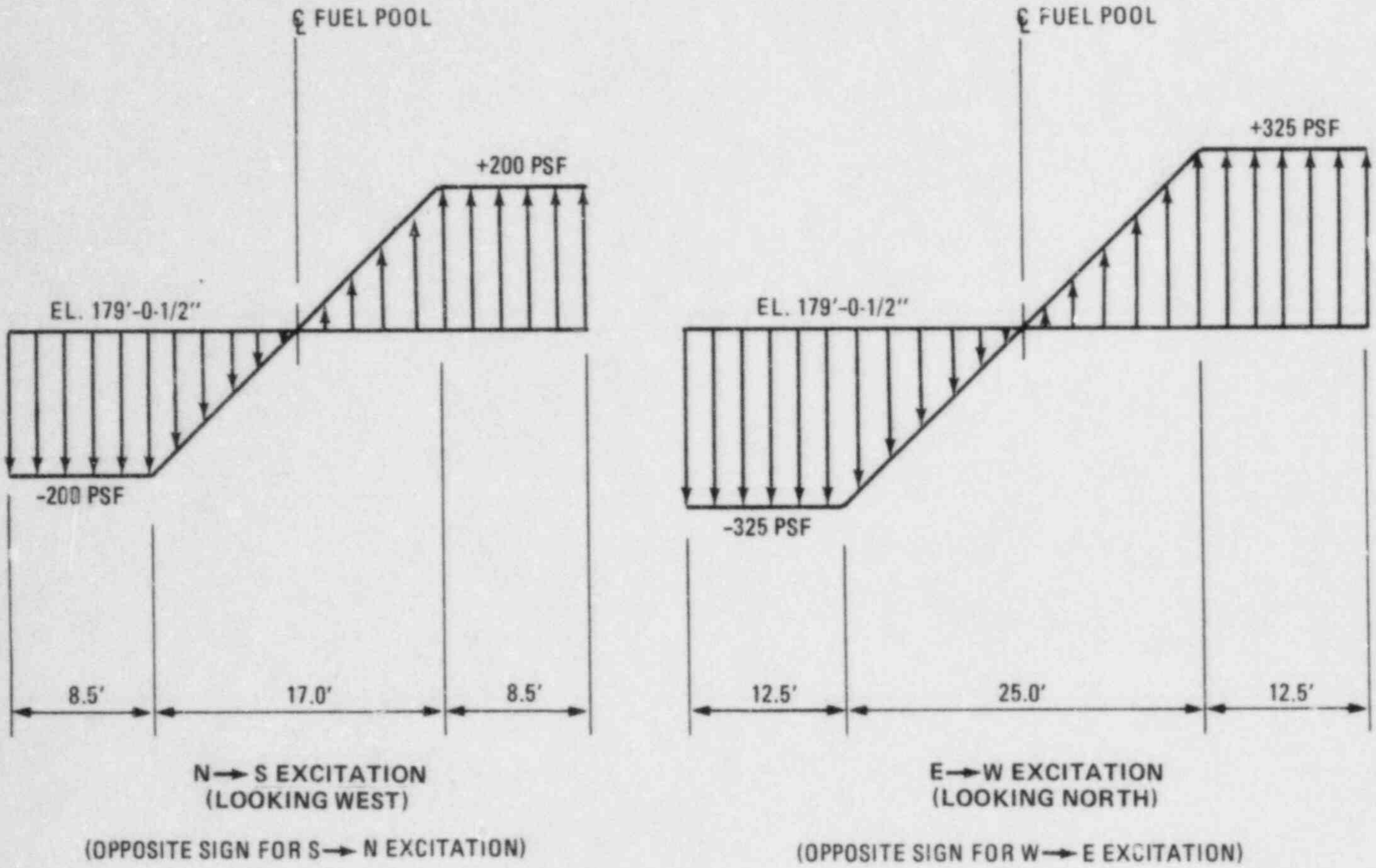


Figure 8
HYDRODYNAMIC PRESSURE PROFILES
ACTING ON BASEMAT UNDER OBE

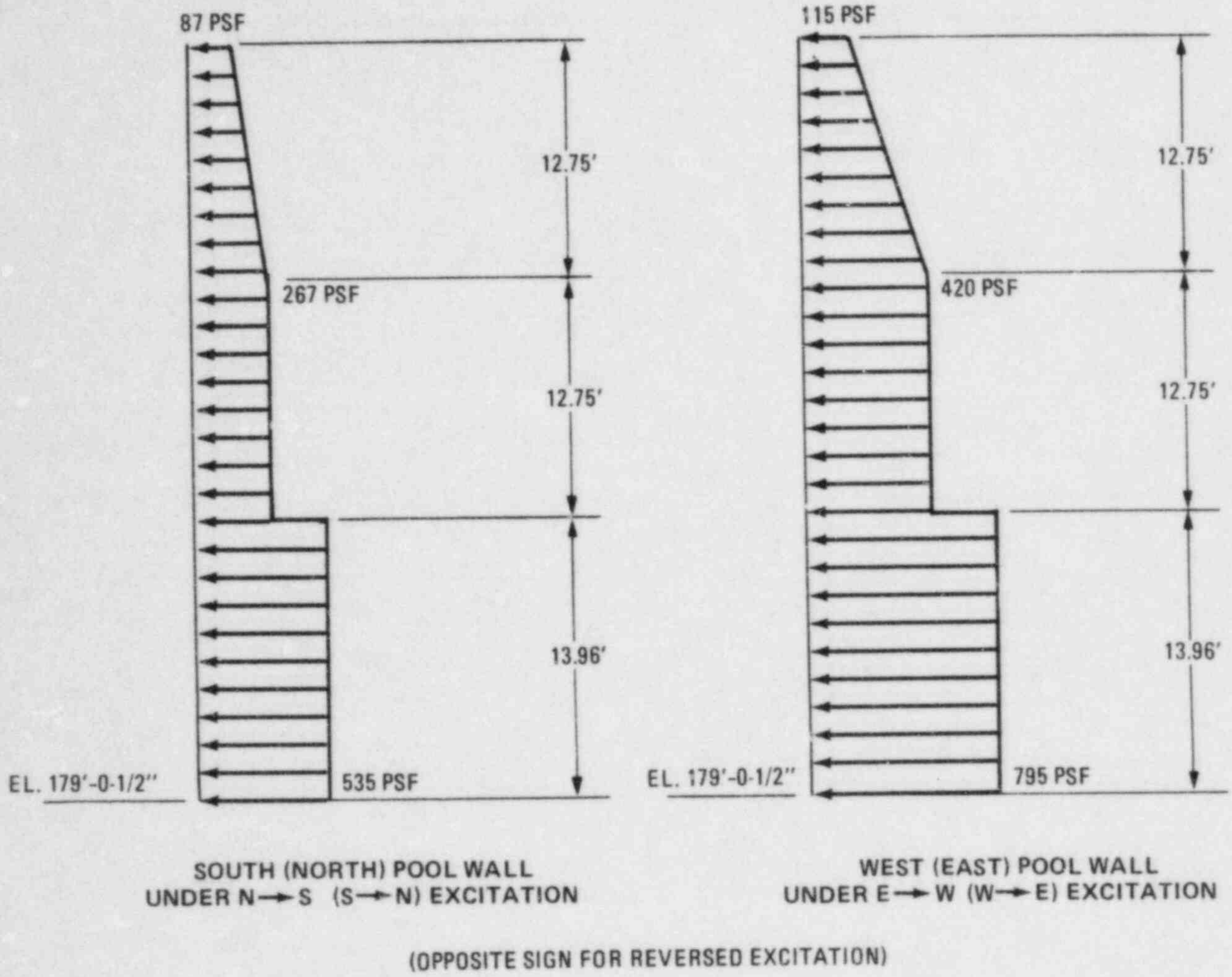


Figure 9
HYDRODYNAMIC PRESSURE PROFILES
ACTING ON POOL WALLS UNDER OBE

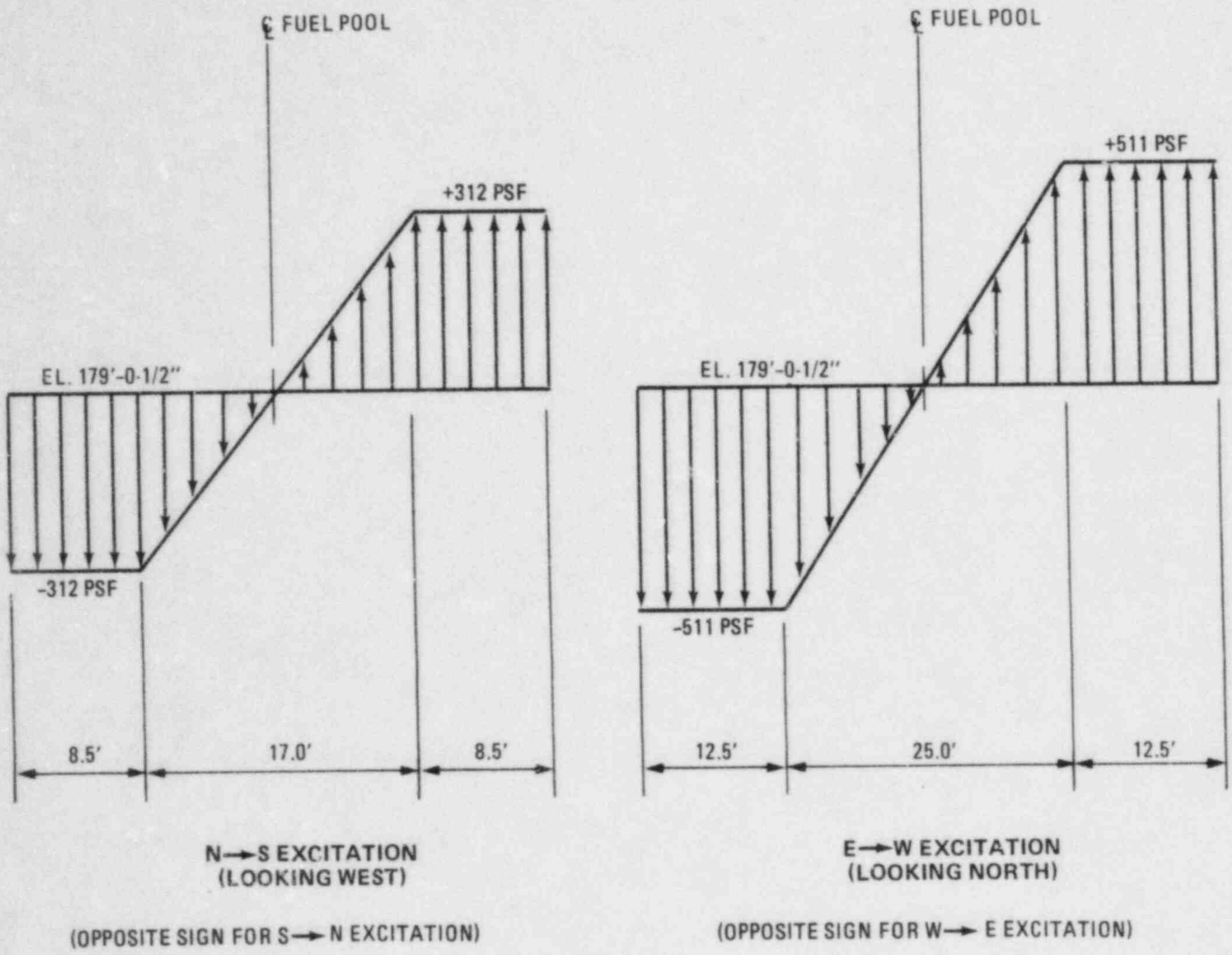


Figure 10
HYDRODYNAMIC PRESSURE PROFILES
ACTING ON BASEMAT UNDER SSE

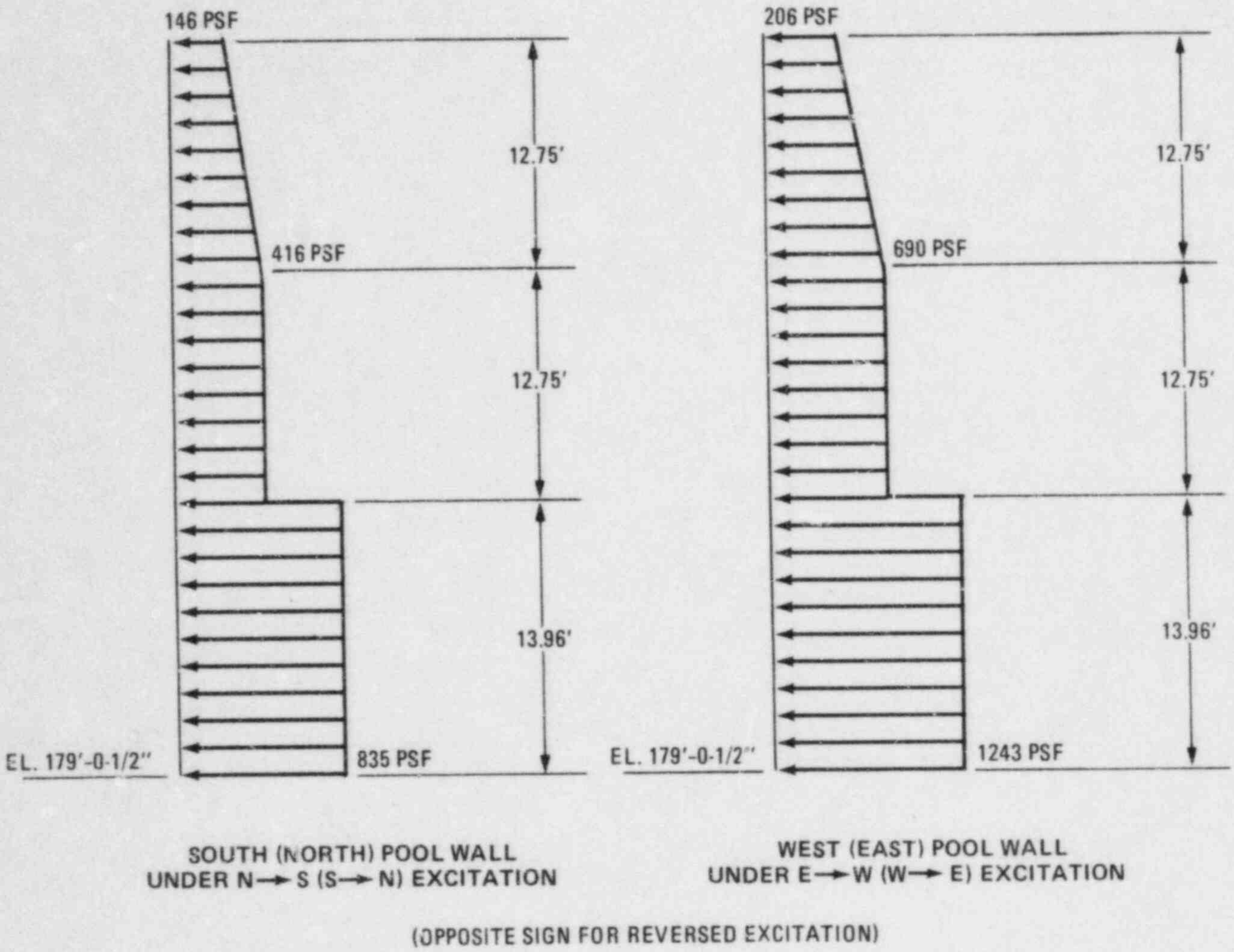
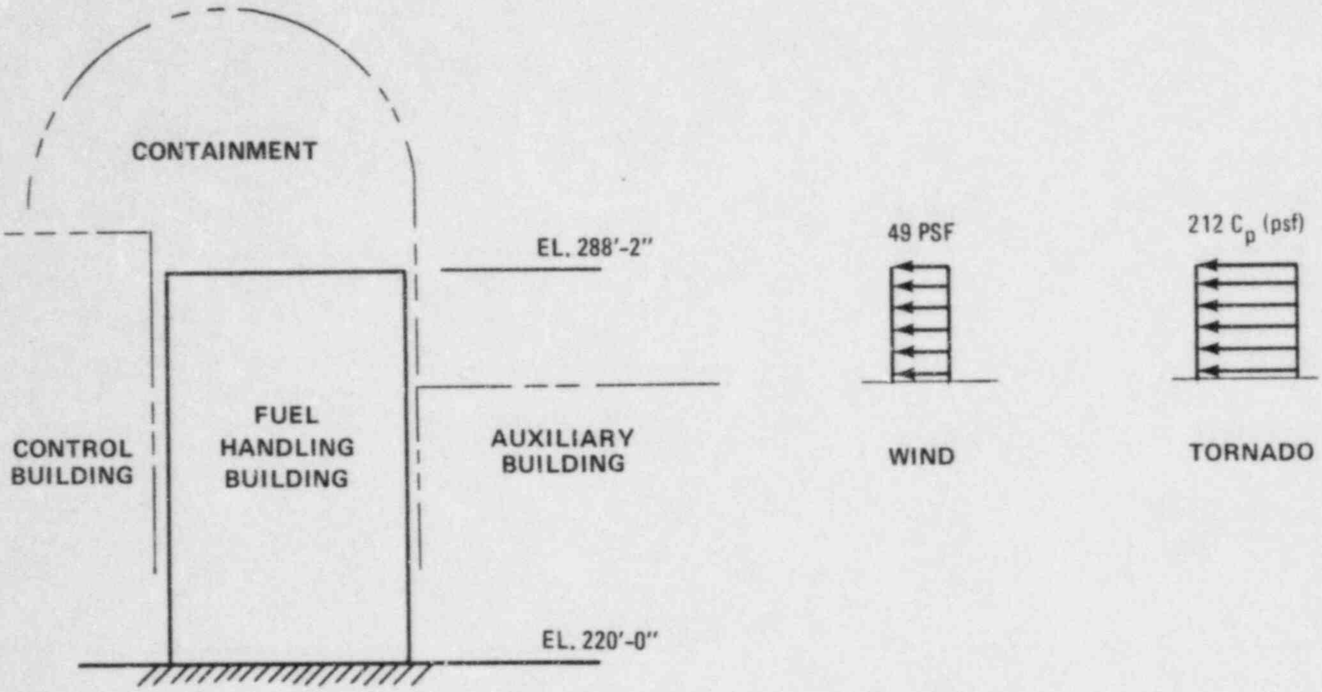


Figure 11
HYDRODYNAMIC PRESSURE PROFILES
ACTING ON POOL WALLS UNDER SSE



$P = C_s P_{max} C_p$

WHERE:

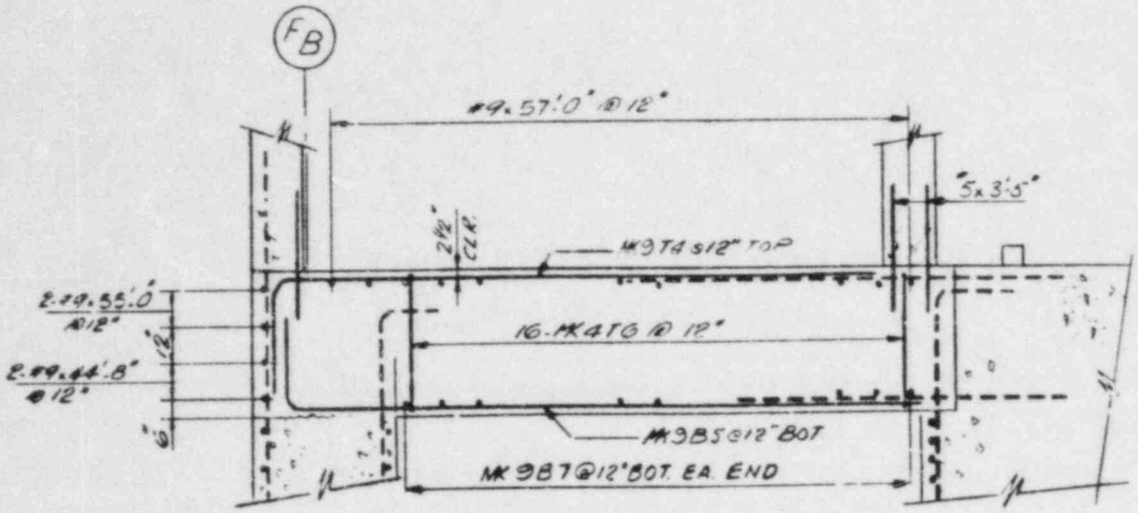
C_s = SIZE COEFFICIENT
= .64

$P_{max} = 0.00256\,(V_{max})^2$
$= 0.00256\,(360 \text{ mph})^2$
$= 332$ Psf

C_p = EFFECTIVE EXTERNAL PRESSURE COEFFICIENT (SEE FIG. 14)

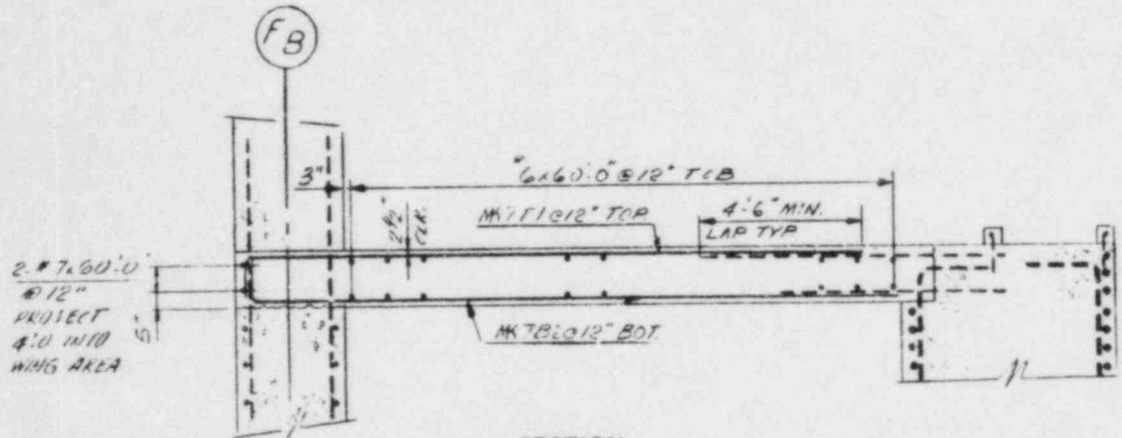
$P = (.64)\,(332 \text{ psf})\, C_p$
$= 212\, C_p$ (psf)

Figure 12
WIND AND TORNADO EFFECTIVE
VELOCITY PRESSURE PROFILES



SECTION

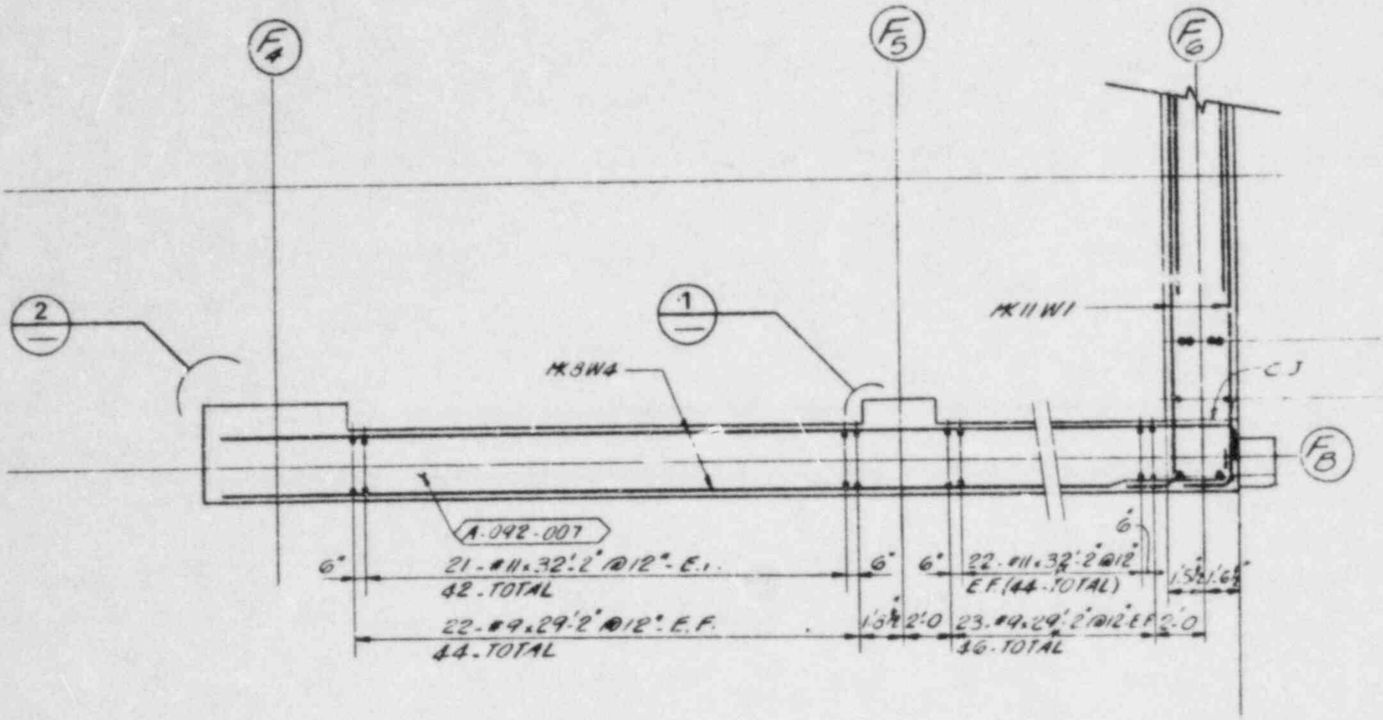
LEVEL 1 CASK WASHDOWN AREA SLAB



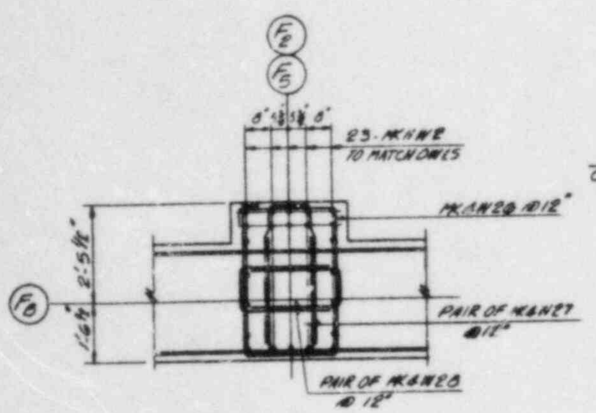
SECTION

LEVEL 1 FLOOR SLAB

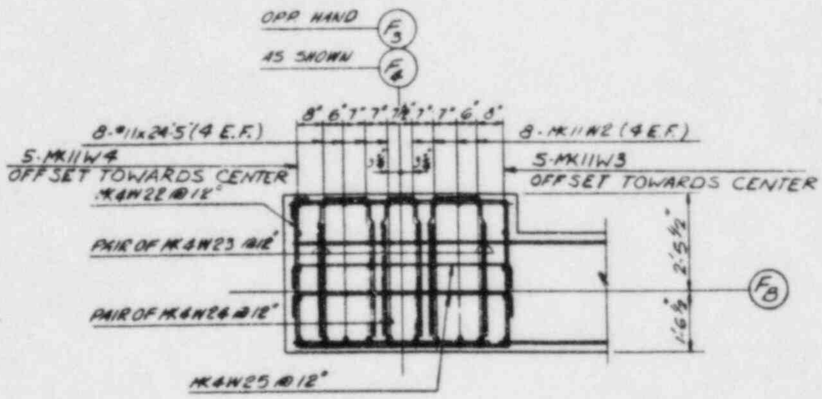
Figure 13
REPRESENTATIVE ROOF AND FLOOR SLAB DETAILS



PARTIAL PLAN AT ELEVATION 240'-0"



DETAIL 1



DETAIL 2

Figure 14
REPRESENTATIVE SHEAR WALL DETAILS

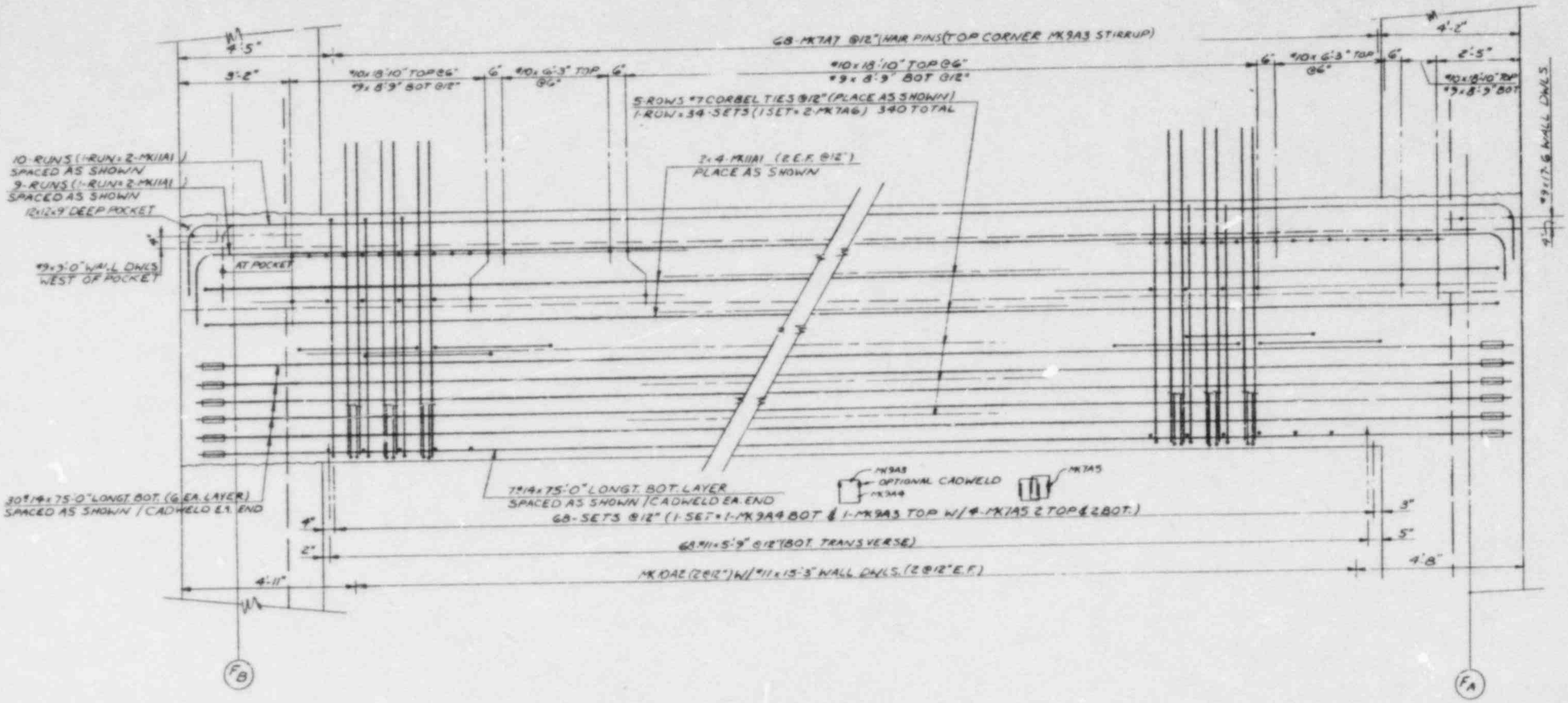


Figure 15
CASK CRANE BEAM ELEVATION

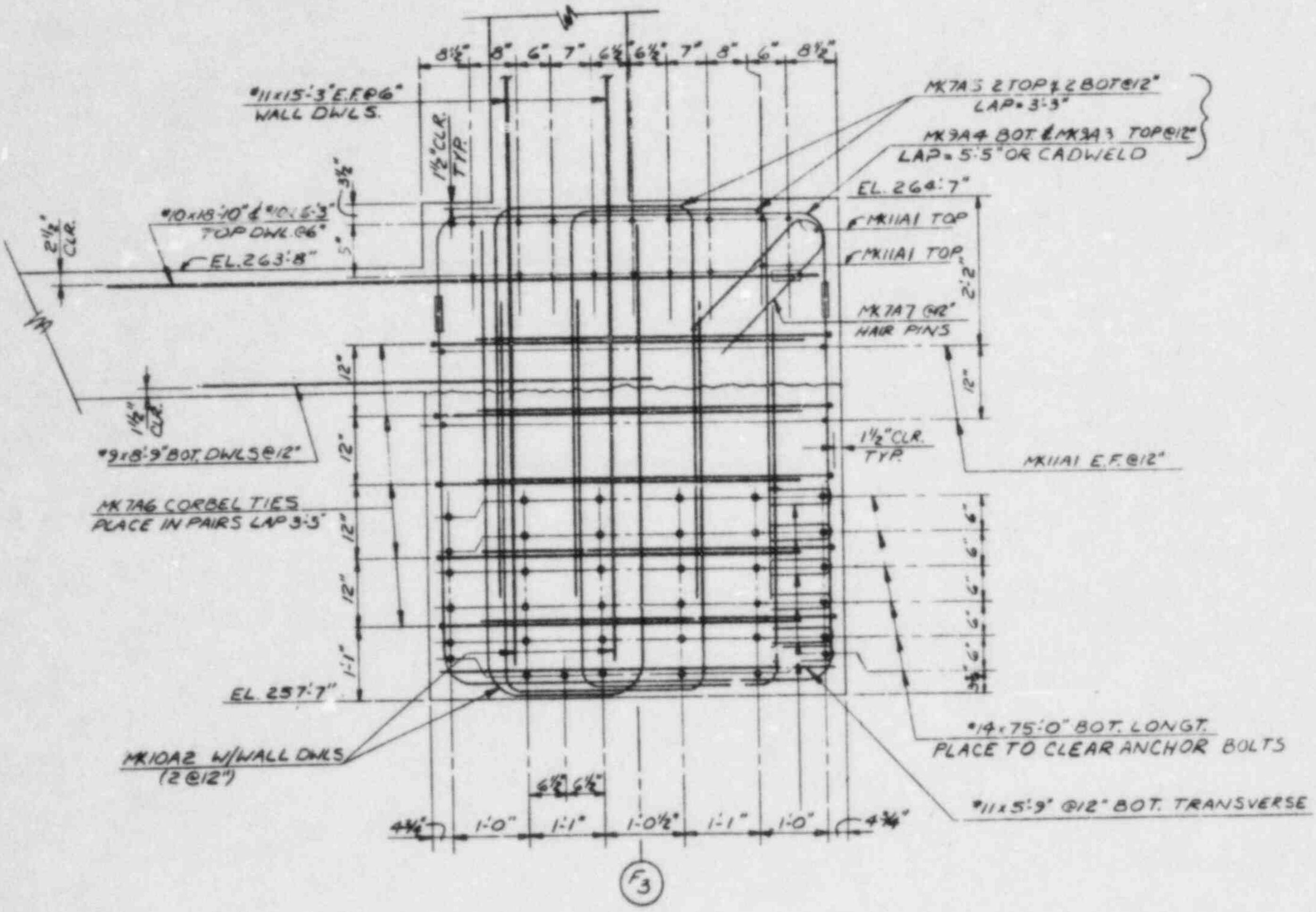
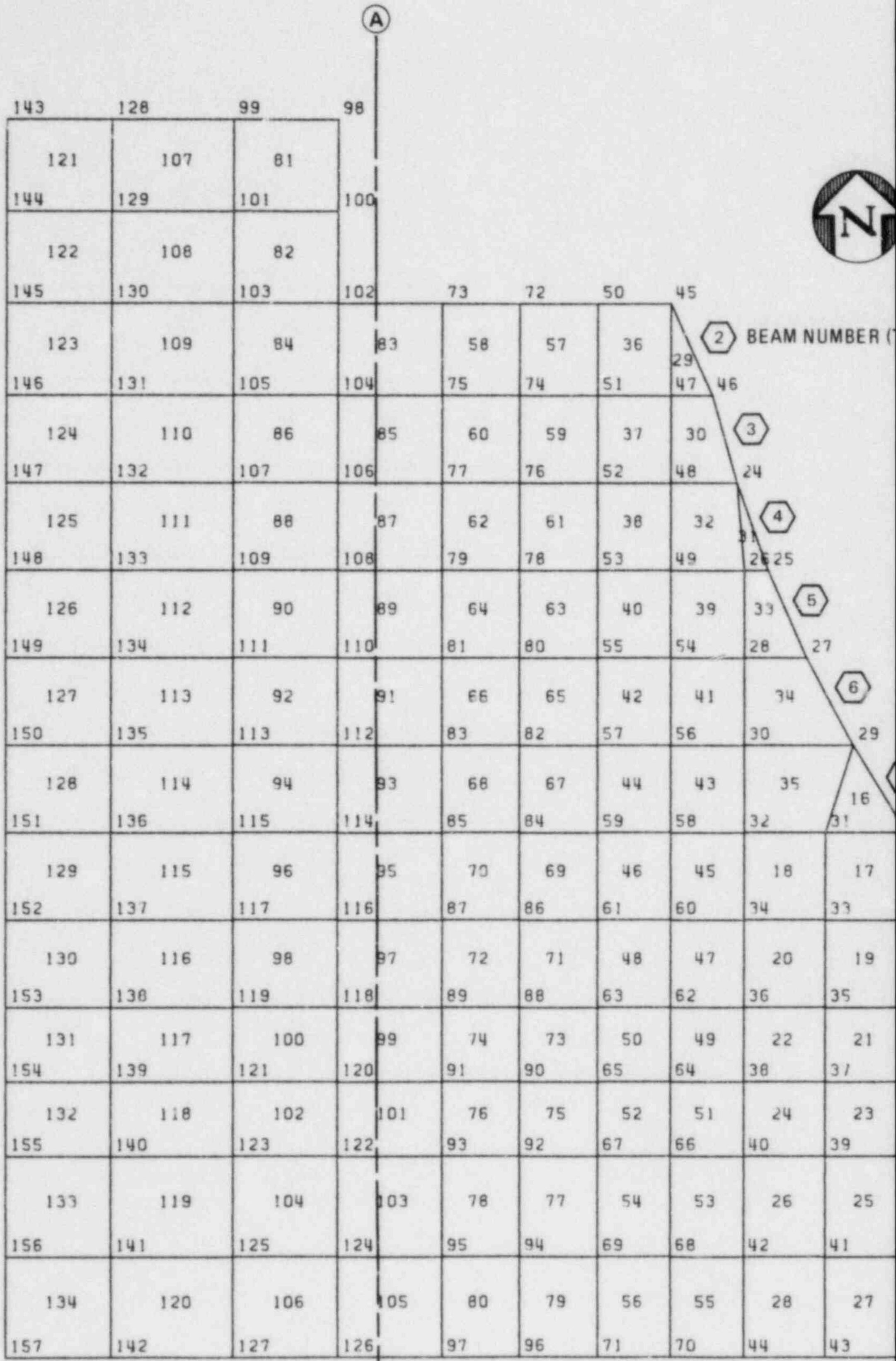
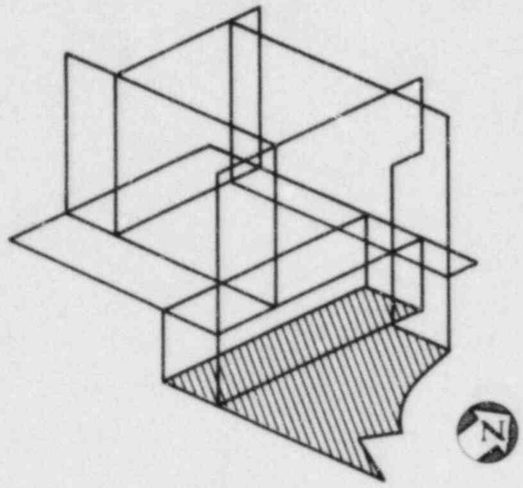


Figure 16
CASK CRANE BEAM SECTION



BASEMAT AT ELEVATION 157.00 FEET

FIGURE 19 SHEET 2 OF 2



KEY ISOMETRIC

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APERTURE
CARD

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Aperture Card

TYP.)

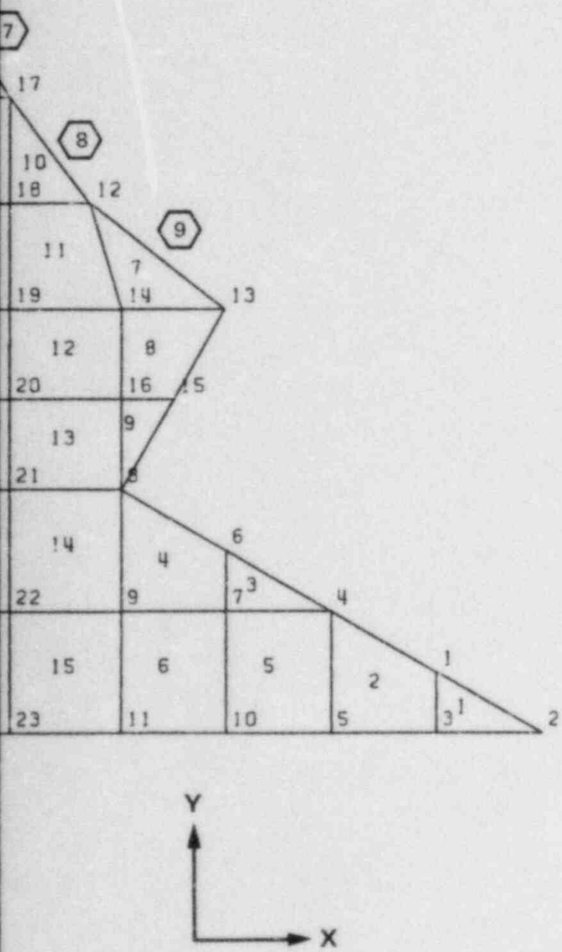
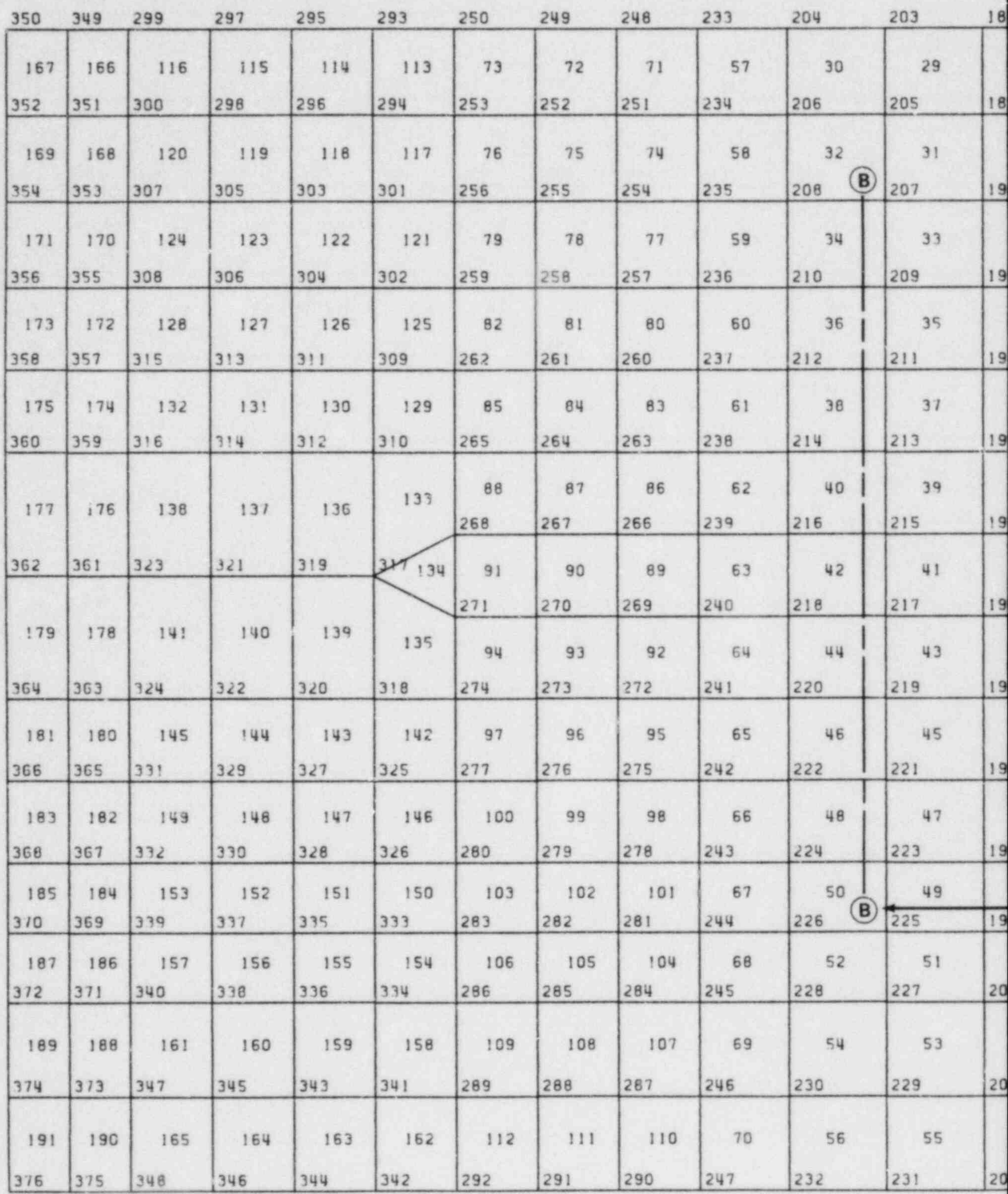
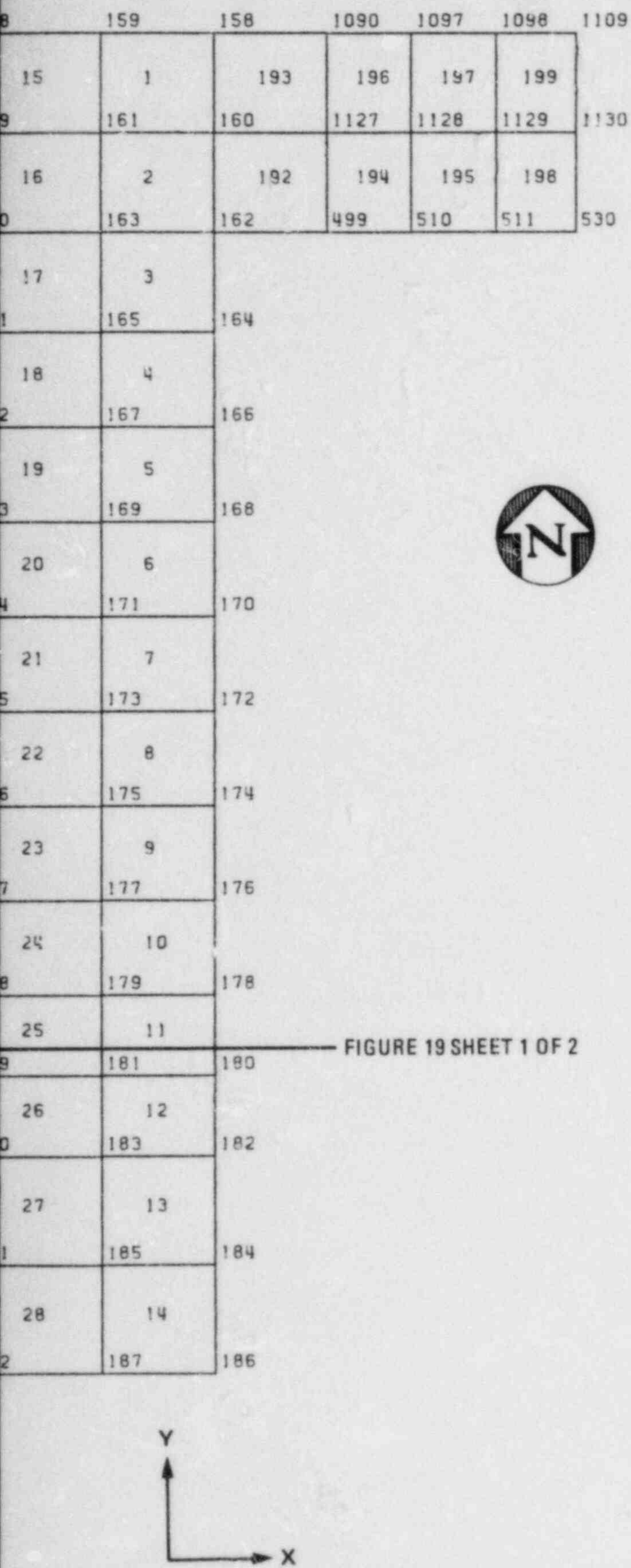


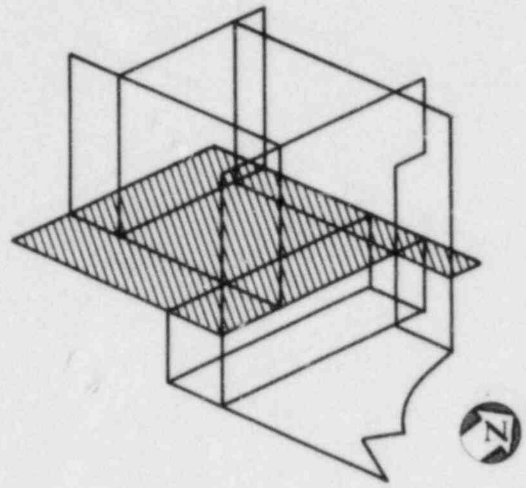
Figure 17
BASEMAT FINITE ELEMENT MODEL
(Sheet 1 of 6)

8411050186-05



BASEMAT AT ELEVATION 176.00 FEET





KEY ISOMETRIC

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APERTURE
CARD

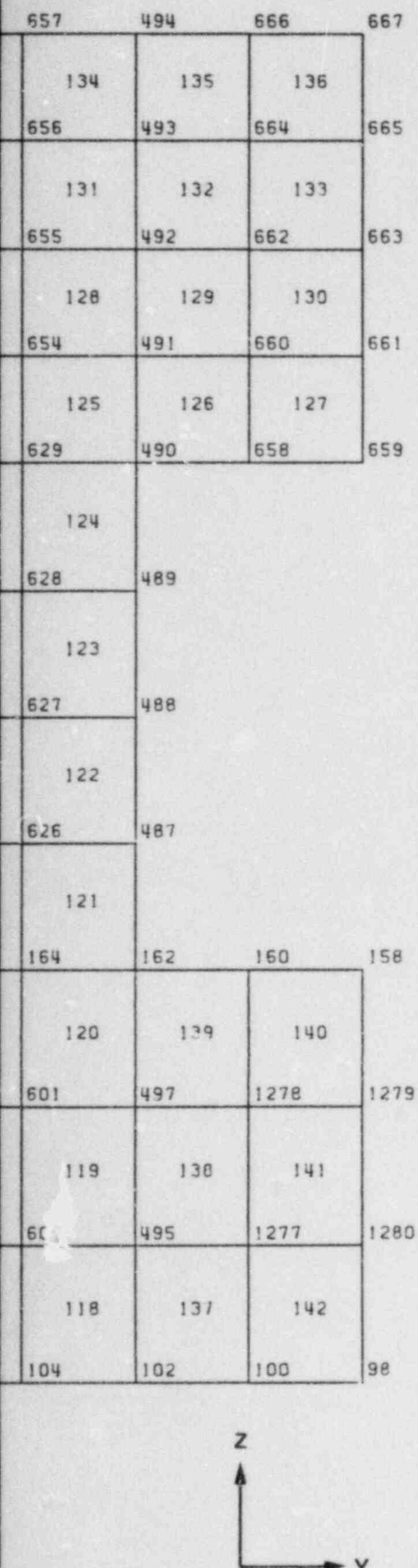
Also Available On
Aperture Card

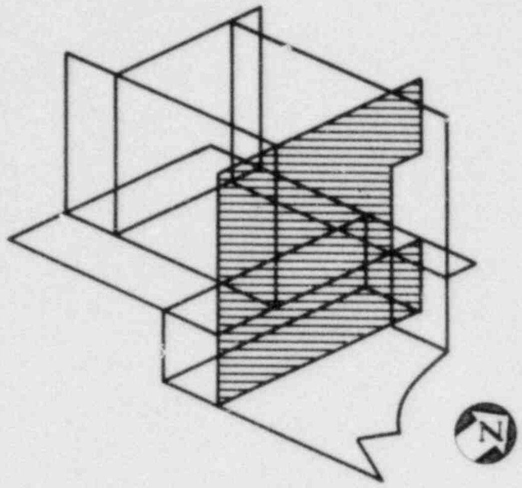
Figure 17
BASEMAT FINITE ELEMENT MODEL
(Sheet 2 of 6)

8411050186 -06

545	566	567	586	587	633	637	641	645	649	653
17 544	18 564	39 565	40 584	111 585	112 632	113 636	114 640	115 644	116 648	117 652
15 543	16 562	37 563	38 582	104 583	105 631	106 635	107 639	108 643	109 647	110 651
14 542	560	35 561	36 580	97 581	98 630	99 634	100 638	101 642	102 646	103 650
13 541	558	33 559	34 578	90 579	91 605	92 609	93 613	94 617	95 621	96 625
11 540	12 556	31 557	32 576	83 577	84 604	85 608	86 612	87 616	88 620	89 624
9 539	10 554	29 555	30 574	76 575	77 603	78 607	79 611	80 615	81 619	82 623
8 538	552	27 553	28 572	69 573	70 602	71 606	72 610	73 614	74 618	75 622
7 186	184	25 182	26 180	62 178	63 176	64 174	65 172	66 170	67 168	68 166
5 549	6 550	23 551	24 570	55 571	56 589	57 591	58 593	59 595	60 597	61 599
3 546	4 547	21 548	22 568	48 569	49 588	50 590	51 592	52 594	53 596	54 598
1 126	2 124	19 122	20 120	41 118	42 116	43 114	44 112	45 110	46 108	47 106

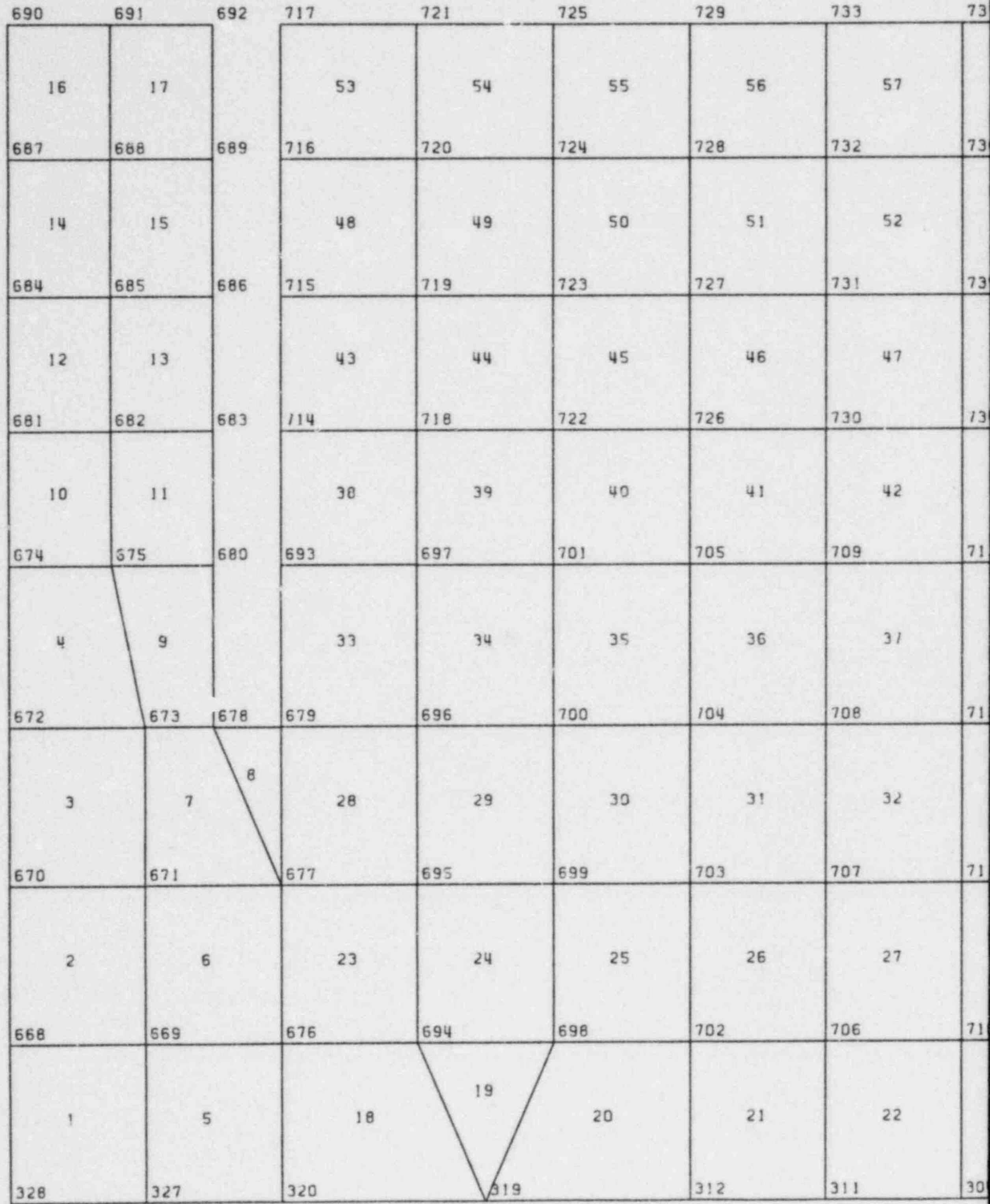
EAST WALL OF FUEL POOL (LOOKING WEST)



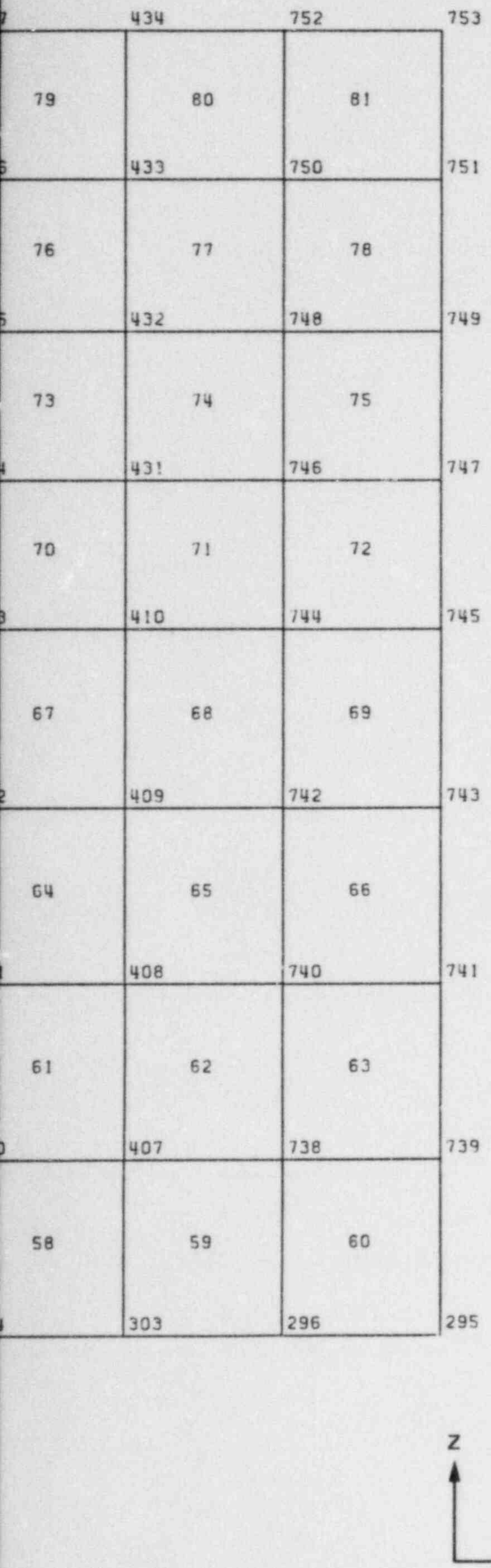


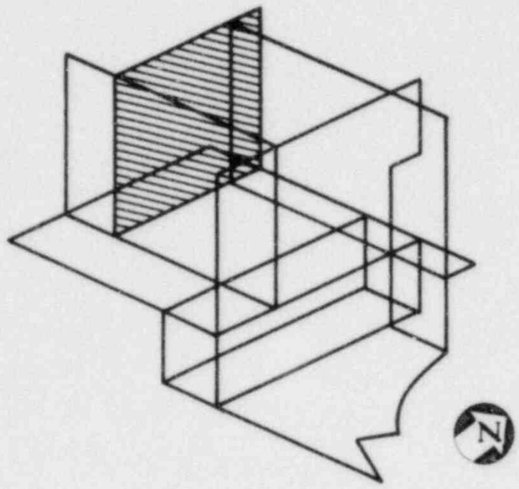
KEY ISOMETRIC

Figure 17
BASEMAT FINITE ELEMENT MODEL
(Sheet 3 of 6)



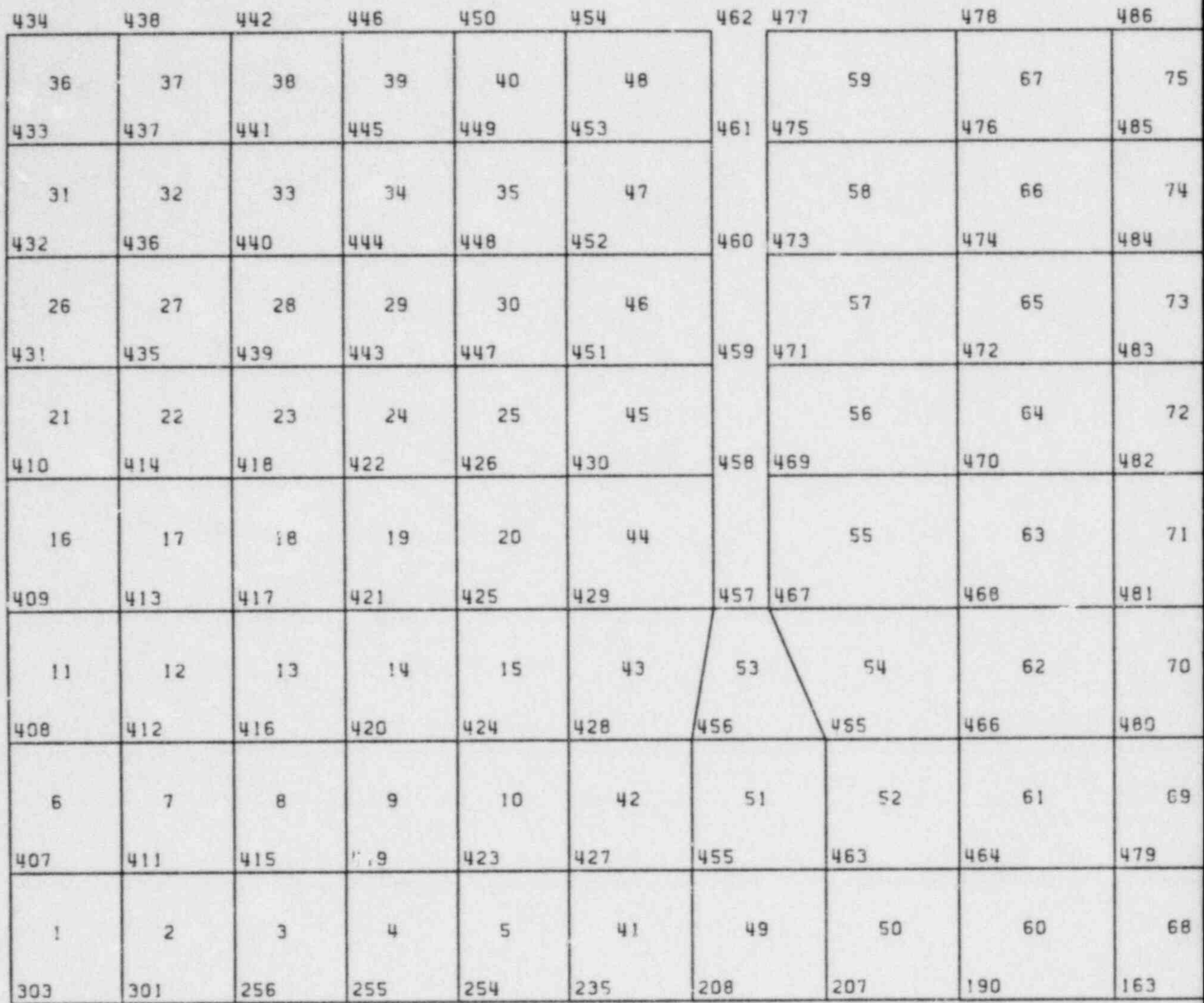
WEST WALL OF FUEL POOL (LOOKING WEST)



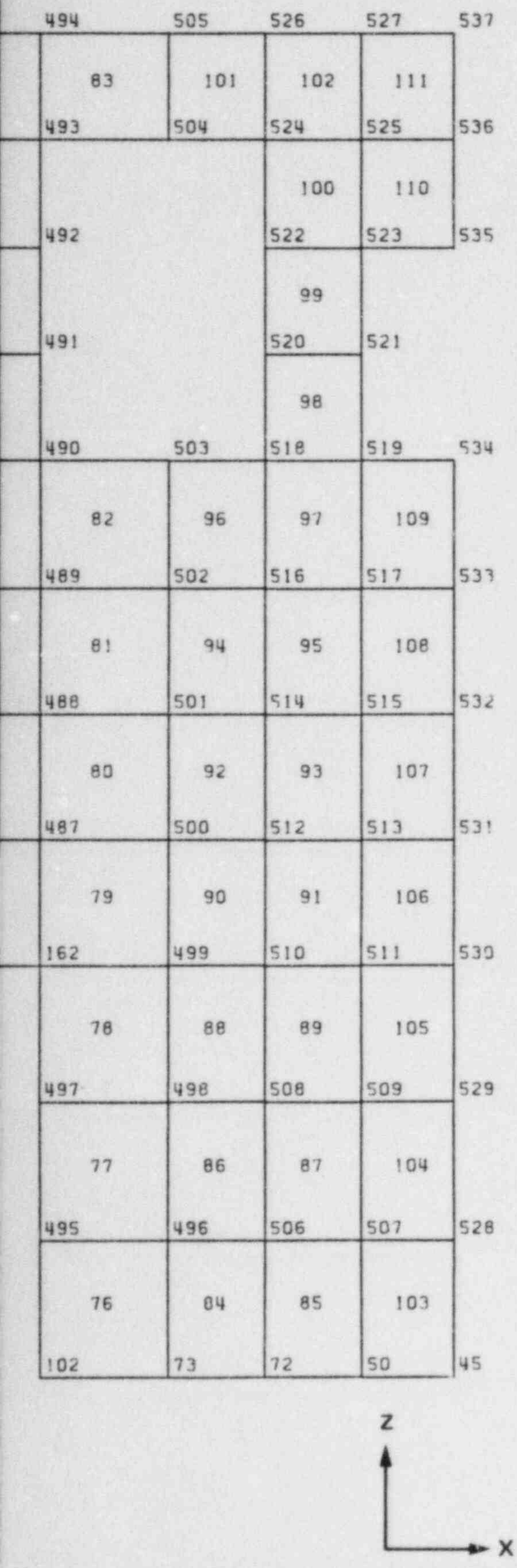


KEY ISOMETRIC

Figure 17
BASEMAT FINITE ELEMENT MODEL
(Sheet 4 of 6)



NORTH WALL OF FUEL POOL (LOOKING NORTH)



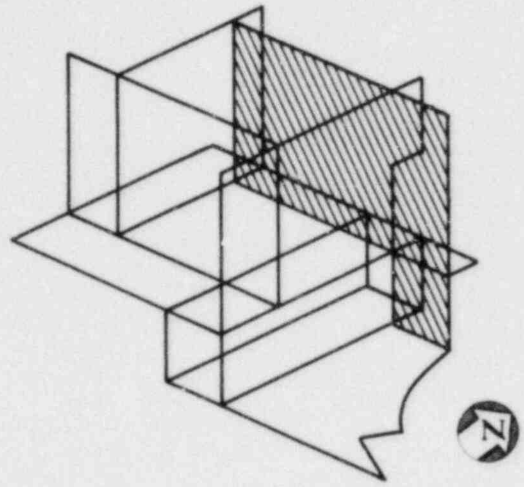


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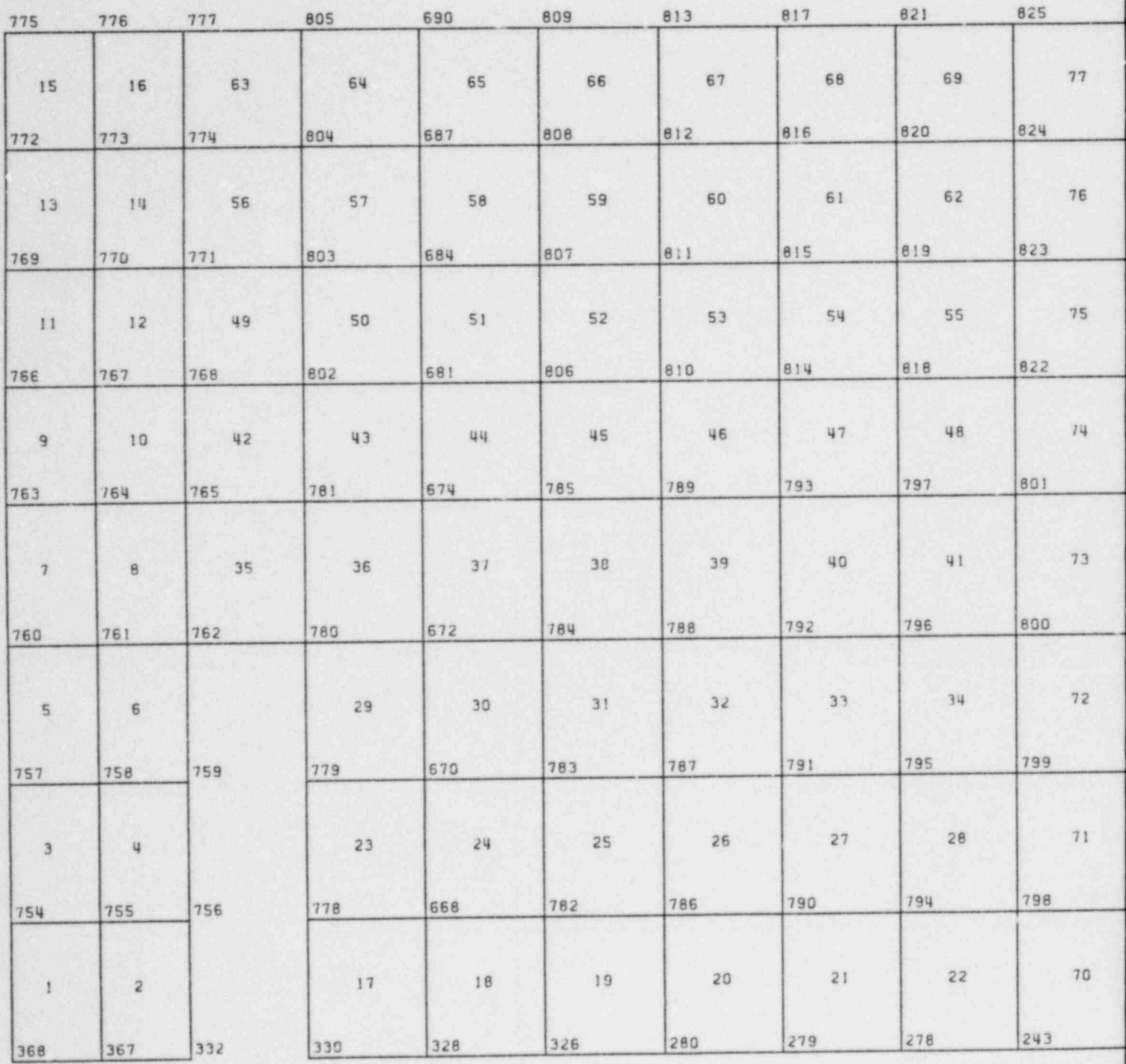
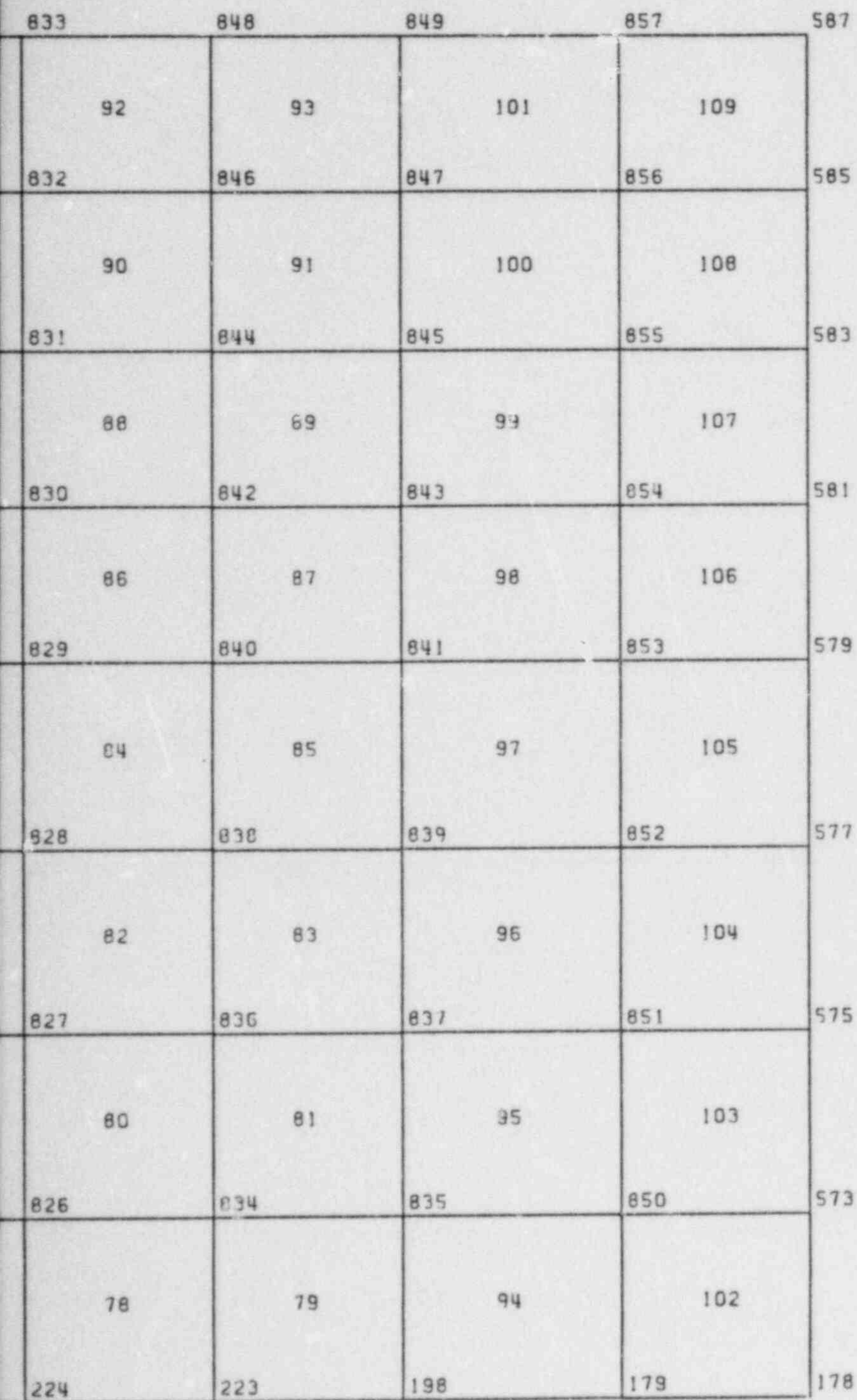
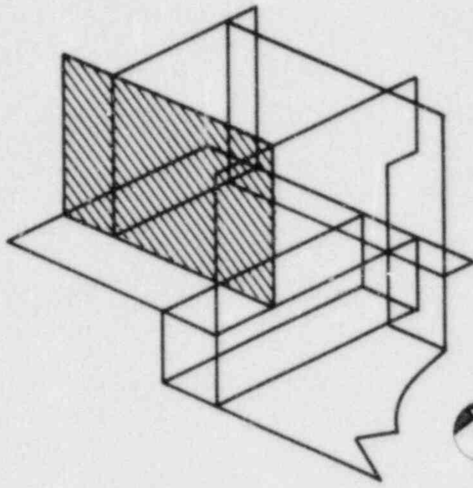
Figure 17
BASEMAT FINITE ELEMENT MODEL
(Sheet 5 of 6)



SOUTH WALL OF FUEL POOL (LOOKING NORT





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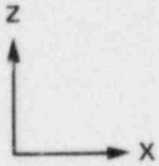


Figure 17
BASEMAT FINITE ELEMENT MODEL
(Sheet 6 of 6)

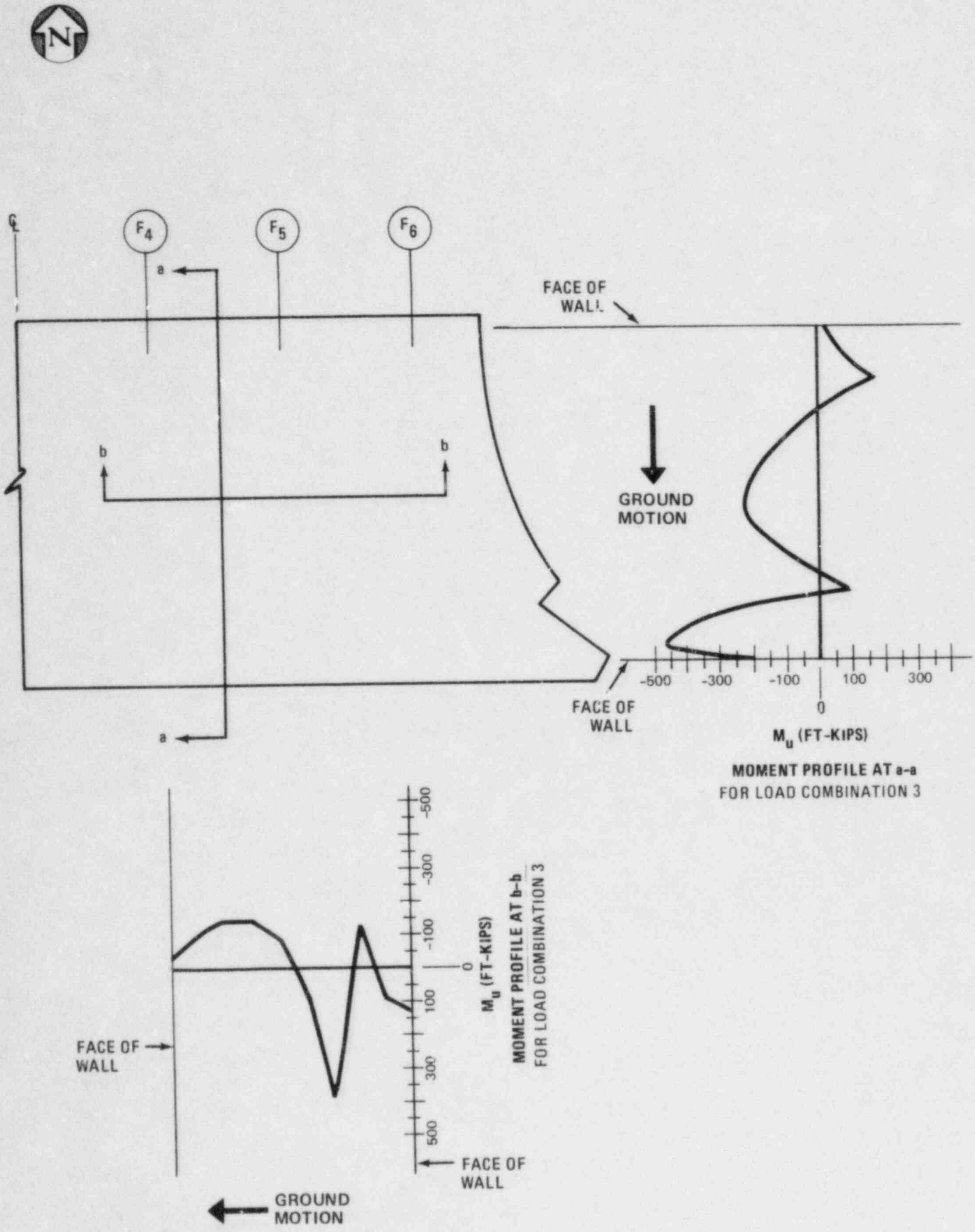


Figure 18
REPRESENTATIVE RESULTS OF BASEMAT ANALYSIS
(Sheet 1 of 2)

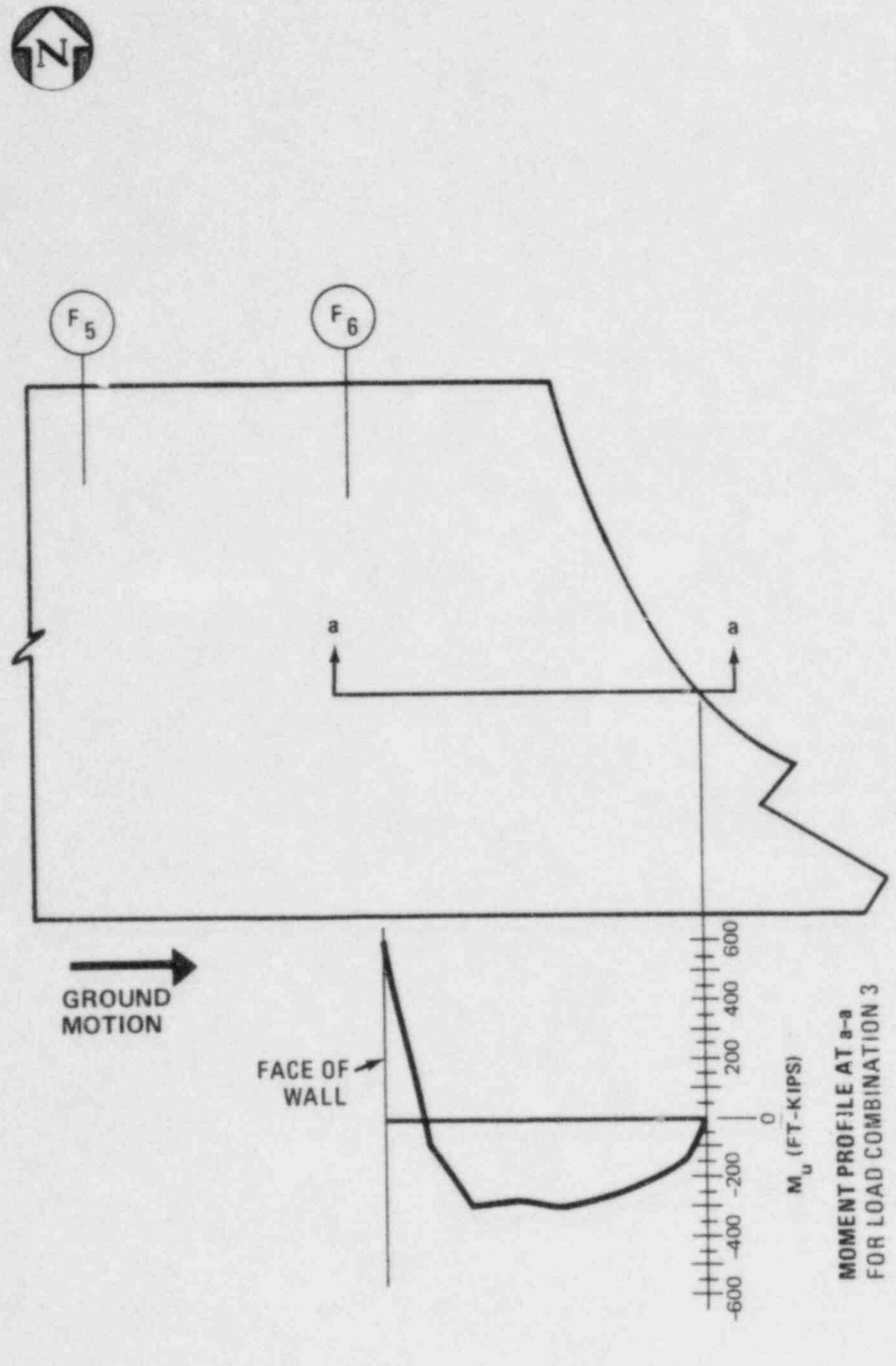
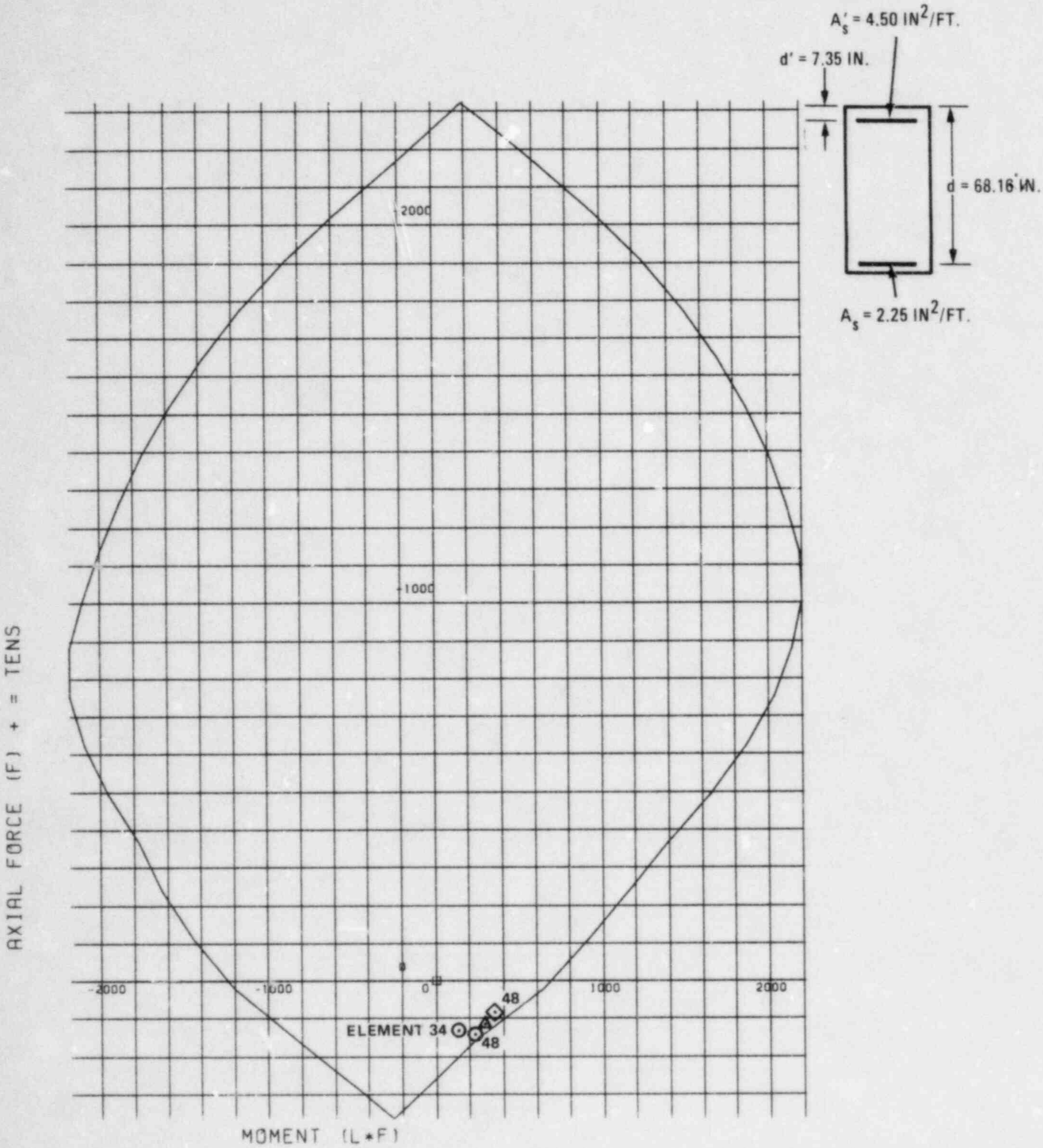


Figure 18
REPRESENTATIVE RESULTS OF BASEMAT ANALYSIS
(Sheet 2 of 2)

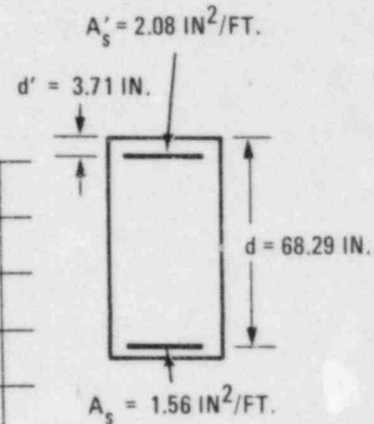


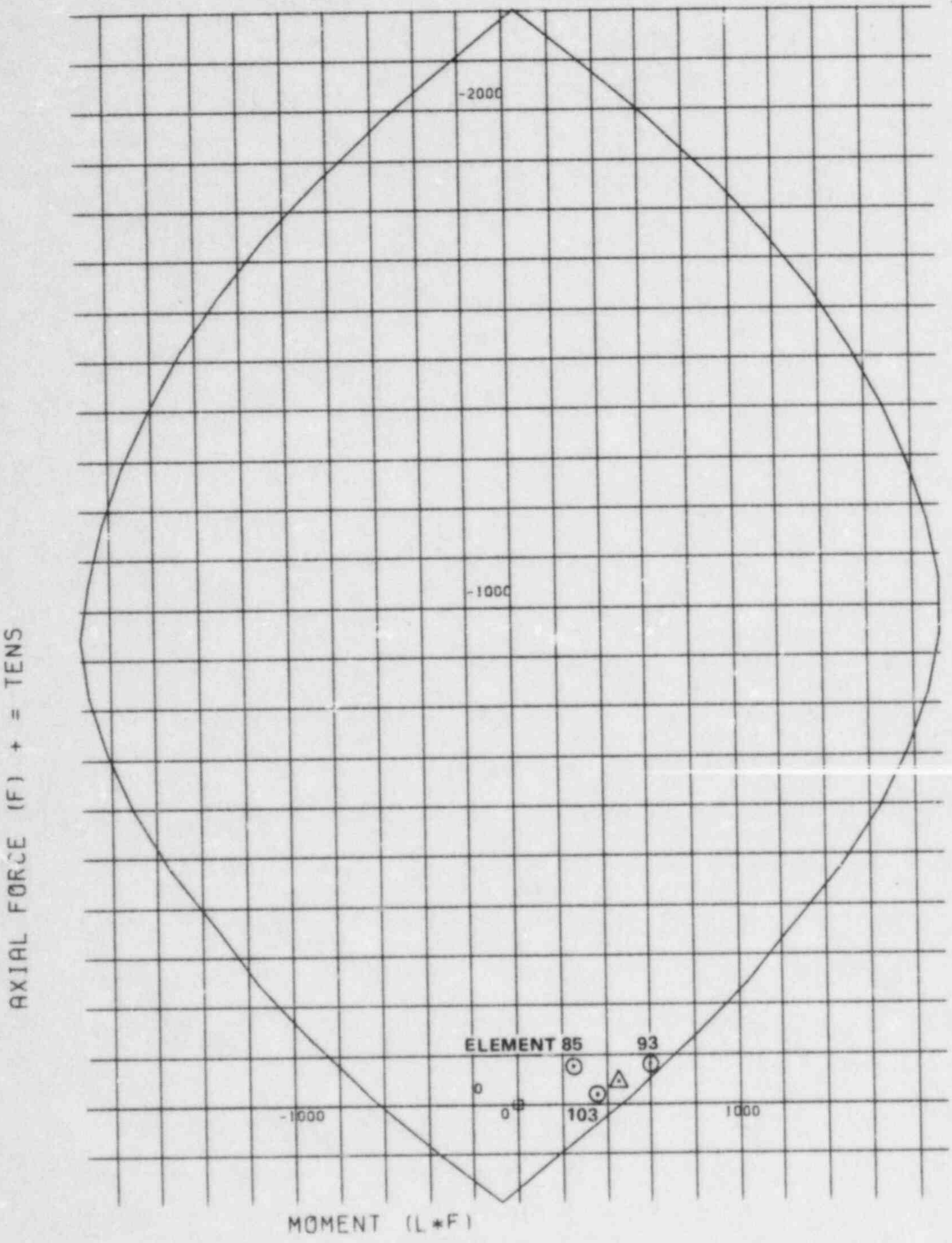
IAD – CENTER SECTION BASEMAT, E-W REINFORCEMENT

LEGEND:

- ⊙ LOAD COMBINATION 3
- ◇ LOAD COMBINATION 10
- △ AVERAGE FOR LOAD COMBINATION 3 ALONG STRIP B-B OF FIGURE 17 SHEET 2 OF 6.

Figure 19
REPRESENTATIVE RESULTS OF BASEMAT DESIGN
(Sheet 1 of 2)





IAD – EAST WING AREA BASEMAT, E-W REINFORCEMENT

LEGEND:

⊙ LOAD COMBINATION 3

△ AVERAGE FOR LOAD COMBINATION 3 ALONG STRIP A-A OF FIGURE 17 SHEET 1 OF 6.

Figure 19
REPRESENTATIVE RESULTS OF BASEMAT DESIGN
(Sheet 2 of 2)

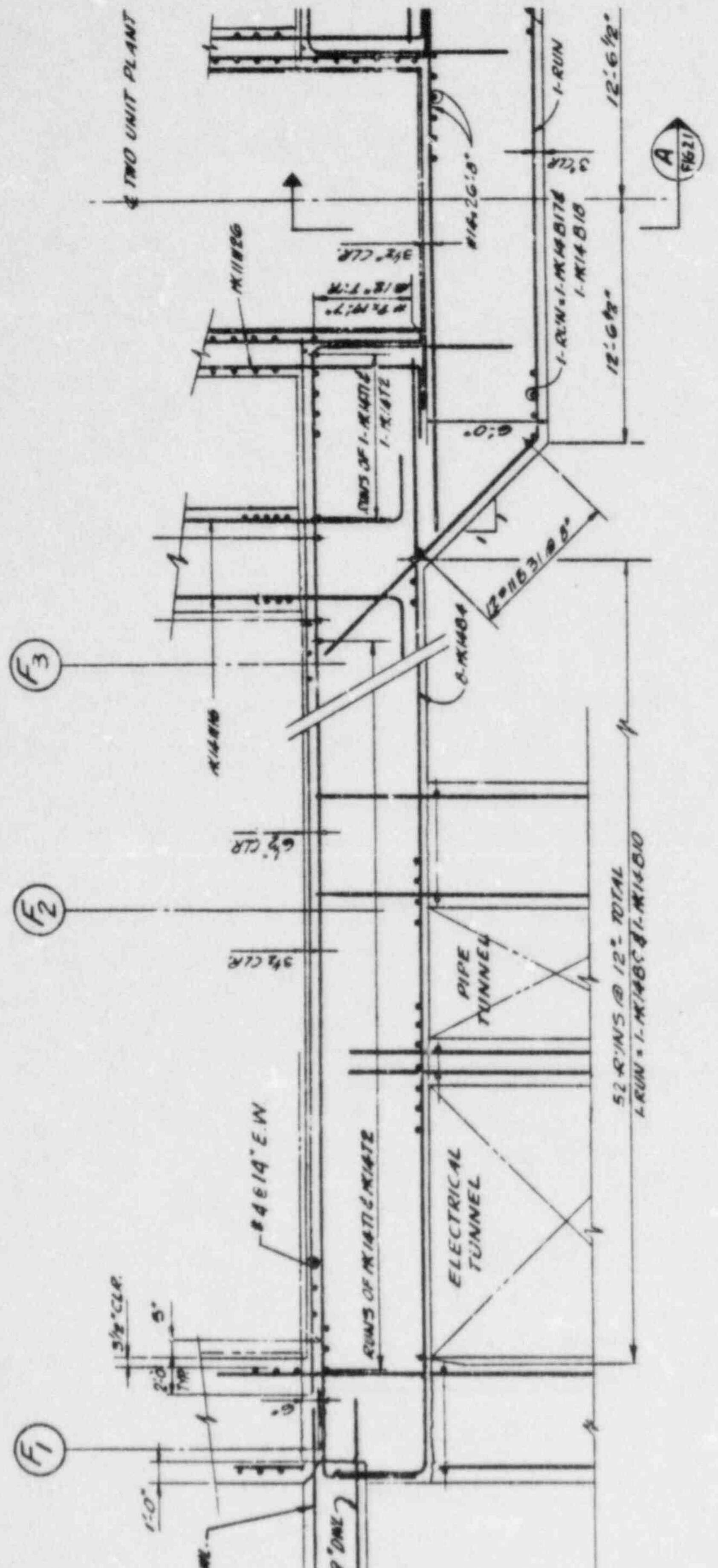
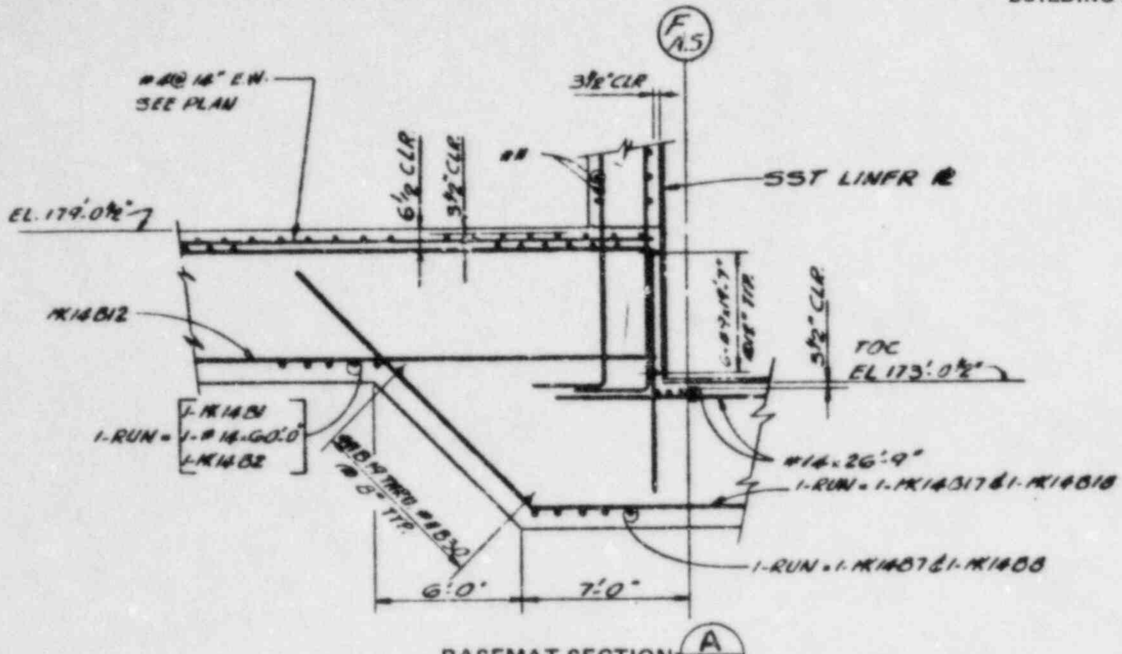
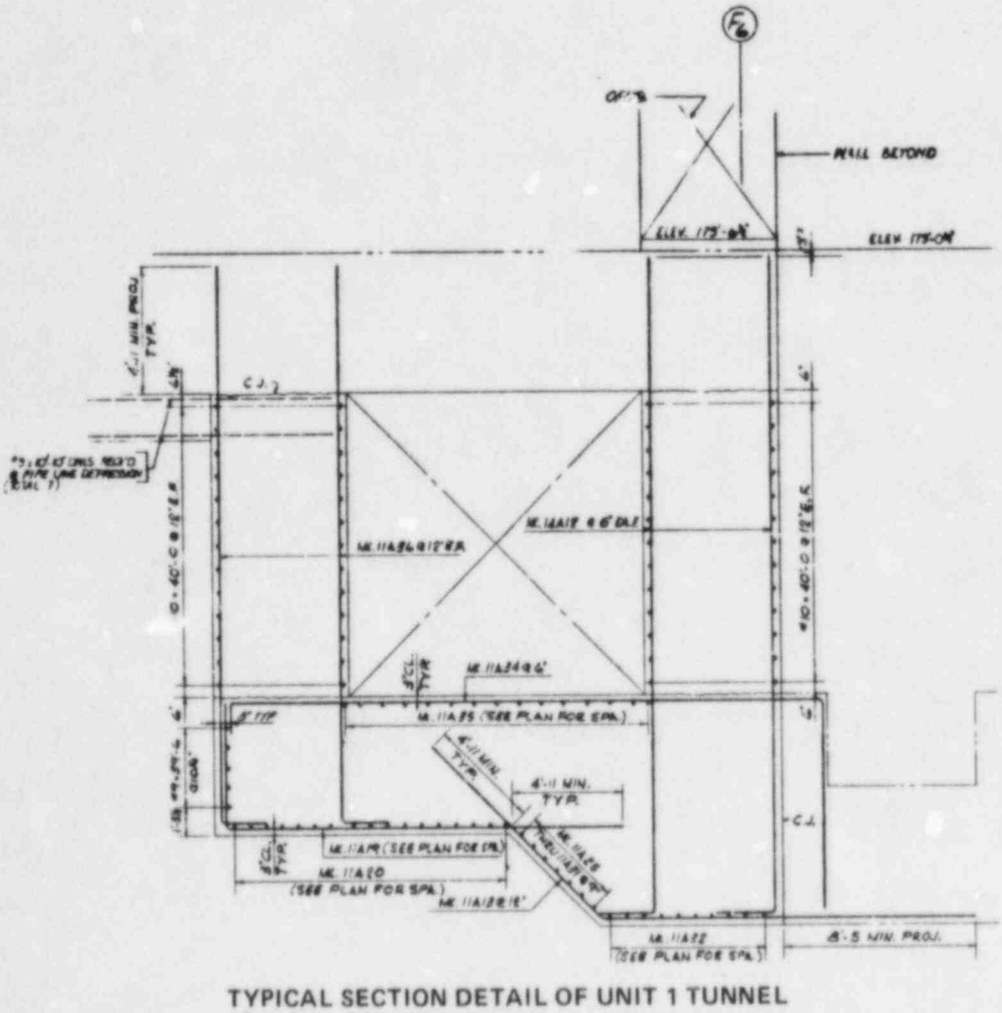


Figure 20
CENTER SECTION BASEMAT REINFORCING



BASEMAT SECTION A/FIG.20



TYPICAL SECTION DETAIL OF UNIT 1 TUNNEL
UNDER CENTER SECTION BASEMAT

Figure 21
BASEMAT SECTION DETAILS

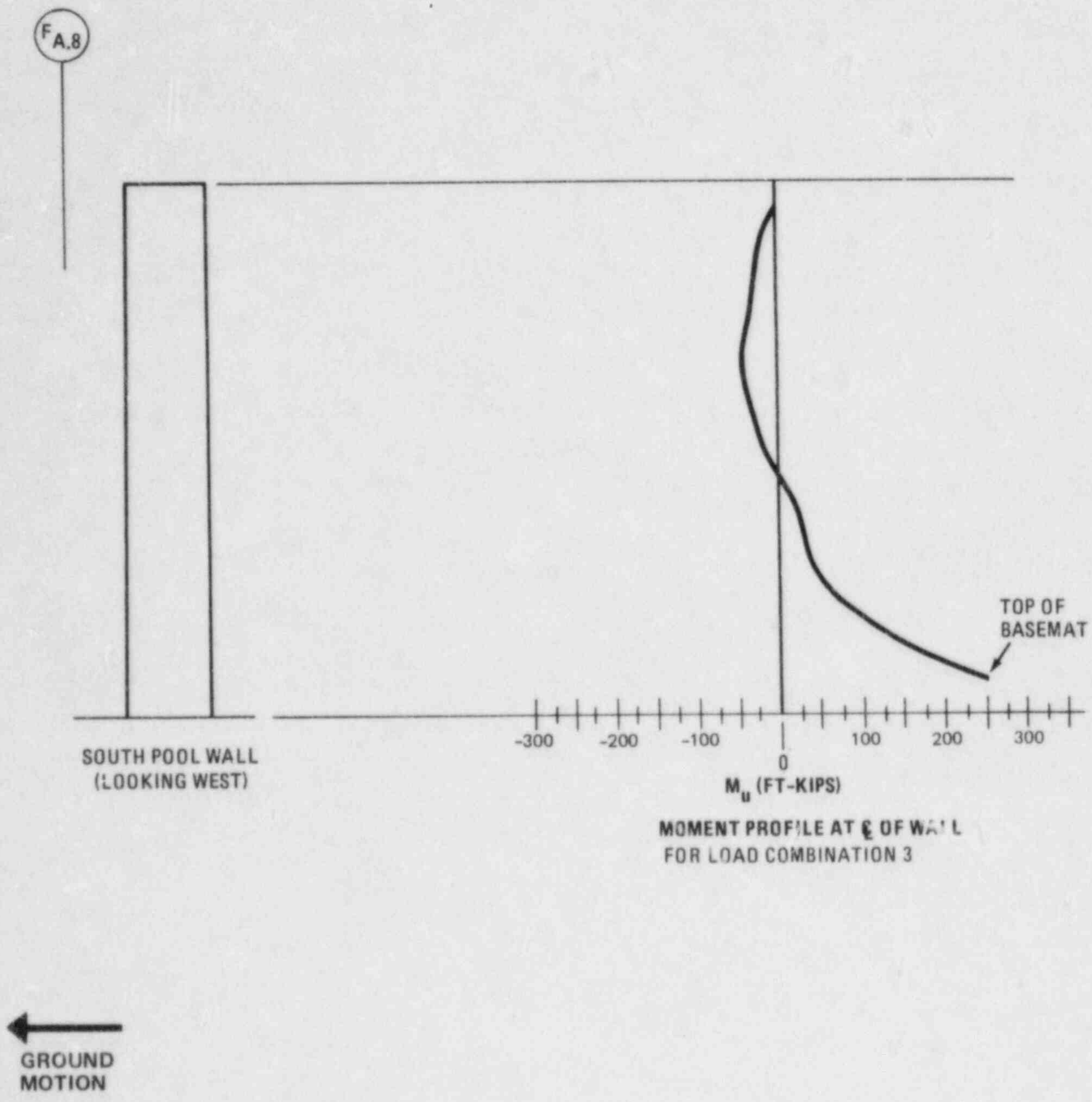
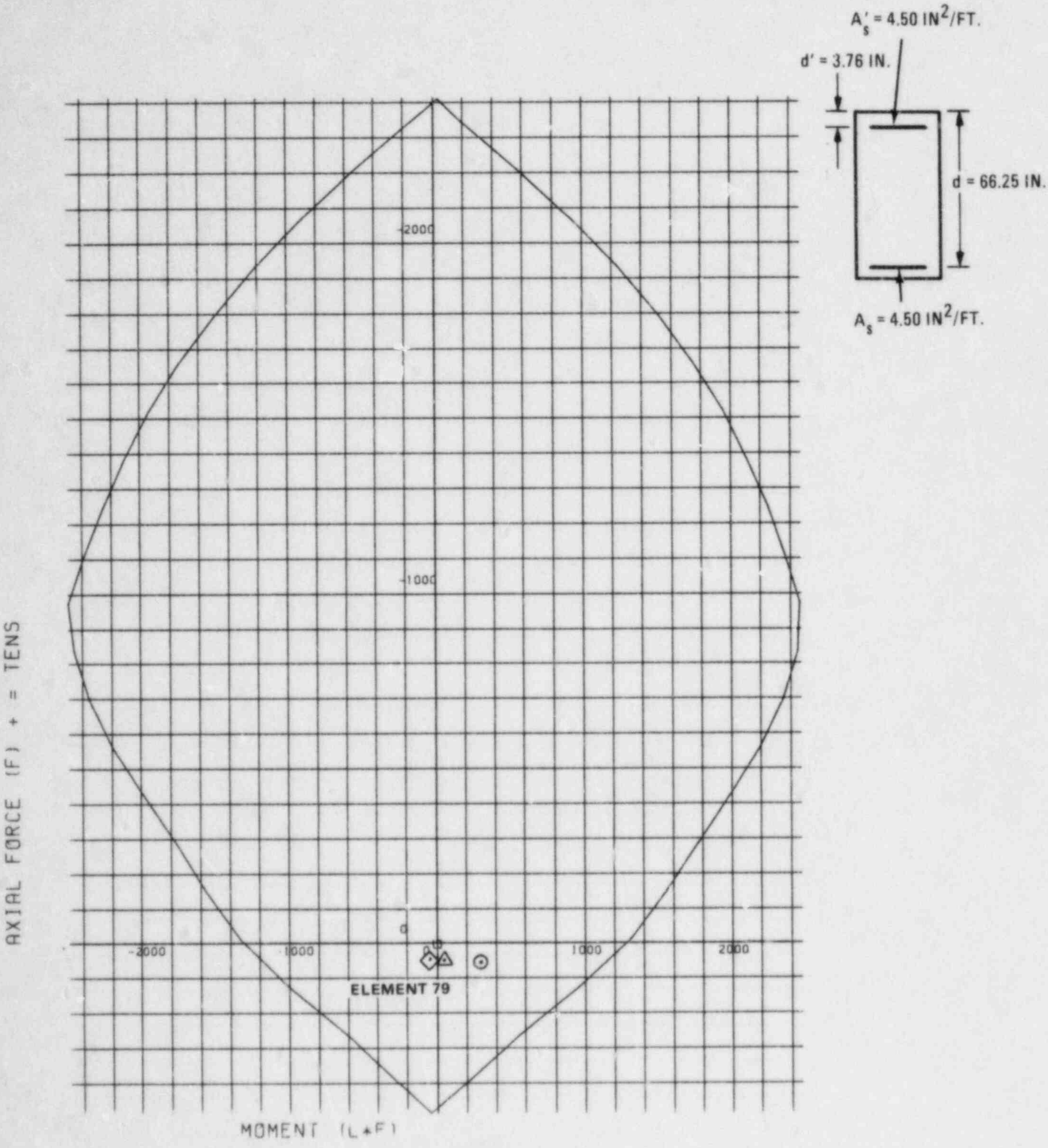


Figure 22
REPRESENTATIVE RESULTS OF SPENT FUEL POOL WALL ANALYSIS

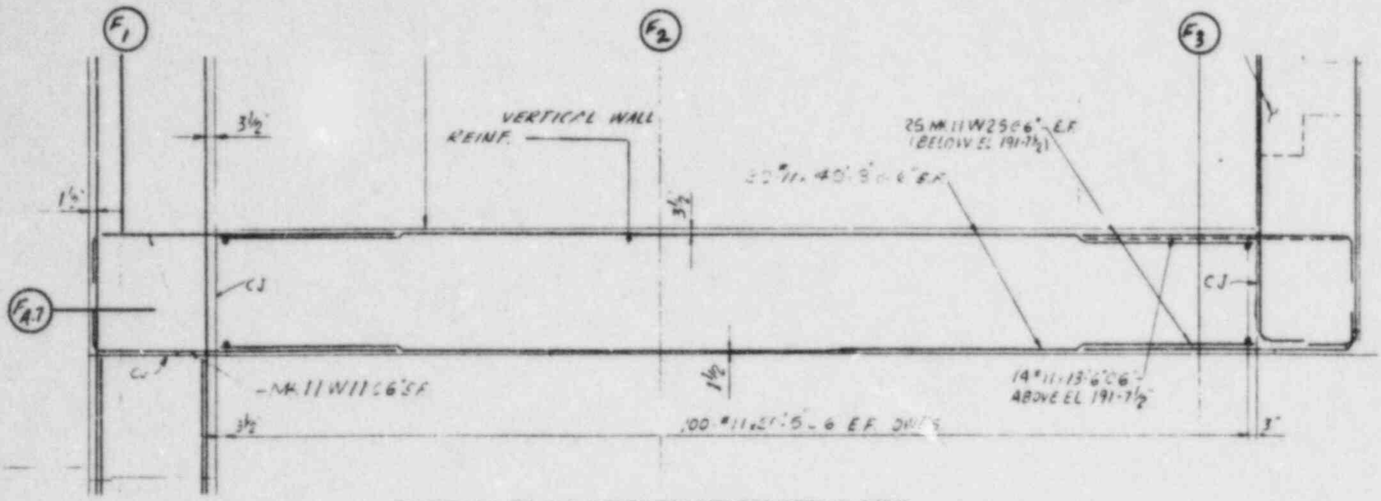


IAD – LEVEL B SOUTH POOL WALL,
VERTICAL REINFORCEMENT

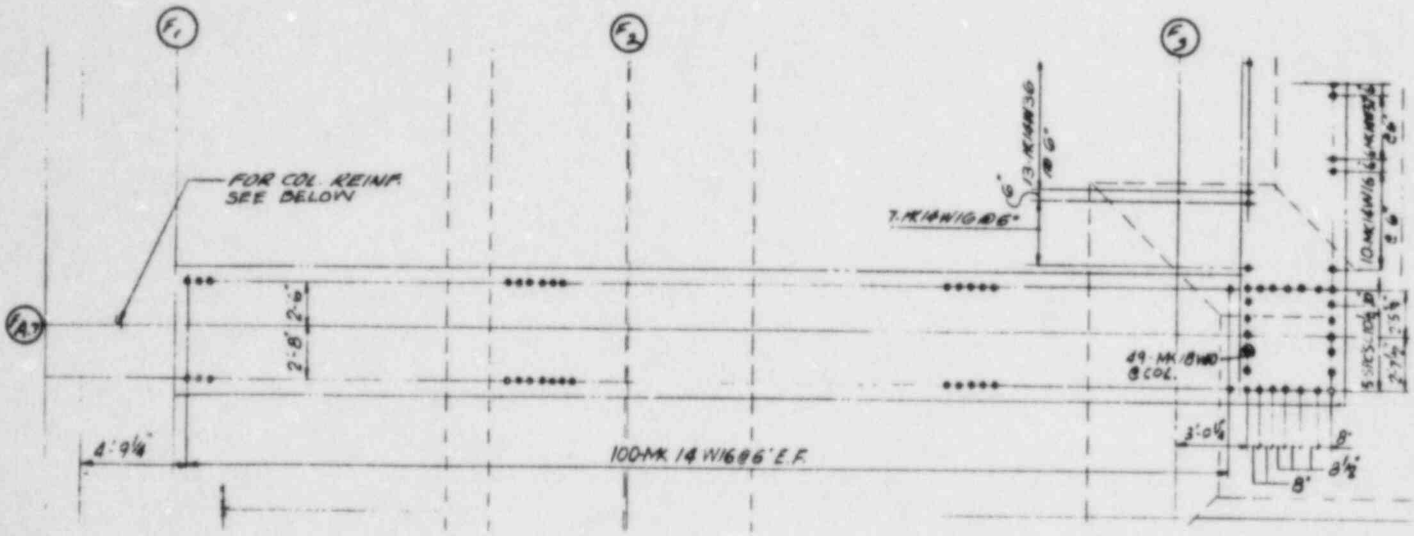
LEGEND:

- ⊙ LOAD COMBINATION 3
- ◇ LOAD COMBINATION 6
- △ LOAD COMBINATION 10

Figure 23
REPRESENTATIVE RESULTS OF SPENT FUEL POOL WALL DESIGN



PARTIAL PLAN AT ELEVATION 179'-0-1/2"



PARTIAL PLAN AT ELEVATION 179'-0-1/2"

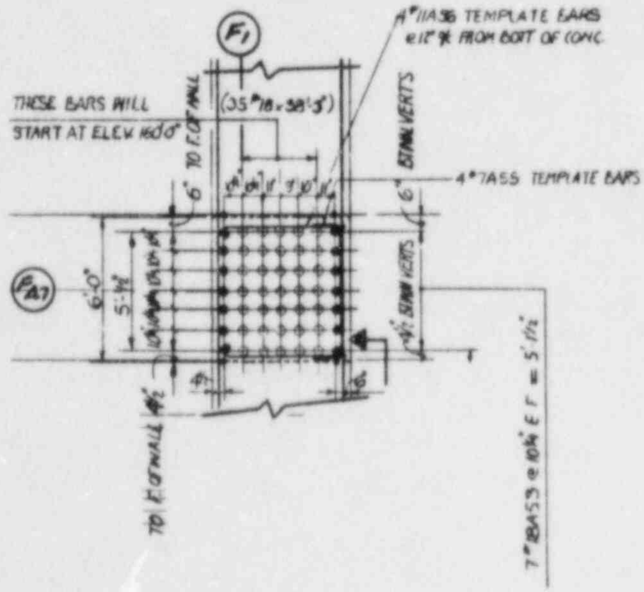


Figure 24
REPRESENTATIVE SPENT FUEL POOL WALLS DETAILS

APPENDIX A

DEFINITION OF LOADS

APPENDIX A

DEFINITION OF LOADS

The loads considered are normal loads, severe environmental loads, extreme environmental loads, abnormal loads, and potential site proximity loads.

A.1 NORMAL LOADS

Normal loads are those loads to be encountered, as specified, during construction stages, during test conditions, and later, during normal plant operation and shutdown. They include the following:

- D — Dead loads or their related internal moments and forces, including hydrostatic loads and any permanent loads except prestressing forces.
- L — Live loads or their related internal moments and forces, including any movable equipment loads and other loads which vary with intensity and occurrence, e.g., lateral soil pressures. Live load intensity varies depending upon the load condition and the type of structural element.
- 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 conditions.

A.2 SEVERE ENVIRONMENTAL LOADS

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

- E Loads generated by the operating basis earthquake (OBE). These include the associated hydrodynamic and dynamic incremental soil pressures.
- W Loads generated by the design wind specified for the plant.

A.3 EXTREME ENVIRONMENTAL LOADS

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

- E' Loads generated by the safe shutdown earthquake (SSE). These include the associated hydrodynamic and dynamic incremental soil pressures.
- W_t Loads generated by the design tornado specified for the plant. They include loads due to wind pressure, differential pressure, and tornado-generated missiles.
- N Loads generated by the probable maximum precipitation.
- B Loads generated by postulated blast along transportation routes.

A.4 ABNORMAL LOADS

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

- P_a Pressure load within or across a compartment and/or building, generated by the postulated break.
- T_a Thermal loads generated by the postulated break and including T_o.

R_a Pipe and equipment reactions under thermal conditions generated by the postulated break and including R_o.

Y_r Load on a structure generated by the reaction of a ruptured high-energy pipe during the postulated event.

Y_j Load on a structure generated by the jet impingement from a ruptured high-energy pipe during the postulated break.

Y_m Load on a structure or pipe restraint resulting from the impact of a ruptured high-energy pipe during the postulated event.

APPENDIX B

LOAD COMBINATIONS

APPENDIX B

LOAD COMBINATIONS

B.1 STEEL STRUCTURES

The steel structures and components are designed in accordance with elastic working stress design methods of Part 1 of the American Institute of Steel Construction (AISC) specification, using the load combinations specified in table B.1.

B.2 CONCRETE STRUCTURES

The concrete structures and components are designed in accordance with the strength design methods of the American Concrete Institute (ACI) Code, ACI 318, using the load combinations specified in table B.2.

TABLE B.1[a]

STEEL DESIGN LOAD COMBINATIONS
ELASTIC METHOD

	EQN	D	L	P_a	T_o	T_a	E	E'	W	W_t	R_o	R_a	Y_j	Y_r	Y_m	N	B	Strength Limit(f_s)
Service Load Conditions																		
	1	1.0	1.0															1.0
	2	1.0	1.0				1.0											1.0
	3	1.0	1.0						1.0									1.0
	4	1.0	1.0		1.0						1.0							1.5
	5	1.0	1.0		1.0		1.0				1.0							1.5
	6	1.0	1.0		1.0				1.0		1.0							1.5
Factored Load																		
	7	1.0	1.0		1.0			1.0			1.0							1.6
(See note b.)	8	1.0	1.0		1.0					1.0	1.0							1.6
	9	1.0	1.0	1.0		1.0						1.0						1.6
(See notes c and d.)	10	1.0	1.0	1.0		1.0	1.0					1.0	1.0	1.0	1.0			1.6
(See notes c and d.)	11	1.0	1.0	1.0		1.0		1.0				1.0	1.0	1.0	1.0			1.7
	12	1.0	1.0		1.0						1.0						1.0	1.6
	13	1.0	1.0		1.0						1.0					1.0		1.6

---

a. See Appendix A for definition of load symbols. f_s is the allowable stress for the elastic design method defined in Part 1 of the AISC, "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings." The one-third increase in allowable stresses permitted for seismic or wind loadings is not considered.

b. When considering tornado missile load, local section strength may be exceeded provided there will be no loss of function of any safety-related system. In such cases, this load combination without the tornado missile load is also to be considered.

c. When considering Y_j, Y_r and Y_m loads, local section strength may be exceeded provided there will be no loss of function of any safety-related system. In such cases, this load combination without Y_j, Y_r, and Y_m is also to be considered.

d. For this load combination, in computing the required section strength, the plastic section modulus of steel shapes, except for those which do not meet the AISC criteria for compact sections, may be used.

TABLE B.2[a][f]

CONCRETE DESIGN LOAD COMBINATIONS
STRENGTH METHOD

	EQN	D	L	P_a	T_o	T_a	E	E'	W	W_t	R_o	R_a	Y_j	Y_r	Y_m	N	B	Strength Limit
Service Load Conditions																		
	1	1.4	1.7															U
(See note b.)	2	1.4	1.7						1.7									U
(See note c.)	3	1.4	1.7				1.9											U
	4	1.05	1.275		1.275						1.275							U
	5	1.05	1.275		1.275				1.275		1.275							U
	6	1.05	1.275		1.275		1.425				1.275							U
Factored Load Conditions																		
	7	1.0	1.0		1.0			1.0			1.0							U
(See note d.)	8	1.0	1.0		1.0					1.0	1.0							U
	9	1.0	1.0	1.5		1.0						1.0						U
(See note e.)	10	1.0	1.0	1.25		1.0	1.25					1.0	1.0	1.0	1.0			U
(See note e.)	11	1.0	1.0	1.0		1.0		1.0				1.0	1.0	1.0	1.0			U
	12	1.0	1.0		1.0						1.0						1.0	U
	13	1.0	1.0		1.0						1.0					1.0		U

---

a. See Appendix A for definition of load symbols. U is the required strength based on strength method per ACI 318-71.
b. Unless this equation is more severe, the load combination 1.2D+1.7W is also to be considered.
c. Unless this equation is more severe, the load combination 1.2D+1.9E is also to be considered.
d. When considering tornado missile load, local section strength may be exceeded provided there will be no loss of function of any safety-related system. In such cases, this load combination without the tornado missile load is also to be considered.
e. When considering Y_j, Y_r, and Y_m loads, local section strength may be exceeded provided there will be no loss of function of any safety-related system. In such cases, this load combination without Y_j, Y_r, and Y_m is also to be considered.
f. Actual load factors used in design may have exceeded those shown in this table.

APPENDIX C

DESIGN OF STRUCTURES FOR TORNADO MISSILE IMPACT

APPENDIX C

DESIGN OF STRUCTURES FOR TORNADO MISSILE IMPACT

C.1 INTRODUCTION

This appendix contains methods and procedures for analysis and design of steel and reinforced concrete structures and structural elements subject to tornado-generated missile impact effects. Postulated missiles, and other concurrent loading conditions are identified in Section 3.2 of the Design Report.

Missile impact effects are assessed in terms of local damage and structural response. Local damage (damage that occurs in the immediate vicinity of the impact area) is assessed in terms of perforation and scabbing.

Evaluation of local effects is essential to ensure that protected items would not be damaged directly by a missile perforating a protective barrier or by scab particles. Empirical formulas are used to assess local damage.

Evaluation of structural response is essential to ensure that protected items are not damaged or functionally impaired by deformation or collapse of the impacted structure.

Structural response is assessed in terms of deformation limits, strain energy capacity, structural integrity, and structural stability. Structural dynamics principles are used to predict structural response.

C.1.1 Procedures

The general procedures for analysis and design of structures or structural elements for missile impact effects include:

a. Defining the missile properties (such as type, material, deformation characteristics, geometry, mass, trajectory, strike orientation, and velocity).

b. Determining impact location, material strength, and thickness required to preclude local failure (such as perforation for steel targets and scabbing for reinforced concrete targets).

c. Defining the structure and its properties (such as geometry, section strength, deformation limits, strain energy absorption capacity, stability characteristics, and dynamic response characteristics).

d. Determining structural response considering other concurrent loading conditions.

e. Checking adequacy of structural design (stability, integrity, deformation limits, etc.) to verify that local damage and structural response (maximum deformation) will not impair the function of safety-related items.

C.2 LOCAL EFFECTS

Evaluation of local effects consists of estimating the extent of local damage and characterization of the interface force-time function used to predict structural response. Local damage is confined to the immediate vicinity of the impact location on the struck element and consists of missile deformation, penetration of the missile into the element, possible perforation of the element, and, in the case of reinforced concrete, dislodging of concrete particles from the back face of the element (scabbing).

Because of the complex physical processes associated with missile impact, local effects are evaluated primarily by application of empirical relationships based on missile impact test results. Unless otherwise noted, these formulas are applied considering a normal incidence of strike with the long axis of the missile parallel to the line of flight.

C.2.1 Reinforced Concrete Elements

The parts of the building structure that offer protection for safety-related equipment against tornado-generated missiles are provided with $f_c' = 4000$ psi minimum concrete strength, have 24-inch-minimum-thick walls, and have 21-inch-minimum-thick roofs. Therefore, the walls and roofs of these structures are resistant to perforation and scabbing by the postulated missiles discussed in Section 3.2 of the Design Report under tornado loads.

C.2.2 Steel Elements

Steel barriers subjected to missile impact are designed to preclude perforation. An estimate of the steel element thickness for threshold of perforation for nondeformable missiles is provided by equation 2-1, which is a more convenient form of the Ballistic Research Laboratory (BRL) equation for perforation of steel plates with material constant taken as unity (reference 1).

$$T_p = \frac{(E_k)^{2/3}}{672D} \qquad E_k = \frac{M_m V_s^2}{2} \tag{2-1}$$

where:

T_p = steel plate thickness for threshold of perforation (in.).

E_k = missile kinetic energy (ft-lb).

M_m = mass of the missile ($lb\text{-}s^2/ft$).

V_s = missile striking velocity (ft/s).

D = missile diameter (in.).[a]

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a. For irregularly shaped missiles, an equivalent diameter is used. The equivalent diameter is taken as the diameter of a circle with an area equal to the circumscribed contact, or projected frontal area, of the noncylindrical missile. For pipe missiles, D is the outside diameter of the pipe.

The design thickness to prevent perforation, t_p, must be greater than the predicted threshold value. The threshold value is increased by 25 percent to obtain the design thickness.

$$t_p = 1.25\ T_p \tag{2-2}$$

where:

t_p = design thickness to preclude perforation (in.).

C.3 STRUCTURAL RESPONSE DUE TO MISSILE IMPACT LOADING

When a missile strikes a structure, large forces develop at the missile-structure interface, which decelerate the missile and accelerate the structure. The response of the structure depends on the dynamic properties of the structure and the time-dependent nature of the applied loading (interface force-time function). The force-time function is, in turn, dependent on the type of impact (elastic or plastic) and the nature and extent of local damage.

C.3.1 <u>General</u>

In an elastic impact, the missile and the structure deform elastically, remain in contact for a short period of time (duration of impact), and subsequently disengage due to the action of elastic interface restoring forces.

In a plastic impact, the missile or the structure or both may deform plastically or sustain permanent deformation or damage (local damage). Elastic restoring forces are small, and the missile and the structure tend to remain in contact after impact. Plastic impact is much more common in nuclear plant design than elastic impact, which is rarely encountered. For example, test data indicate that the impact from all postulated tornado-generated missiles can be characterized as a plastic collision.

If the interface forcing function can be defined or conservatively idealized (from empirical relationships or from theoretical considerations), the structure can be modeled mathematically, and conventional analytical or numerical techniques can be used to predict structural response. If the interface forcing function cannot be defined, the same mathematical model of the structure can be used to determine structural response by application of conservation of momentum and energy balance techniques with due consideration for type of impact (elastic or plastic).

In either case, in lieu of a more rigorous analysis, a conservative estimate of structural response can be obtained by first determining the response of the impacted structural element and then applying its reaction forces to the supporting structure. The predicted structural response enables assessment of structural design adequacy in terms of strain energy capacity, deformation limits, stability, and structural integrity.

Three different procedures are given for determining structural response: the force-time solution, the response chart solution, and the energy balance solution. The force-time solution involves numerical integration of the equation(s) of motion and is the most general method applicable for any pulse shape and resistance function. The response chart solution can be used with comparable results, provided the idealized pulse shape (interface forcing function) and the resistance function are compatible with the response chart. The energy balance solution is used in cases where the interface forcing function cannot be defined or where an upper limit check on structural response is desired. This method will consistently overestimate structural response, since the resisting spring forces during impact are neglected.

In defining the mass-spring model, consideration is given to local damage that could affect the response of the element. For concrete slab elements, the beneficial effect of formation of a fracture plane which propagates from the impact zone to the back of the slab (back face fracture plane) just prior to scabbing (reference 2) is neglected. The formation of this fracture plane limits the forces transferred to the surrounding slab and significantly reduces overall structural response. Since scabbing is to be precluded in the design, the structural response check is made assuming the fracture plane is not formed. It is recognized, however, that should the missile velocity exceed that for threshold of scabbing, structural response would be limited by this mechanism.

Therefore, the structural response is conservatively evaluated ignoring formation of the fracture plane and any reduction in response.

C.3.2 Structural Assessment

The predicted structural response enables assessment of design adequacy in terms of strain energy capacity, deformation limits, stability, and structural integrity.

For structures allowed to displace beyond yield (elasto-plastic response), a check is made to ensure that deformation limits would not be exceeded, by comparing calculated displacements or required ductility ratios with allowable values (such as those contained in table C-1).

C.4 REFERENCES

1. Gwaltney, R. C., "Missile Generation and Protection in Light-Water-Cooled Power Reactor Plants," ORNL NSIC-22, Oak Ridge National Laboratory, Oak Ridge, Tennessee, for the USAEC, September 1968.

2. Rotz, J. V., "Results of Missile Impact Tests on Reinforced Concrete Panels," Vol 1A, pp 720-738, Second Specialty Conference on Structural Design of Nuclear Power Plant Facilities, New Orleans, Louisiana, December 1975.

TABLE C-1

DUCTILITY RATIOS (Sheet 1 of 2)

Member Type and Load Condition	Maximum Allowable Value of Ductility Ratio (μ)
Reinforced Concrete	
Flexure(1):	
Beams and one-way slabs(2)	$\frac{0.10}{p-p'} \leq 10$
Slabs with two-way reinforcing(2)	$\frac{0.10}{p-p'} \leq 10$ or 30 (See 3 and 4)
Axial compression(1):	
Walls and columns	1.3
Shear, concrete beams and slabs in region controlled by shear:	
Shear carried by concrete only	1.3
Shear carried by concrete and stirrups	1.6
Shear carried completely by stirrups	2.0
Shear carried by bent-up bars	3.0
Structural Steel	
Columns(5) $\ell/r \leq 20$	1.3
$\ell/r > 20$	1.0
Tension due to flexure	10
Shear	10
Axial tension and steel plates in membrane tension(6)	$0.5 \frac{e_u}{e_y}$
Compression members not required for stability of building structures	10

TABLE C-1

DUCTILITY RATIOS (Sheet 2 of 2)

Notes:

(1) The interaction diagram used to determine the allowable ductility ratio for elements subject to combined flexure and axial compression is provided in figure C-1.

(2) p and p' are the positive and negative reinforcing steel ratios, respectively.

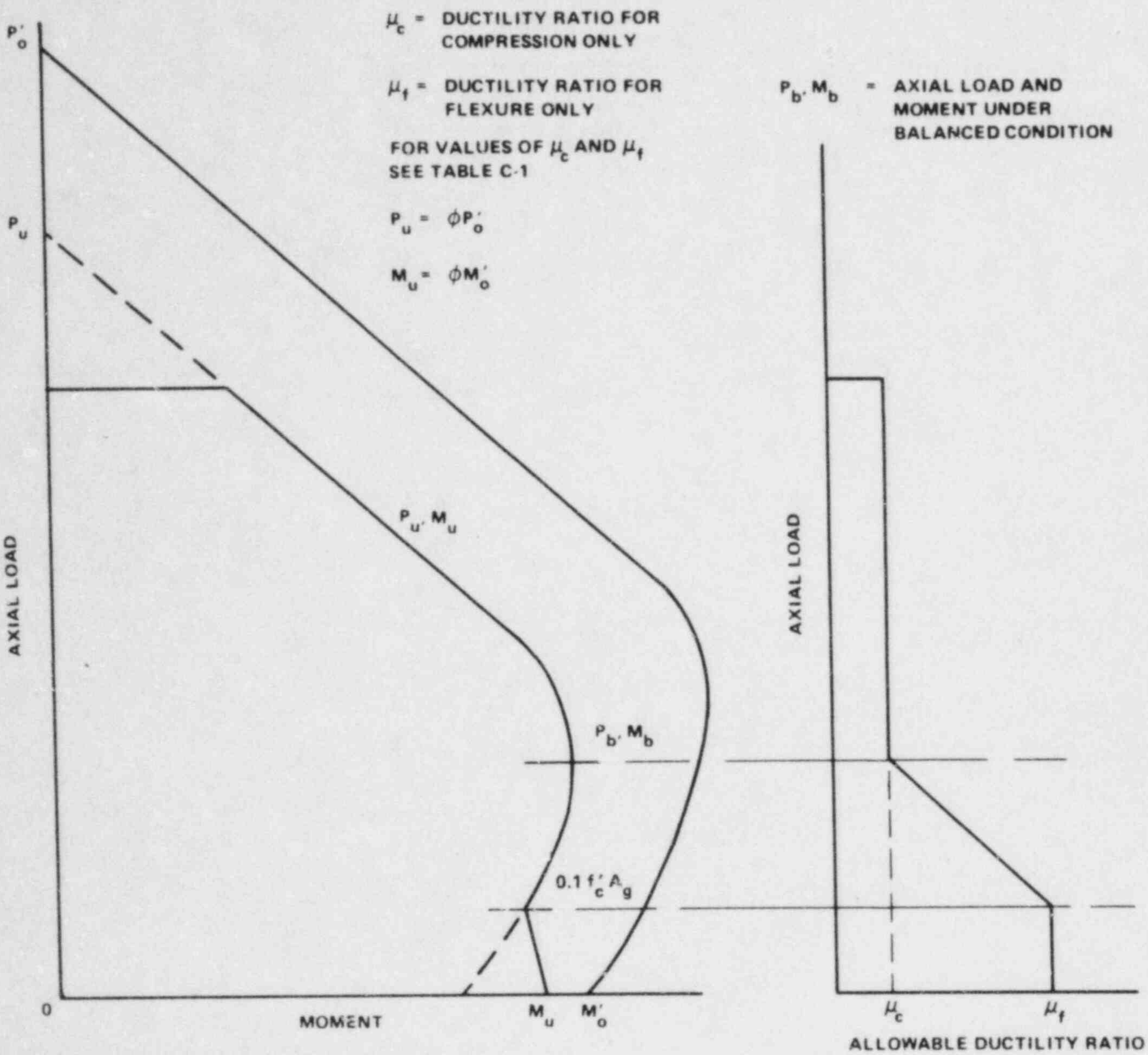
(3) Ductility ratio up to 10 can be used without an angular rotation check.

(4) Ductility ratio up to 30 can be used provided an angular rotation check is made.

(5) ℓ/r is the member slenderness ratio. The value specified is for axial compression. For columns and beams with uniform moment the following value is used:

$$\frac{14 \times 10^4}{F_y \left(\frac{k\ell}{r}\right)^2} + \frac{1}{2} \leq 10$$

(6) e_u and e_y are the ultimate and yield strains. e_u shall be taken as the ASTM-specified minimum.



(A) REINFORCED CONCRETE INTERACTION DIAGRAM (P VS M)

(B) ALLOWABLE DUCTILITY RATIO μ VS P

Figure C–1
MAXIMUM ALLOWABLE DUCTILITY RATIO
FOR REINFORCED CONCRETE SECTION
WITH BEAM–COLUMN ACTION