VOGTLE ELECTRIC GENERATING PLANT

GEORGIA POWER COMPANY

# CATEGORY 1 TANKS

# DESIGN REPORT

Prepared

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VEGP-CATEGORY 1 TANKS DESIGN REPORT

## 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 Category 1 tanks, 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, and a design summary of representative key structural elements, including the governing design forces.

## 2.0 DESCRIPTION OF STRUCTURES

### 2.1 GENERAL DESCRIPTION

In addition to the NSCW systems, there are three other safety-related systems that require the storage of large volumes of water. These are the condensate, refueling, and reactor makeup water systems. The large storage volumes are retained in independent tank structures. There are four tank structures per unit, i.e., one for reactor makeup water storage, one for refueling water storage, and two for condensate water storage. These tanks are constructed of reinforced concrete and are lined with stainless steel plate.

#### 2.1.1 Reactor Makeup Water Storage Tank (RMWST)

The RMWST supplies water for makeup to the spent fuel pool, the component cooling water and auxiliary component cooling water surge tanks, and the engineered safety feature (ESF) chiller expansion tanks. It provides water to the boric acid blender for daily usage as a diluter to the reactor coolant system. It also serves as a source of demineralized water for the flushing and cleaning of various evaporators, gas strippers, pumps, tanks, and pipelines.

#### 2.1.2 Refueling Water Storage Tank (RWST)

The main function of the RWST is to provide water to flood the containment refueling canal during refueling operations. It also provides borated water to the safety injection, residual heat removal, and chemical and volume control systems as well as to the containment spray system under loss-of-coolant accident (LOCA) and main steam line break conditions.

### 2.1.3 Condensate Water Storage Tanks (CSTs)

The CSTs provide makeup and surge capacity to the turbine plant and system inventories as well as auxiliary feedwater supply for emergency shutdown decay heat removal upon a postulated failure of the normal feedwater system.

## 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".

Due to the large storage volumes required, it is not practical to place these inventories in tanks located within any of the major Category 1 structures. They are instead placed in reinforced concrete tank structures located in the Category 1 yard area. Location drawings for Units 1 and 2 are shown in figures 1 and 2.

The Category 1 tanks are founded on Category 1 backfill placed on the marl bearing stratum. The Category 1 tanks are located at grade and supported on mat foundations. The foundation mats are square except for the two CSTs which are supported on a common rectangular mat. The top of each mat is located at elevation 220'-0". The foundation dimensions are described in section 2.3.

## 2.3 GEOMETRY AND DIMENSIONS

Each of the Category 1 tanks consists of a cylindrical shell supported on a mat foundation with a matching and continuous circular roof. Missile protection structures are provided which enclose and support the connecting piping as it runs from each tank to its associated safety-related tunnel. The roofs are sloped from the center to the edges and have a maximum thickness of 24 inches at the center and a minimum of 21 inches at the edges. Schematic plans and elevations for each tank are provided in figures 3 through 8. A summary of the major dimensions is as follows.

| Tank Structure | Normal Capacity | Inside Diameter | Cylinder Wall Thickness | Height to Top of Roof | Foundation Dimensions and Thickness |
|---|---|---|---|---|---|
| RMWST | 165,000 gal. | 33' | 2' | 42'-1" | 51'x51'x3' |
| RWST | 715,000 gal. | 48' | 3' | 62'-1" | 62'x62'x4' |
| CST | 480,000 gal. (per tank) | 44' | 2' | 56'-1" | 63'x115'x4' |

## 2.4 KEY STRUCTURAL ELEMENTS

The key structural elements of each tank are the basemat, cylindrical shell, roof, and missile protection structure for external piping. The basemats for the RMWST and CSTs incorporate recessed moats around their perimeters. The roofs consist of a system of precast beams and slab panel sections which are unified with a cast-in-place slab that is continuous with the walls. The missile structures are supported on the same basemats as the tanks, but are separated from the tanks by a gap (minimum of 3 inches) in order to maintain seismic independence and avoid stress concentrations in the cylindrical shell.

## 2.5 MAJOR EQUIPMENT

There is no major equipment located in or on any of the tank structures. The RMWST and CSTs have associated nonsafety-related vacuum degasifier systems located on adjacent pads.

## 2.6 SPECIAL FEATURES

### 2.6.1 Liner Plate

The liner plate is used as the inside form for the placement of the tank cylinder wall. It is hydrotested for leak-tight integrity prior to wall placement. After wall placement, it is supported by the wall and has no structural function. The liner is stiffened by vertical angles added to increase its bending resistance to the weight of the wet concrete. The stiffeners also serve to anchor the liner system into the tank wall after the concrete has been placed. The liner plate is nominally 1/4 inch thick although some of the lower courses are 5/16 inch thick. Wherever it is required that loads from penetrations or inner attachments be transmitted through the liner, the liner is locally thickened, and welded studs are added to insure transfer of the loads directly to the reinforced concrete. The floors of the tanks are lined by welding plate sections to embedded strips cast into the top of the basemat. There is no liner on the underside of the roof.

### 2.6.2 Dikes

Dikes have been provided to allow retention of at least 5 percent of each tank's capacity as a conservative measure. This is accomplished by providing moats around the RMWST and CSTs and a surrounding retaining wall for the RWST. The moats are recessed into the basemats, which have been thickened to accommodate them. The moat concept is impractical for the RWST due to its size.

### 2.6.3 Tank Wall Penetrations

Piping penetrations are embedded in the reinforced concrete walls and are considered anchor points for the connecting piping. Direct load transfer is assured by provision of shear plates on each penetration sleeve.

# 3.0 DESIGN BASES

## 3.1 CRITERIA

The following documents are applicable to the design of the Category 1 tanks.

### 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

The basic loads applicable for consideration in design of the tanks are individually discussed below. A summary of load term definitions is provided in Appendix A.

### 3.2.1 Normal Loads

#### 3.2.1.1 Dead Loads (D)

| | |
|---|---|
| • Reinforced concrete | 150 pcf |
| • Storage water | 62.4 pcf |
| • Steel liner | 490 pcf |
| • Piping applied to slabs and walls where applicable | 50 psf |

#### 3.2.1.2 Live Loads (L)

| | |
|---|---|
| • Concentrated load on slabs (applied to maximize moment and shear) | 5 k |
| • Distributed snow or other load on roofs | 30 psf |
| • Distributed load on platforms and interior slabs | 100 psf |
| • At rest soil pressure | 0.7 $\gamma_m H$ (refer to section 3.4.6) |

#### 3.2.1.3 Operating Thermal Loads ($T_o$)

The tanks are vented to the atmosphere and are not heated. The only normal temperature differential that will be experienced by the structural elements is that due to the time lag in the equilibration of the inside tank temperature to the outside ambient temperature during daily variations. This differential is minimal and, therefore, not considered.

#### 3.2.1.4 Pipe Reactions ($R_o$)

The local effect of pipe reactions on the tank walls are investigated where the wall penetrations are used as anchor points for large lines.

### 3.2.2 Severe Environmental Loads

#### 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 and selected levels are discussed in the Seismic Analysis Report. The horizontal and vertical in-structure OBE accelerations are shown in tables 1 through 3.

The OBE damping values, as percentages of critical damping applicable to the Category 1 tanks, are as follows:

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

Dynamic lateral earth pressures are developed by applying the Mononabe-Okabe method for active earth pressure above the water table using peak ground accelerations. The dynamic incremental soil pressure profile is shown in figure 9.

Hydrodynamic fluid forces are developed by applying reference 1. The OBE hydrodynamic fluid pressure profiles are shown in figure 10.

#### 3.2.2.2 Design Wind (W)

Applicable wind load is the 100-year mean recurrence interval 110 mph wind per American National Standards Institute (ANSI) A58.1-1972 (reference 2). Coefficients are per Exposure C, applicable for flat open country. The basic wind effective velocity pressure profile is shown in figure 11.

### 3.2.3 Extreme Environmental Loads

#### 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 and selected levels are discussed in the Seismic Analysis Report. The horizontal and vertical in-structure SSE accelerations are shown in tables 1 through 3.

The SSE damping values, as percentages of critical damping, applicable to the Category 1 tanks are as follows.

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

Dynamic lateral earth pressures are developed by applying the Mononabe-Okabe method for active earth pressure above the water table using the peak free field accelerations. The dynamic incremental soil pressure profile is shown in figure 9. Hydrodynamic fluid forces are developed by applying reference 3.

3.2.3.2 Tornado ($W_t$)

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<br>5 mph minimum |
| Maximum wind speed | 360 mph |
| Radius of tornado at maximum rotational speed | 150 ft |
| Atmospheric pressure differential | -3 psi |
| Rate of pressure differential change | 2 psi/sec |

The tornado effective velocity pressure profile used in the design (see figure 11) 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. No reduction in pressure is made for the shielding effects that may be provided by adjacent structures.

The Category 1 tanks and missile structures are also designed to withstand tornado missile impact effects from airborne objects transported by the tornado. The tornado missile parameters are listed in table 4. Missile trajectories up to and including 45 degrees off of 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, PMP (N)

PMP loads are not applicable to the Category 1 tanks as there are no parapets and the roofs are substantially sloped. Special roof scuppers are provided on the CST missile structure 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 inwards or outwards) applied as a static load.

### 3.2.4 Abnormal Loads

#### 3.2.4.1 Thermal Loads Under Accident Conditions ($T_a$)

A thermal load generated by a plant system failure is considered for the RMWST only. This load is postulated to be a one time only load. The peak internal tank temperature is predicted to be 150°F. The walls are analyzed for a maximum differential temperature of 133°F by conservatively assuming the peak internal temperature occurs simultaneous with the minimum postulated ambient temperature of 17°F.

There are no other significant abnormal loads applicable to the Category 1 tanks.

## 3.3 LOAD COMBINATIONS AND STRESS/STRENGTH LIMITS

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

## 3.4 MATERIALS

The following materials and material properties were used in the design of the Category 1 tanks.

### 3.4.1 Concrete

- Compressive strength — $f_c' = 4.0$ ksi
- Modulus of elasticity — $E_c = 3,834$ ksi
- Shear modulus — $G = 1440$ ksi
- Poissons ratio — $\nu = 0.17 - 0.25$

### 3.4.2 Reinforcement

#### 3.4.2.1 American Society for Testing Materials (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

3.4.2.2 ASTM A685 Welded Wire Fabric

| | |
|---|---|
| • Minimum yield stress | $F_y$ = 56 ksi |
| • Minimum tensile strength | $F_{to}$ = 70 ksi |

### 3.4.3 Structural Steel - ASTM A36

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

### 3.4.4 Structural Bolts - ASTM A325

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

### 3.4.5 Liner Plate and Nozzles

| | |
|---|---|
| • Liner plate | ASTM A240 Type 304L |
| • Concrete embeds | ASTM A36 |
| • Pipe penetrations | ASME SA-312, Type 304L |

### 3.4.6 Foundation Media

3.4.6.1 General Description

See section 2.2.

3.4.6.2 Category 1 Backfill

| | |
|---|---|
| • Moist unit weight | $\gamma_m$ = 126 pcf |
| • Saturated unit weight | $\gamma_t$ = 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 $C = 0$

3.4.6.3 Modulus of Subgrade Reaction

| | | |
|---|---|---|
| • Static | RWST | 25 kcf |
| | RMWST | 40 kcf |
| | CST | 20 kcf |
| • Dynamic | RWST | 75 kcf |
| | RMWST | 120 kcf |
| | CST | 60 kcf |

3.4.6.4 Net Bearing Capacities

| | | |
|---|---|---|
| • Ultimate | RWST | 88.9 ksf |
| | RMWST | 95.7 ksf |
| | CST | 115.3 ksf |
| • Allowable static | RWST | 29.6 ksf |
| | RMWST | 31.9 ksf |
| | CST | 38.4 ksf |
| • Allowable dynamic | RWST | 44.5 ksf |
| | RMWST | 47.9 ksf |
| | CST | 57.7 ksf |

## 4.0 STRUCTURAL ANALYSIS AND DESIGN

This section provides the methodologies employed to analyze the Category 1 tanks and to design the 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 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 primarily performed by manual analysis, with the exception of the CST basemat which is analyzed using a computer model.

In the manual analyses, each tank is considered as an assemblage of roof, wall, and basemat, and the analyses are performed using standard structural analysis techniques. The analysis techniques, application of loads, and treatment of boundary conditions are provided to illustrate the method of analysis.

The CST basemat is modeled as an assemblage of finite elements and the analysis is performed using the standard finite element method utilizing a computer program. The modeling techniques, application of loads, and boundary conditions are provided to illustrate the method of analysis.

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 each tank. It is determined that load combination equation 3 for concrete design (Appendix B, Table B.2) containing OBE, governs over all other load combinations, and hence forms the basis for the overall structural analysis and design of the Category 1 tanks.

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 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.

## 4.3 ROOF STRUCTURES

### 4.3.1 Analysis and Design Methodology

The structural system used for the roof structure of each Category 1 tank consists of precast beams and individual slab panels set in place as shoring support for the placement of a monolithic slab on top. The precast beams and panels remain in place as an integral part of the completed roof structure system. An extensive grid system of evenly spaced vertical ties unifies both the precast beams and panels with the monolithic roof slab. Typical cross sections of the roof structure are provided in figure 12.

The two T-beam sections, formed by unifying the precast beam and monolithic slab to act compositely, are designed to span between the tank walls. They are designed to be stiff relative to the slab in order to limit the moment developed at the junction of the slab and wall. The additional stiffness provided by the precast panels is conservatively neglected. The roof loads are applied to the T-beams as equivalent uniform pressure loads and are determined on the basis of tributary areas. The T-beams are analyzed using standard beam formulas.

The T-beams are designed to satisfy relative stiffness deflection, and strength requirements. The design vertical earthquake load for the T-beams is obtained by multiplying the tributary mass, from the applied loading (including the T-beams own mass) by the maximum roof acceleration. The main reinforcing steel and beam stirrups are proportioned and detailed to meet the ACI 318 code requirements. Details of the reinforcing steel for the T-beams are provided in figure 13.

The monolithic roof slab is designed on the basis of continuous beam strips spanning in one direction over three spans. The stiffening effect of the precast panels is conservatively neglected. The loadings on the continuous beam strips are applied as uniform pressure loads.

The design vertical earthquake loads for the monolithic roof slab are obtained by multiplying the tributary mass from the applied loading (including its own mass and the mass of the precast panels) by the maximum roof acceleration.

The reinforcing is determined based on the strength requirements of the maximum span using standard beam formulas and is determined for the governing face of the slab and conservatively provided on both faces and in both directions. The reinforcing steel is sized and detailed to meet the ACI 318 code requirements. Details of the reinforcing steel for the roof slab are provided in figure 13.

#### 4.3.2 <u>Design Results</u>

The design results for the monolithic roof slab and T-beams of the RWST are summarized in table 5. The RWST has the largest roof spans of any Category 1 tank.

## 4.4 TANK WALL

### 4.4.1 Analysis and Design Methodology

The walls of the Category 1 tanks are analyzed by applying classical cylindrical shell solutions to determine the profiles of vertical moment and hoop tension forces along the height of each tank. The applied loads are envelope pressure profiles of the hydrostatic, hydrodynamic, and concrete inertial loads. The hydrodynamic pressure profile is determined in accordance with reference 1. The envelope is separated into uniform and linearly varying components, and the final force profiles are calculated by superposition of classical cylinder solutions for triangular and uniform pressure profiles. The radial growth of the cylinder is checked to insure compatibility with the liner plate.

For the determination of the hoop force profile, a pinned condition is conservatively assumed at the junction of the wall and basemat. For the vertical moment profile, a fixed condition is considered. The interaction of the wall and basemat is accounted for by distributing the moment determined for a critical basemat strip to the wall in proportion to their relative stiffness. The tension or compression of the tank wall due to the gross overturning moment is considered simultaneous with the walls' out-of-plane moment profile. Profiles are derived for both the tension and compression sides of the tank. The tanks are relatively tall so that the moment profiles derived independently for the top and bottom boundary restraints have no interaction. The radial shear is calculated based on the moment gradient.

The maximum design forces used to determine the vertical reinforcing steel requirements are based on the governing combinations of tension or compression in the wall with the corresponding vertical bending moment. The vertical reinforcing steel in the tank wall required by design is calculated using the OPTCON module of the BSAP-POST computer program.

BSAP-POST (which consists of a collection of modules that perform specific independent tasks) is a general purpose, post-processor program for the Bechtel Structural Analysis Program (BSAP) finite element analysis program. BSAP-POST reads computed BSAP results, which are usually stored on a magnetic tape, into an internal common data storage base and optionally performs one or several additional operations (e.g., plotting) or calculations (e.g., 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 that all stress and strain code criteria are satisfied.

The vertical and hoop reinforcing steel is proportioned and detailed to meet the ACI 318 code requirements. Details of the wall reinforcing for the RWST are provided in figure 13.

### 4.4.2 Design Results

The design results for the tank wall of the RWST are summarized in tables 5 and 6. The variation of RWST wall moment is provided in figure 14. The variation of hoop tension with height is provided in figure 15.

## 4.5 BASEMATS

### 4.5.1 RWST and RMWST Basemat Analysis and Design Methodology

Plan views showing the RWST and RMWST basemat dimensions are provided in figures 3 and 5 respectively.

The basemat stiffness of each tank is checked for rigidity, relative to soil stiffness, by evaluating a beam strip spanning between the tank cylinder walls at the tank centerline, using standard beam-on-elastic-foundation criteria. The basemat of each tank is determined to be rigid. A linear soil reaction profile is therefore justified in the analysis of each basemat.

The magnitude and distribution of the soil reaction loads are determined by applying statics to the overall tank structure and summing equilibrium forces at the bottom of the basemat. The result is a linearly varying soil reaction pressure profile.

The basemat is analyzed by statically applying the soil reaction pressure profile to the basemat. The centerline strip is analyzed as a beam that spans between opposite sides of the tank wall with two overhanging ends. Opposite sides of the cylindrical wall are considered as support points. The interaction of the wall and basemat moments is taken into account by distributing the critical wall moment to the basemat in proportion to relative stiffness. A second evaluation is performed to analyze the moment and shear at the corner of the square mat.

The structural design is primarily based on strength considerations and consists of proportioning and detailing the reinforcing steel to meet the ACI 318 requirements. In general, the reinforcing requirements are determined on the top and bottom faces, respectively, for the controlling design moment, and conservatively placed uniformly across the basemat, in both directions. Details of the reinforcing steel for the RWST and RMWST basemats are provided in figures 16 and 17 respectively.

### 4.5.2 CST Basemat Analysis and Design Methodology

The CST basemat supports two tanks and is analyzed utilizing a finite element model with the BSAP which is a general purpose computer program for finite element analysis. This program uses the direct stiffness approach to perform a linear elastic analysis of a three dimensional finite element model.

The finite element model is prepared using conventional modeling techniques. All critical combinations of the relative dynamic motion between adjacent tanks are considered. The model is limited to the basemat and an eight-foot high portion of each tank wall to account for its stiffening effect on the basemat. The moat area around the perimeter of the basemat is modeled using a second level of horizontal plate elements rigidly linked vertically at their common points. The plate properties in the zone of basemat thickness transition (from the upper to lower basemat sections) are approximated by assigning average thicknesses to those elements.

An isometric view and typical cross section of the model is shown in figure 18.

The loads are applied to the model as nodal and pressure loads at the baesmat level. The dead and live loads as well as the overturning moments due to the lateral loads on the tank superstructures and the missile protection structure are resolved into vertical and horizontal component forces at the top of the basemat. All vertical surface pressure loads on the basemat (including fluid forces) are input as uniformly distributed pressure loads on each basemat element.

The basic loads are input to represent the various states of fluid height and seismic motion. The basic load cases include the motion of each tank and the missile structure, respectively, in each principal direction.

The superpositions of all controlling permutations of these fundamental load cases is performed in the load combination investigation. Moment profiles are plotted for the controlling locations to investigate the top and bottom reinforcing required in each direction. Shears are determined on the basis of moment gradient. The sizing of the reinforcing steel is based on strength considerations and is determine based on the controlling design moments in each face and in each direction. The reinforcing steel is proportioned and detailed to meet the ACI 318 code requirements. Details of the CST basemat reinforcing steel are provided in figure 19.

### 4.5.3 Design Results

The design results for the RWST basemat are summarized in table 5.

## 5.0 MISCELLANEOUS ANALYSIS AND DESIGN

As described in section 4.1, the Category 1 tanks are evaluated for the effects of abnormal loads and tornado loads, where applicable on a local area basis. In addition, the overall stability of the control building is evaluated. This section describes these analyses and significant special provisions employed in the Category 1 tanks design.

## 5.1 STABILITY ANALYSIS

The overall stability of the Category 1 tanks is evaluated by determining the factor of safety against overturning and sliding. Since the foundation level (the lowest of the foundation elevations is 212'-0") is above the high water table elevation (elevation 165'-0"), the Category 1 tanks are not subject to flotation effects.

### 5.1.1 Overturning

The factor of safety against overturning is determined using the equivalent static method. 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 effect of the vertical component of the design earthquake.

### 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 Analysis Results

The minimum required factors of safety and the calculated factors of safety for stability are provided in table 7.

## 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, as necessary to satisfy design requirements.

The steel access manholes and covers, provided on each of the tank roofs, are designed to meet the tornado design requirements. Independent missile protection structures are provided for each tank which enclose and protect the penetration piping. Continuous protection is provided for all connecting piping running to each of the tanks. Each of the enclosure structures is designed to meet all Category 1 requirements.

The methodology used to analyze and design the structural elements to withstand the tornado load effects is described in reference 2. 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 8.

## 5.3 ABNORMAL LOADS EFFECTS

The abnormal thermal effects applicable to the RMWST are generated by postulated plant accidents involving safety-related plant components. The RMWST water may become heated for a sustained period which allows a temperature differential to develop across the cylinder wall.

The wall of the RMWST is analyzed for abnormal temperature effects using the OPTON module of the BSAP-POST computer program discussed in section 4.4.1. OPTCON also has the capability of calculating the thermal moment, considering the concrete cracking and reinforcement yielding effects, due to a given linear thermal gradient (i.e., a difference in temperature between the two concrete faces). For each load combination, the state of stress and strain is determined before the thermal load is applied. Then the thermal moment is approximated based upon an iterative approach which considers equilibrium and compatibility conditions, and is based on the assumption that the section is free to expand axially without any constraints. 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.

The effect of abnormal temperature loads is considered in load combination equations 9 through 11 of Appendix B, Table B.2 and governing forces are determined. For these governing forces, the reinforcing steel is determined using the provisions of ACI 318 Code and compared with the steel determined for load combination equation 3 and the governing steel is provided.

## 5.4 FOUNDATION BEARING PRESSURE

The maximum calculated bearing pressures under the governing design load conditions are provided in table 9.

## 6.0 CONCLUSION

The analysis and design of the Category 1 tanks 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.

TABLE 1

REACTOR MAKEUP WATER STORAGE TANK SEISMIC ACCELERATION VALUES

| Structure Accelerations (g's)(1) | | | | | |
|---|---|---|---|---|---|
| | SSE | | OBE | | |
| Elevation | Horiz. | Vert. | Horiz. | Vert. | Remarks |
| 220'-0" | 0.32 | 0.31 | 0.20 | 0.19 | Basemat (grade level) |
| 233'-0" | 0.32 | 0.36 | 0.20 | 0.22 | Missile structure |
| 241'-0" | 0.40 | 0.31 | 0.26 | 0.19 | Tank mid-height |
| 262'-1" | 0.55 | 0.31 | 0.32 | 0.19 | Roof |

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

TABLE 2

REFUELING WATER STORAGE TANK SEISMIC ACCELERATION VALUES

| Structure Accelerations (g's)(1) | | | | | |
|---|---|---|---|---|---|
| | SSE | | OBE | | |
| Elevation | Horiz. | Vert. | Horiz. | Vert. | Remarks |
| 220'-0" | 0.26 | 0.30 | 0.16 | 0.18 | Basemat (grade level) |
| 234'-6" | 0.29 | 0.36 | 0.18 | 0.22 | Missile structure |
| 252'-0" | 0.40 | 0.30 | 0.24 | 0.18 | Tank mid-height |
| 284'-0" | 0.58 | 0.30 | 0.35 | 0.18 | Roof |

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

TABLE 3

CONDENSATE STORAGE TANK SEISMIC ACCELERATION VALUES

| Structure Accelerations (g's)[1] | | | | | |
|---|---|---|---|---|---|
| Elevation | SSE | | OBE | | Remarks |
| | Horiz. | Vert. | Horiz. | Vert. | |
| 220'-0" | 0.27 | 0.33 | 0.16 | 0.20 | Basemat (grade level) |
| 245'-6" | 0.32 | 0.33 | 0.19 | 0.20 | Tank mid-height |
| 246'-1" | 0.32 | 0.33 | 0.19 | 0.20 | Missile structure |
| 269'-3" | 0.37 | 0.33 | 0.22 | 0.20 | Roof |

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

TABLE 4

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-$ft^2$ projected area) | 4000 | 0 | 75 | 60 |

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

TABLE 5

REFUELING WATER STORAGE TANK DESIGN RESULTS

| Element | Governing Load Combination Equation | Design Force | $A_s$ Required (in.$^2$/ft) | $A_s$ Provided (in.$^2$/ft) | Reinforcement Provided |
|---|---|---|---|---|---|
| T-Beam | 3 | Mu = 4941 (ft-k) | 17.7 | 24.0 | 6 No. 18 on tension face |
| Roof Slab | 3 | Mu = 30 (ft-k/ft) | 0.75 | 1.0 | No. 9 @ 12" on center each face each way |
| Wall-Hoop Tension | 3 | Pu = 126 (k/ft) | 2.31 | 3.12 | 1 - No. 11 @ 12" on center each face |
| Basemat - Top (@ Center) | 3 | Mu = 298 (ft-k/ft) | 1.59 | 2.08 | 1 - No. 11 @ 9" on center each way |
| Basemat - Bottom (@ Corner) | 3 | Mu = 630 (ft-k/ft) | 3.68 | 4.16 | 2 - No. 11 @ 9" on center each way |

TABLE 6

REFUELING WATER STORAGE TANK WALL DESIGN RESULTS FOR MOMENT

| Wall Condition | Governing Load Combination Equation | Design Forces | | Moment Capacity of Reinforced Section For Given Axial Force (ft-k/ft) | Reinforcement Provided |
|---|---|---|---|---|---|
| | | Axial Force (k/ft) | Vertical Moment (ft-k/ft) | | |
| Moment at Base of Wall Under Maximum Tension | 3 | 78.8 | 76.2 | 123.4 | No. 11 @ 12" on center each face |
| Maximum Moment at Base of Wall Under Tension | 3 | 23.5 | 101.5 | 194.0 | No. 11 @ 12" on center each face |
| Maximum Moment at Base of Wall Under Compression | 3 | -78.7 | 157.6 | 299.3 | No. 11 @ 12" on center each face |
| Maximum Moment at Base of Wall Under Minimum Compression | 3 | -34.5 | 109.7 | 257.5 | No. 11 @ 12" on center each face |

TABLE 7

FACTORS OF SAFETY FOR STRUCTURAL STABILITY

Refueling Water Storage Tank

| Load(1)(2) Combination | Overturning Factor of Safety | | Sliding Factor of Safety | |
|---|---|---|---|---|
| | Minimum Required | Calculated | Minimum Required | Calculated |
| D + H + E | 1.5 | 2.5 | 1.5 | 2.1 |
| D + H + E' | 1.1 | 2.1 | 1.1 | 1.2 |

Reactor Makeup Water Storage Tank

| Load(1)(2) Combination | Overturning Factor of Safety | | Sliding Factor of Safety | |
|---|---|---|---|---|
| | Minimum Required | Calculated | Minimum Required | Calculated |
| D + H + E | 1.5 | 5.1 | 1.5 | 2.3 |
| D + H + E' | 1.1 | 2.8 | 1.1 | 1.4 |

Condensate Storage Tank

| Load(1)(2) Combination | Overturning Factor of Safety | | Sliding Factor of Safety | |
|---|---|---|---|---|
| | Minimum Required | Calculated | Minimum Required | Calculated |
| D + H + E | 1.5 | 3.33 | 1.5 | 1.8 |
| D + H + E' | 1.1 | 2.1 | 1.1 | 1.12 |

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

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

TABLE 8

TORNADO MISSILE ANALYSIS RESULTS[1]

| Element Description and Location | Element Size | | | Computed Ductility | Allowable Ductility |
|---|---|---|---|---|---|
| | Length (ft) | Width (ft) | Thickness (ft) | | |
| RWST Roof Slab | 15.5 | 13.72 (2) | 1.75 | 7.0 | 10.0 |
| RWST Roof T-Beam | 49 | 12.25 (3) | 5.1 (4) | 1.8 | 10.0 |
| RWST Missile Structure Roof Slab | 13.7 (5) | 9.2 (5) | 2.0 | 3.7 | 10.0 |
| RWST Missile Structure Roof Edge Radius Beam | 37.3 | 1.5 | 3.0 | 7.2 | 10.0 |
| RWST Missile Structure Wall Panel | 14.25 | 6.83 | 2.0 | 5.6 | 10.0 |

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

(2) This is the effective width of the one-way slab used.

(3) Effective width of T-beam.

(4) Effective depth of tension steel.

(5) Dimensions of equivalent rectangular slab.

TABLE 9

MAXIMUM FOUNDATION BEARING PRESSURES(1)

| Tank Structures | Gross Static (ksf) | Net Static (ksf) | Gross Dynamic (ksf) | Net Dynamic (ksf) | Allowable Net(2) Bearing Capacity | | Computed Factor of Safety(3) | |
|---|---|---|---|---|---|---|---|---|
| | | | | | Static (ksf) | Dynamic (ksf) | Static | Dynamic |
| RWST | 3.7 | 3.2 | 11.8 | 11.3 | 29.6 | 44.5 | 27.8 | 7.9 |
| RMWST | 2.3 | 1.3 | 5.3 | 4.3 | 31.9 | 47.9 | 73.6 | 22.3 |
| CST | 3.1 | 2.1 | 5.3 | 4.3 | 38.4 | 57.7 | 54.9 | 26.8 |

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

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

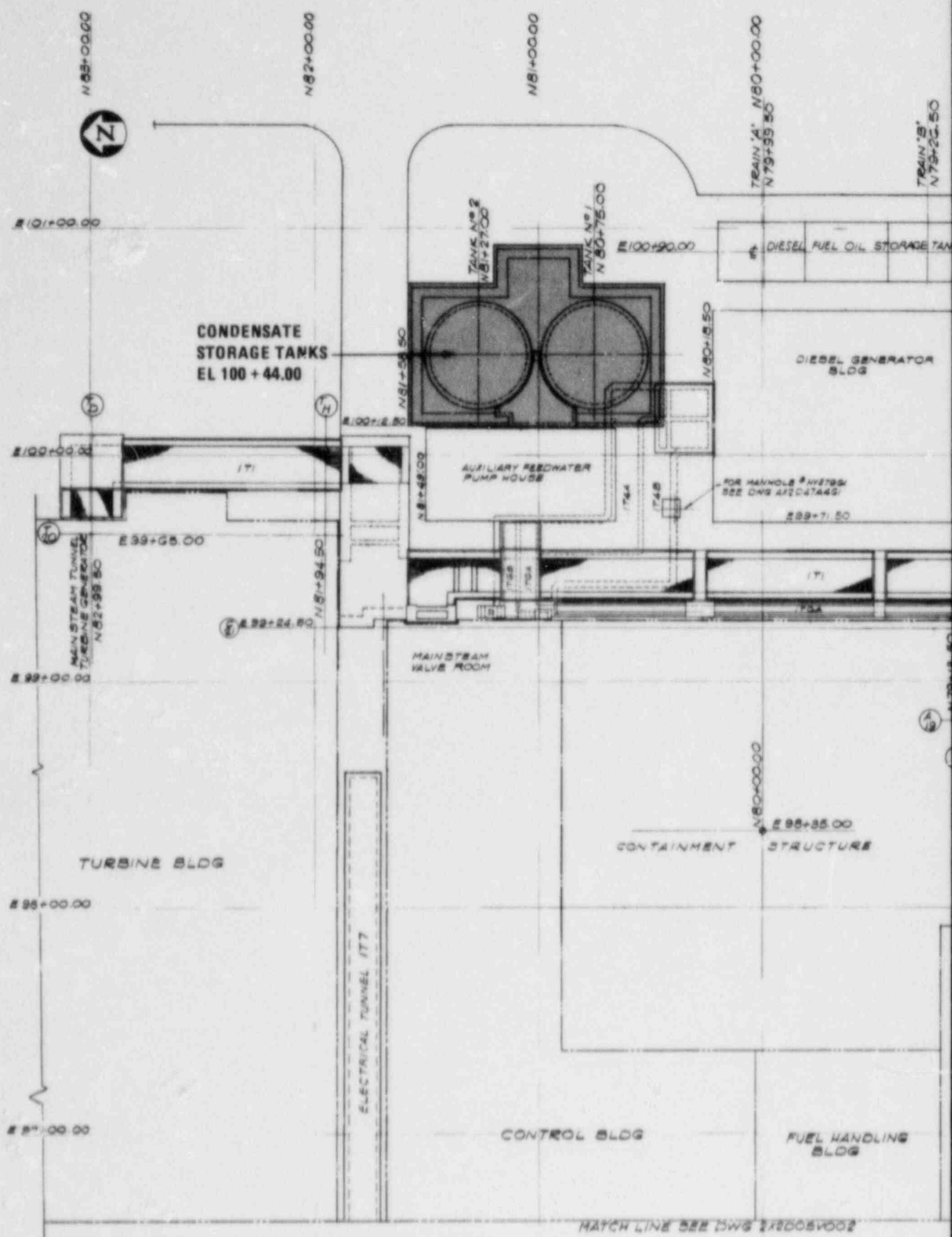
Net Static = The static pressure in excess of the overburden pressure at the base of the structure.

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.


CONDENSATE
STORAGE TANKS
EL 100 + 44.00
AUXILIARY FEEDWATER PUMP HOUSE
DIESEL GENERATOR BLDG
DIESEL FUEL OIL STORAGE TAN
TURBINE BLDG
CONTAINMENT STRUCTURE
CONTROL BLDG
FUEL HANDLING BLDG
MAINSTEAM VALVE ROOM
ELECTRICAL TUNNEL 1T7
MAIN STEAM TUNNEL TURBINE GENERATOR N82+99.50
TANK Nº 2 N81+27.00
TANK Nº 1 N80+75.00
N83+00.00
N82+00.00
N81+00.00
N80+00.00
TRAIN 'A' N79+99.50
TRAIN 'B' N79+26.50
E101+00.00
E100+90.00
E100+00.00
E99+65.00
E99+00.00
E98+00.00
E98+85.00
E99+71.50
E99+24.50
E100+12.50
N81+94.50
N80+15.50
MATCH LINE SEE DWG 2X2D05V002

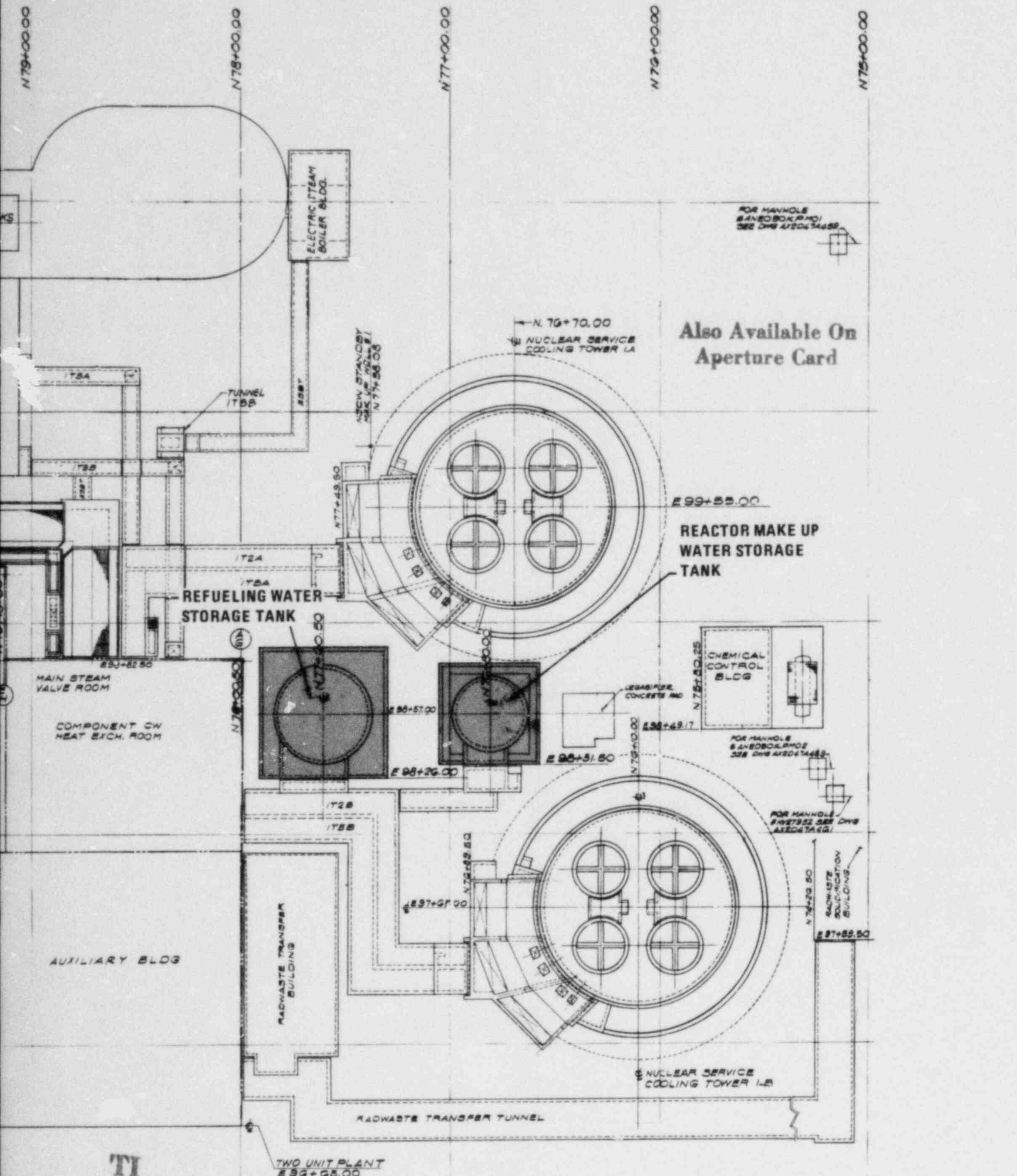
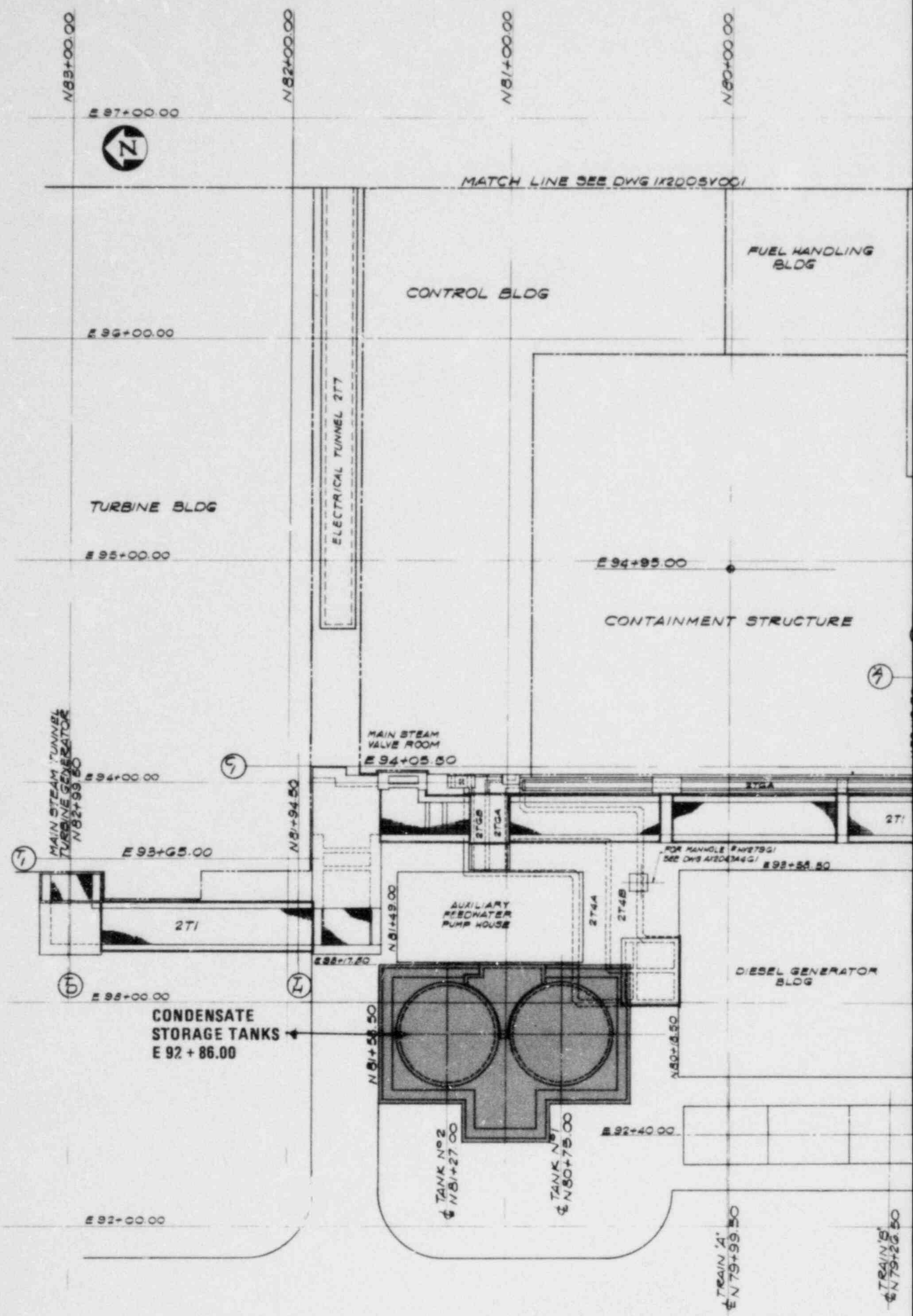





Figure 1
LOCATION OF CATEGORY 1 TANKS – UNIT 1

8411050208 -01



N83+00.00
N82+00.00
N81+00.00
N80+00.00
E 97+00.00
N
MATCH LINE SEE DWG 1X2005Y001
FUEL HANDLING BLDG
CONTROL BLDG
E 96+00.00
ELECTRICAL TUNNEL 2T7
TURBINE BLDG
E 95+00.00
E 94+95.00
CONTAINMENT STRUCTURE
MAIN STEAM VALVE ROOM
E 94+05.50
E 94+00.00
MAIN STEAM TUNNEL TURBINE GENERATOR N82+99.50
E 93+65.00
N81+94.50
2T1
AUXILIARY FEEDWATER PUMP HOUSE
N81+49.00
DIESEL GENERATOR BLDG
E 93+00.00
CONDENSATE STORAGE TANKS
E 92 + 86.00
N81+58.50
N80+15.50
E 92+40.00
℄ TANK No 2 N81+27.00
℄ TANK No 1 N80+75.00
E 92+00.00
TRAIN 'A' ℄ N79+99.50
TRAIN 'B' ℄ N79+26.50

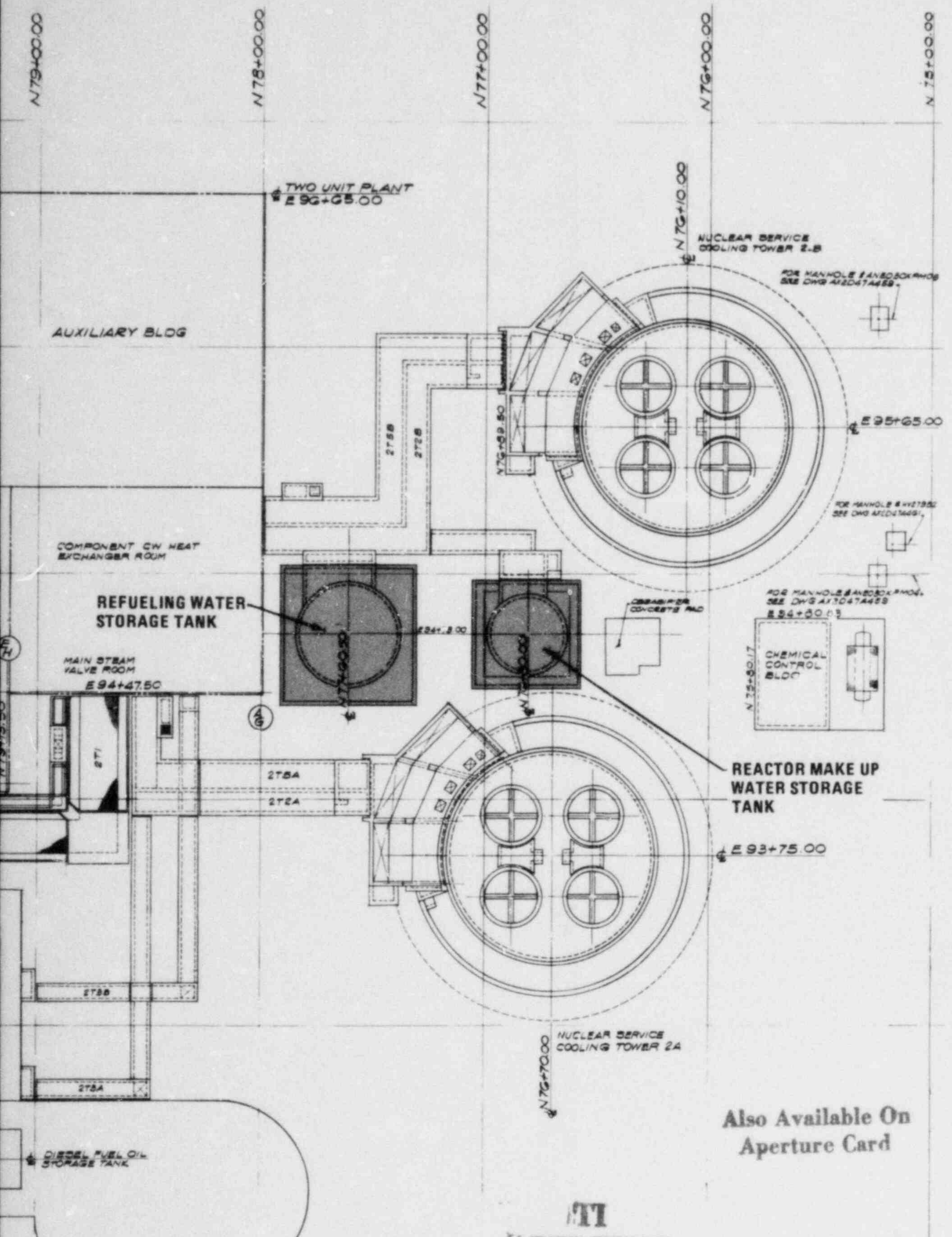



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Figure 2
LOCATION OF CATEGORY 1 TANKS – UNIT 2

8411050208 -02

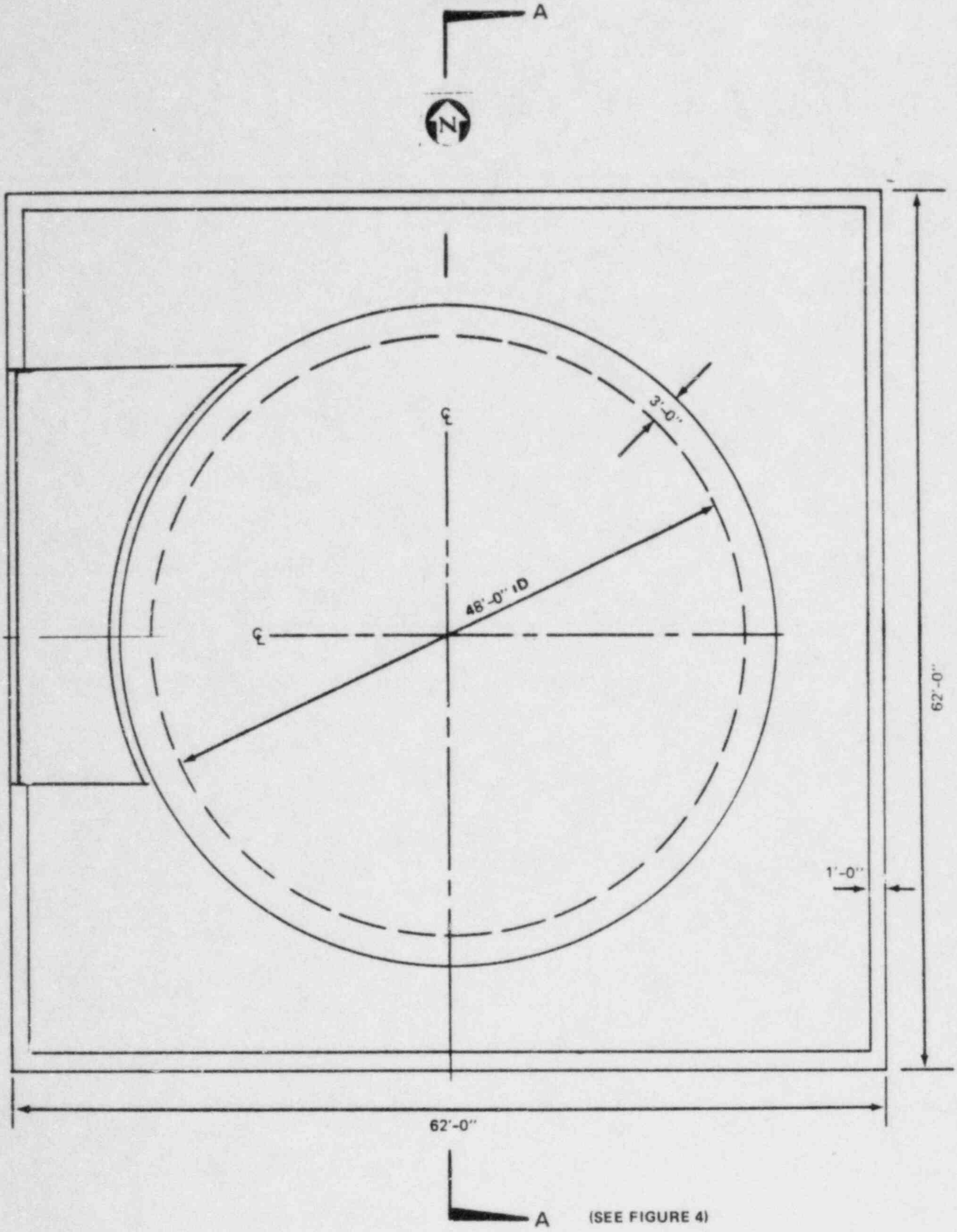



Figure 3
REFUELING WATER STORAGE TANK PLAN

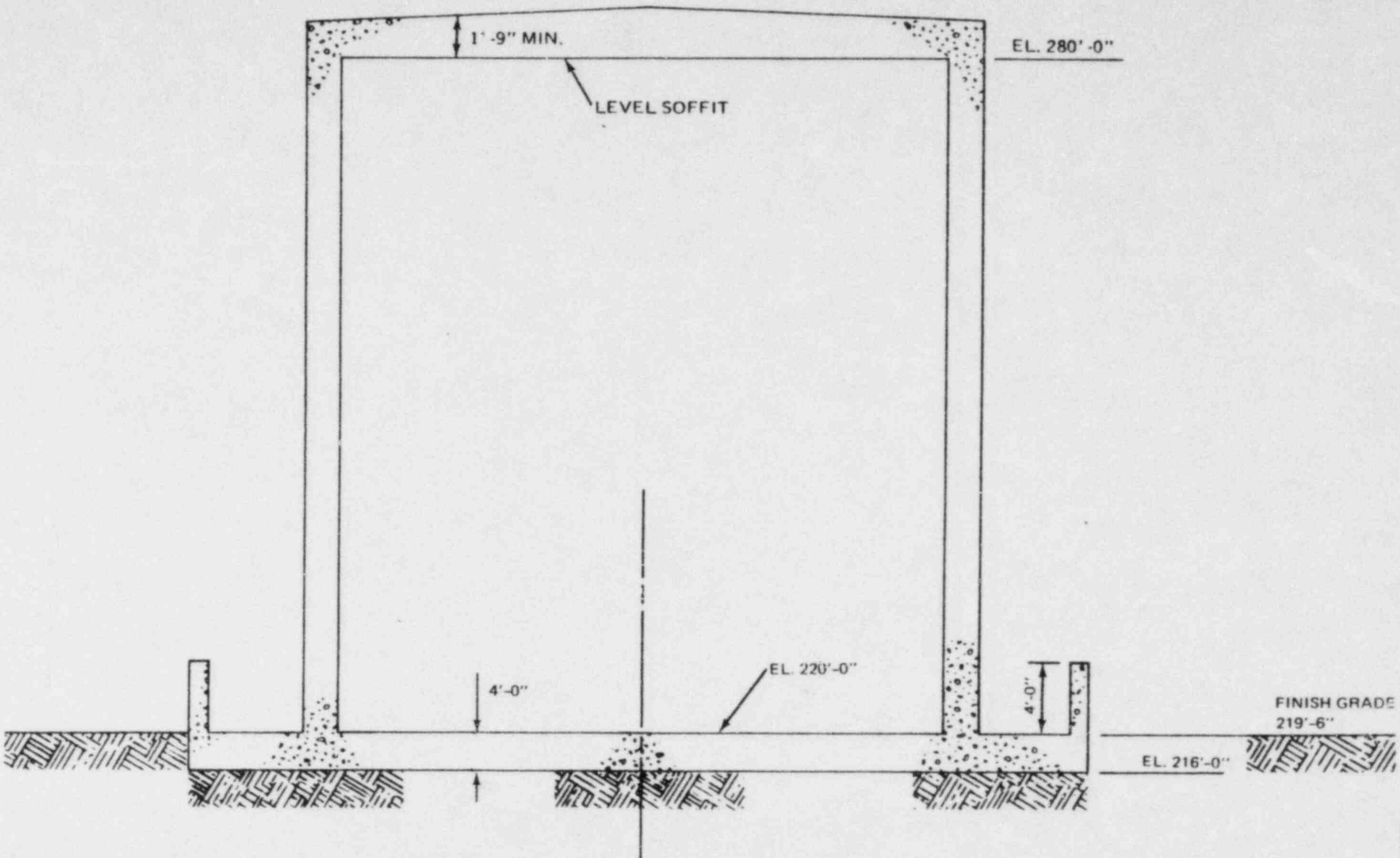



Figure 4
REFUELING WATER STORAGE TANK
SECTION

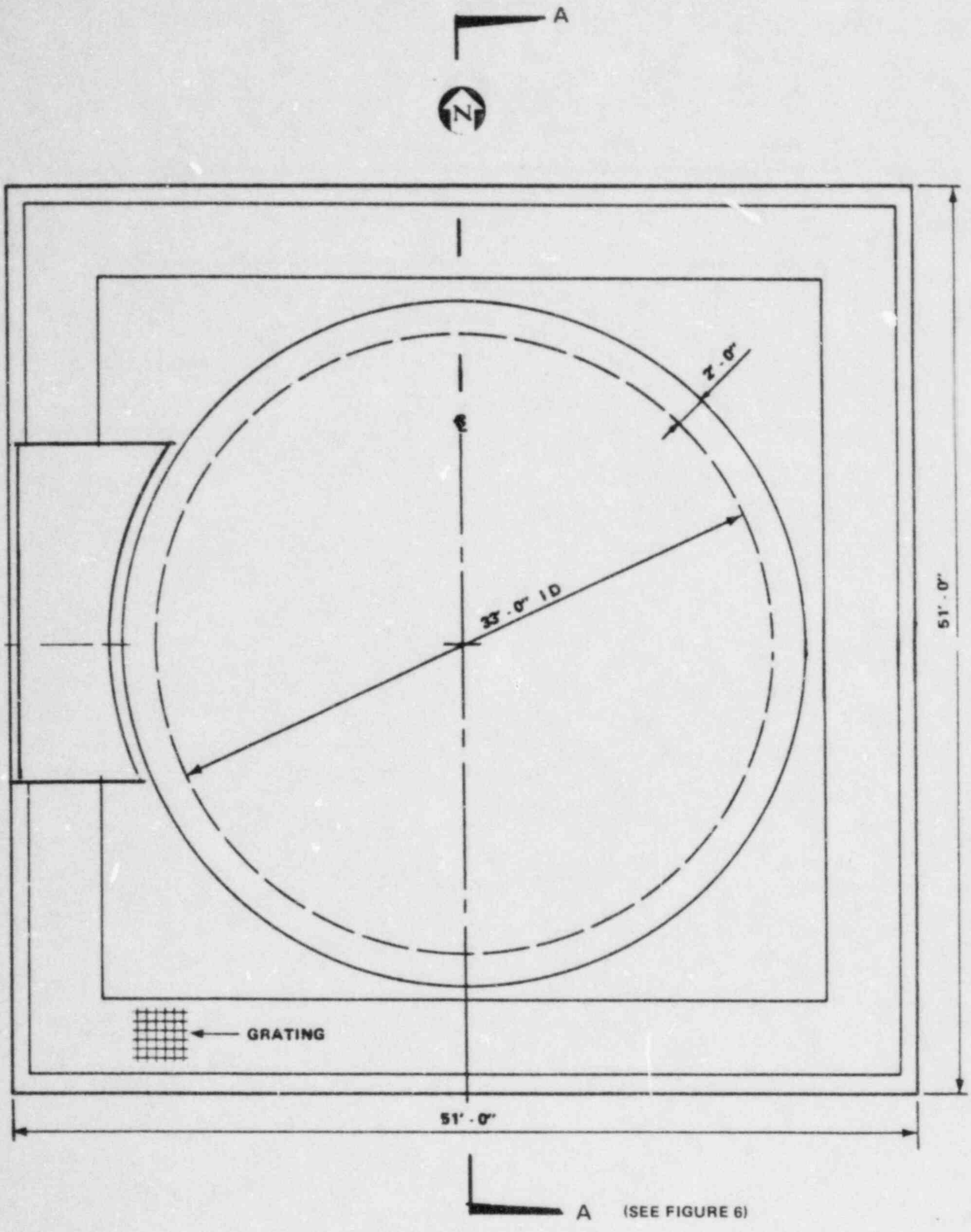



Figure 5
REACTOR MAKEUP WATER STORAGE
TANK PLAN

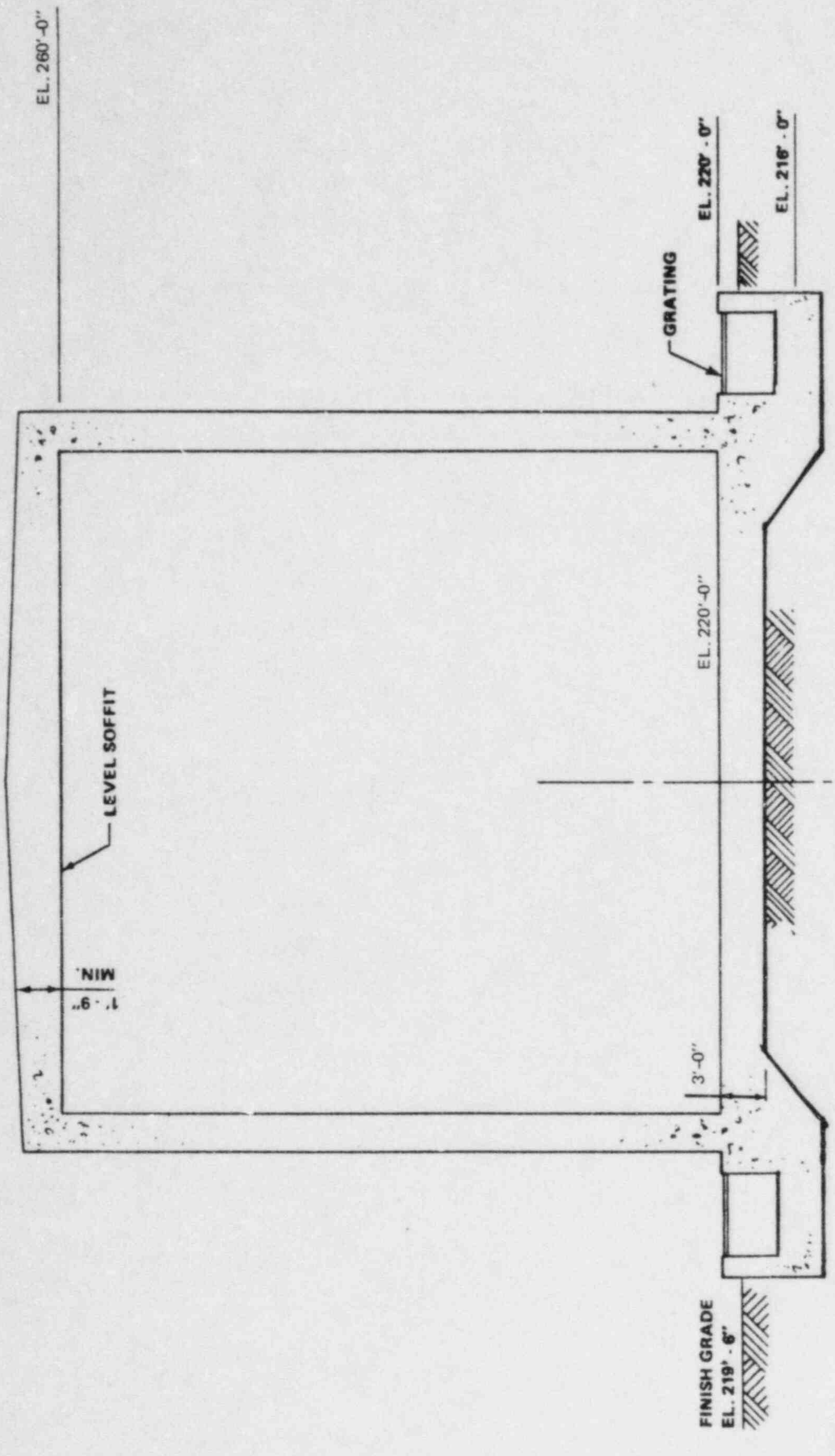



Figure 6
REACTOR MAKEUP WATER STORAGE TANK
SECTION

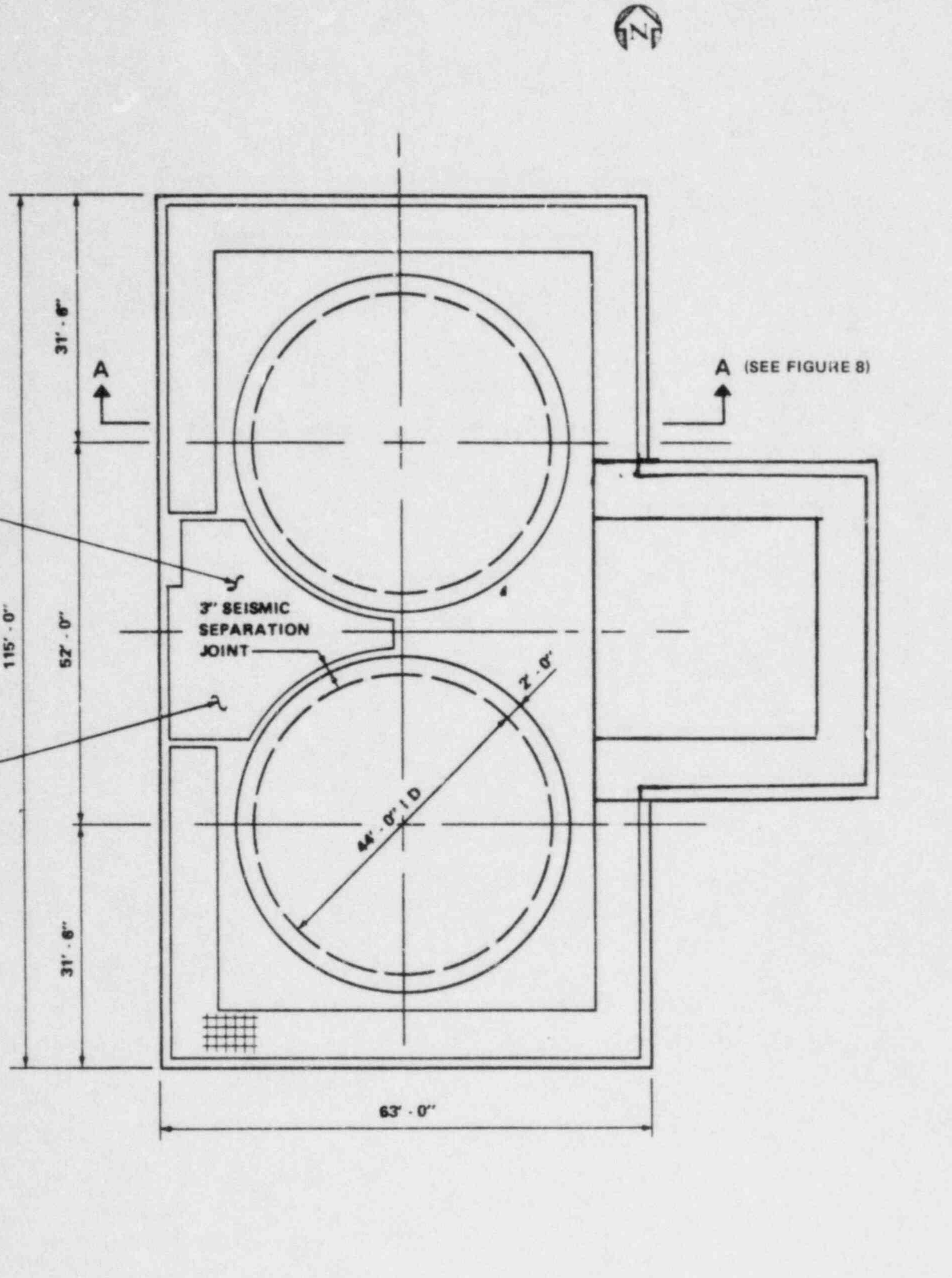



Figure 7
CONDENSATE STORAGE TANKS PLAN

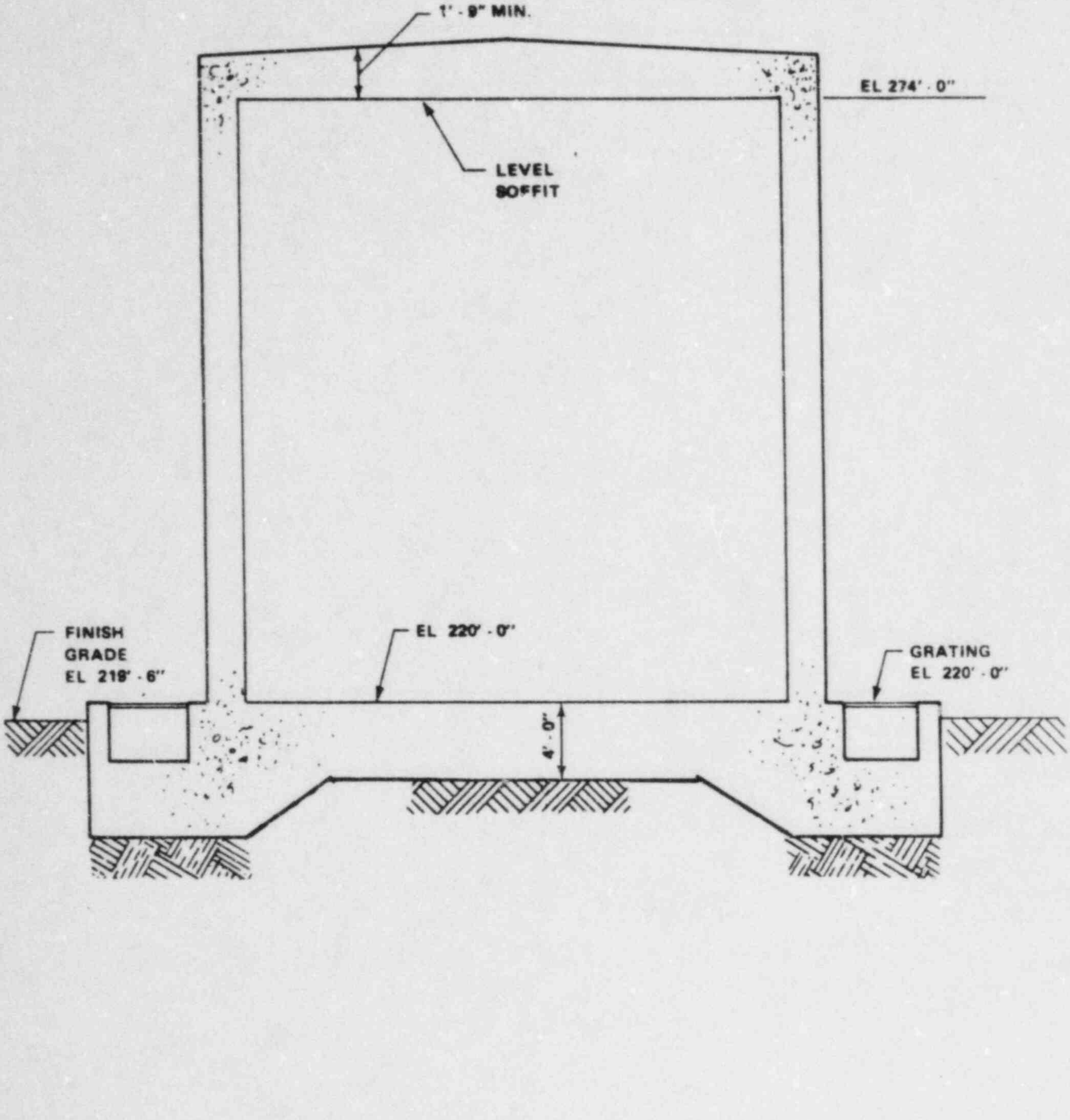
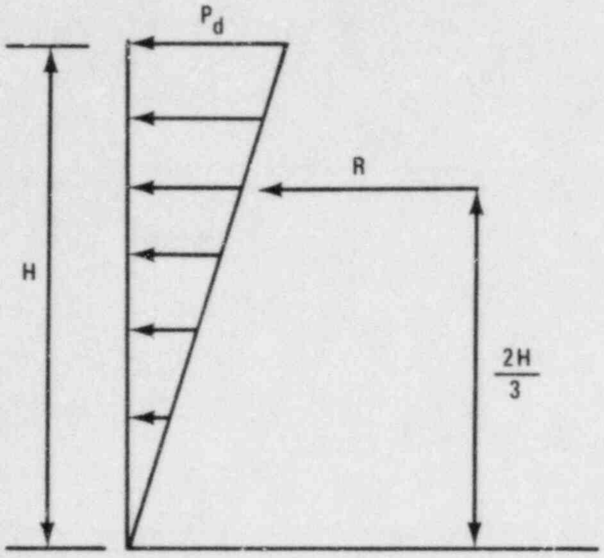



Figure 8
CONDENSATE STORAGE TANKS
SECTION




H: HEIGHT FROM BASE OF STRUCTURE TO SOIL SURFACE

$P_d$ = DYNAMIC INCREMENTAL SOIL PRESSURE

R: RESULTANT FORCE

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

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

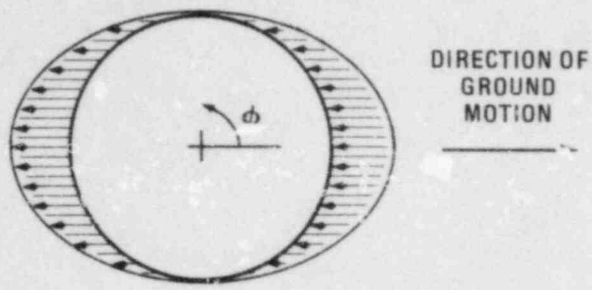
$P_d = \frac{2R}{H}$

$\gamma_M$ = SOIL MOIST UNIT WEIGHT, PCF

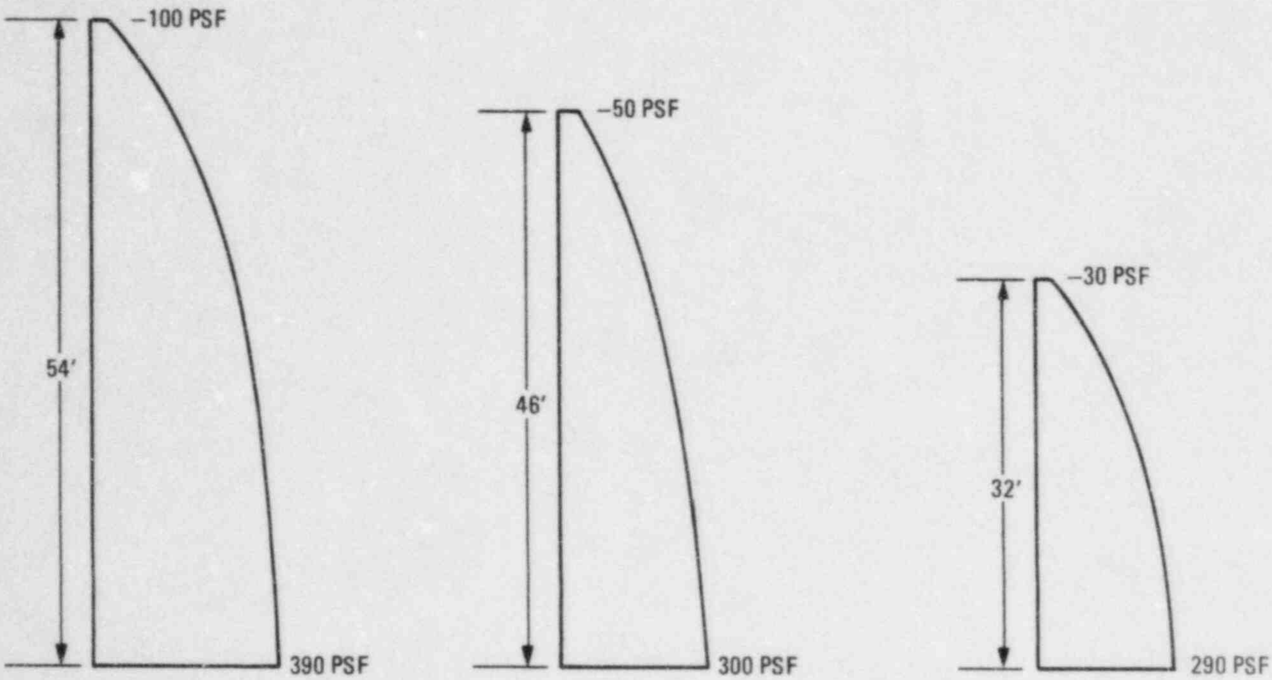
*DERIVED USING THE PEAK GROUND ACCELERATIONS OF 0.12g AND 0.20g FOR OBE AND SSE RESPECTIVELY.

Figure 9
DYNAMIC INCREMENTAL
SOIL PRESSURE PROFILE

**NOTE:** FOR A CYLINDRICAL STRUCTURE, THE EQUATIONS GIVEN IN TID-7024, APPENDIX F, (REFERENCE 4) YIELD A PRESSURE DIAGRAM AS SHOWN, WHERE FOR ANY GIVEN ELEVATION, THE PRESSURE VARIES FROM A PEAK VALUE TO ZERO AT THE SIDES. ALSO, THE PRESSURES ARE INCREMENTAL ON ONE SIDE AND DECREMENTAL ON THE OPPOSITE SIDE.




HORIZONTAL DISTRIBUTION




REFUELING WATER STORAGE TANKS

CONDENSATE STORAGE TANK

REACTOR MAKEUP WATER STORAGE TANK

PEAK VERTICAL PRESSURE PROFILES

Figure 10
OBE HYDRODYNAMIC FLUID PRESSURE PROFILES

**REACTOR MAKEUP WATER STORAGE TANK**

$P = C_s P_{max} C_p$

WHERE:

$C_s$ = SIZE COEFFICIENT
= .88

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

$C_p$ = EFFECTIVE EXTERNAL PRESSURE COEFFICIENT

$P = (.88)(332 \text{ psf}) C_p$
$= 292 C_p$ (psf)

**REFUELING WATER STORAGE TANK**

$P = C_s P_{max} C_p$

WHERE:

$C_s$ = SIZE COEFFICIENT
= .84

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

$C_p$ = EFFECTIVE EXTERNAL PRESSURE COEFFICIENT

$P = (.84)(332 \text{ psf}) C_p$
$= 279 C_p$ (psf)

**CONDENSATE STORAGE TANKS**

$P = C_s P_{max} C_p$

WHERE:

$C_s$ = SIZE COEFFICIENT
= .87

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

$C_p$ = EFFECTIVE EXTERNAL PRESSURE COEFFICIENT

$P = (.87)(332 \text{ psf}) C_p$
$= 289 C_p$ (psf)

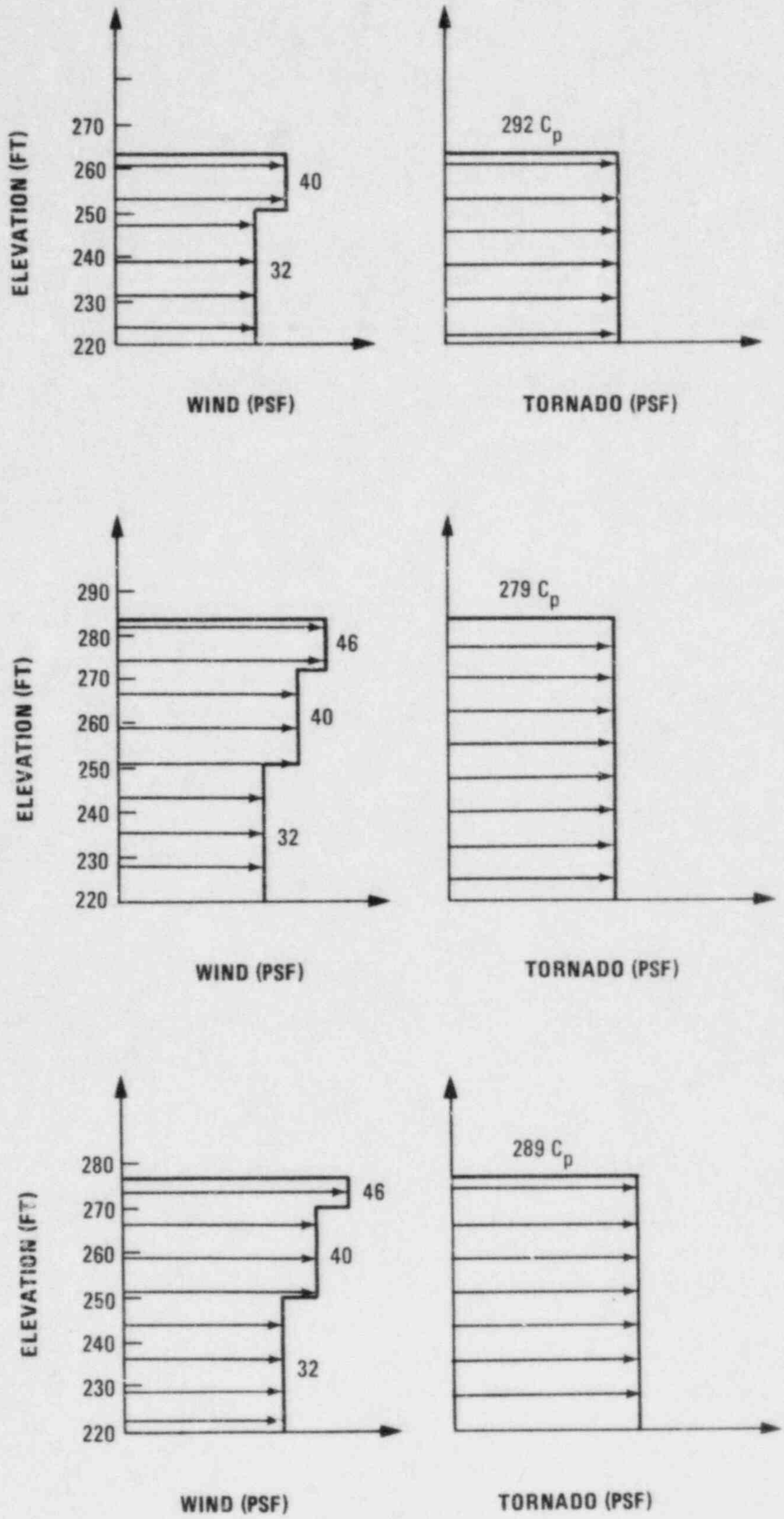
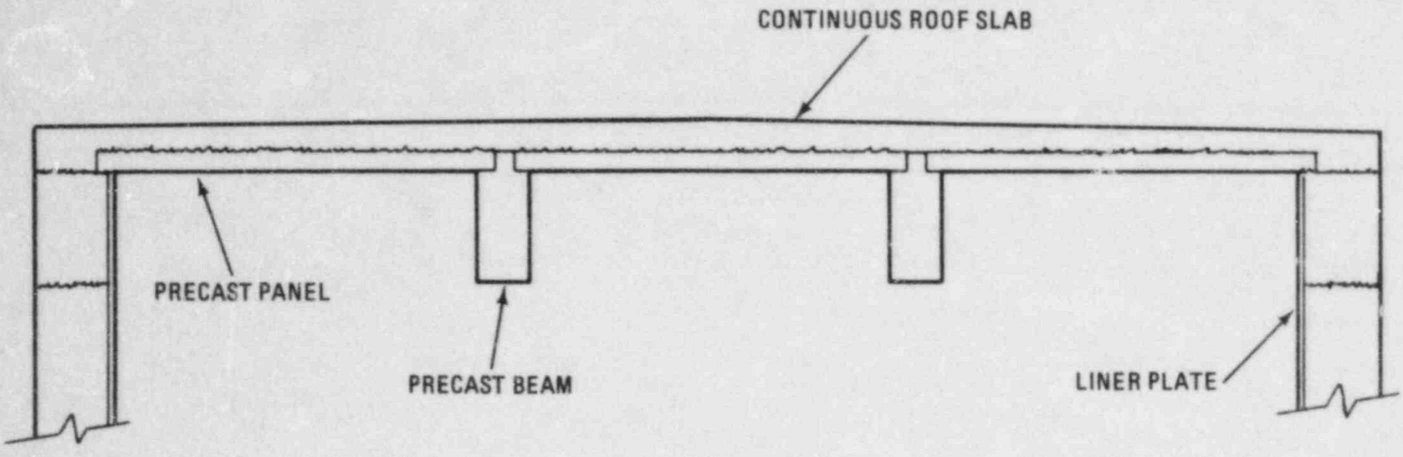
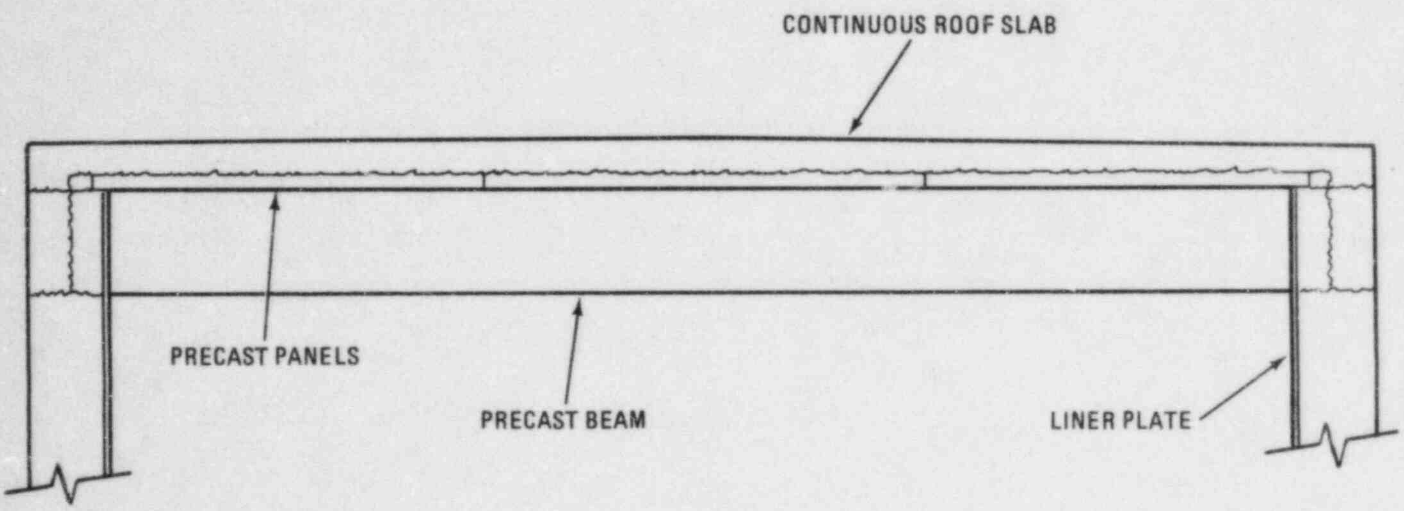



**Figure 11**
**WIND AND TORNADO EFFECTIVE VELOCITY PRESSURE PROFILES**




END VIEW OF PRECAST BEAMS




SIDE VIEW OF PRECAST BEAM

Figure 12
TYPICAL ROOF SYSTEM CROSS SECTIONS

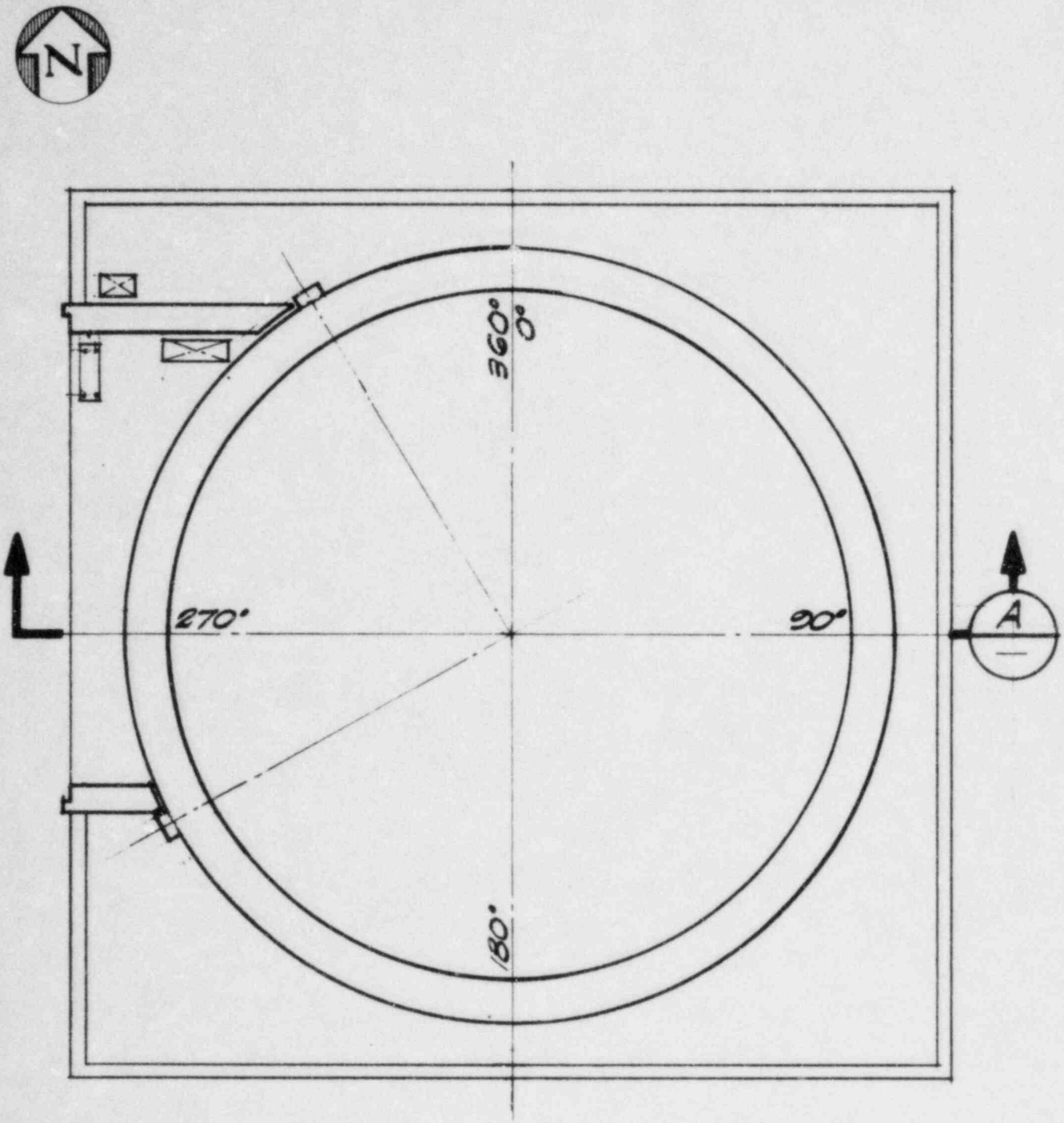



Figure 13
REFUELING WATER STORAGE TANK
REINFORCING DETAILS
(Sheet 1 of 3)

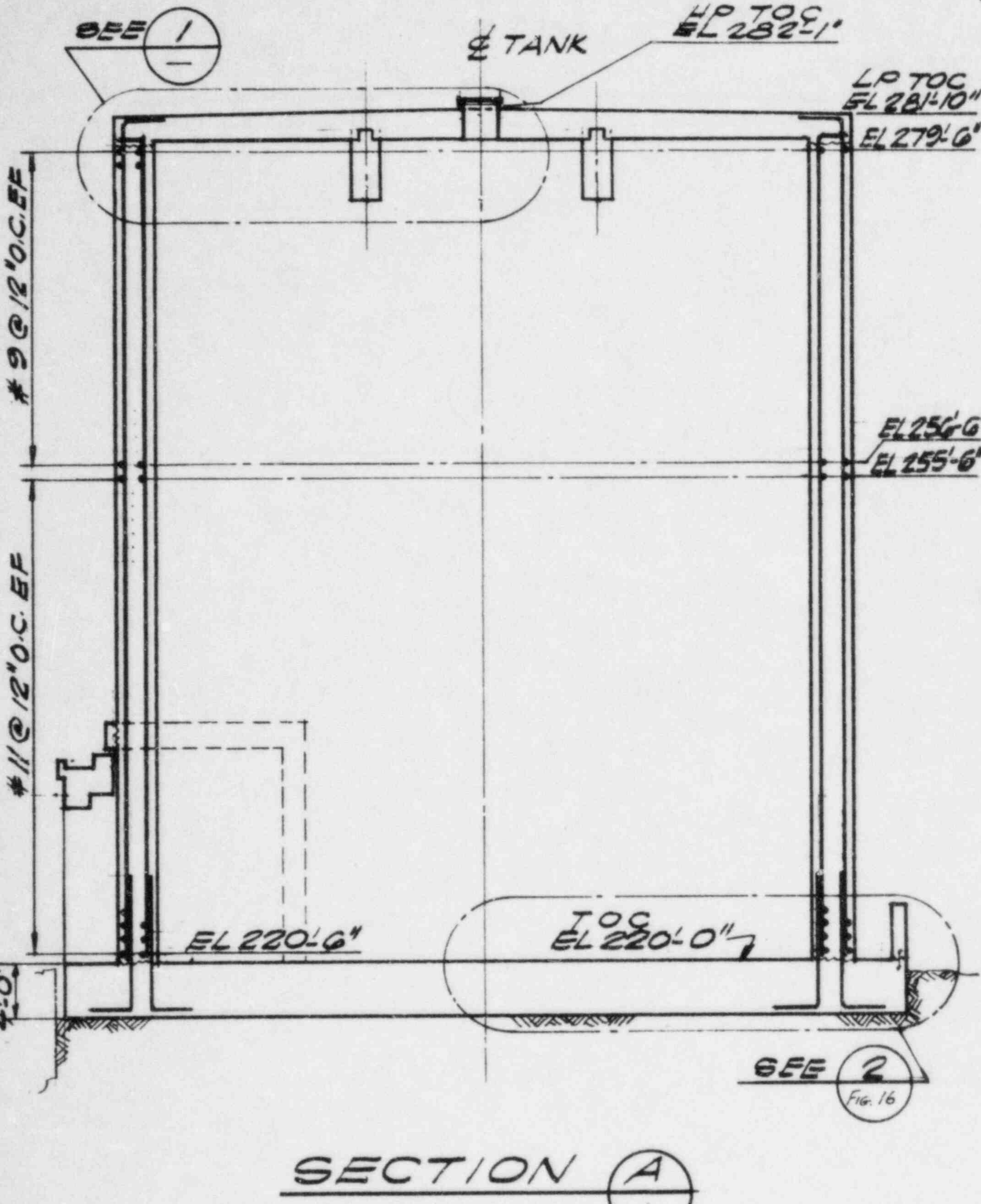



Figure 13
REFUELING WATER STORAGE TANK
REINFORCING DETAILS
(Sheet 2 of 3)

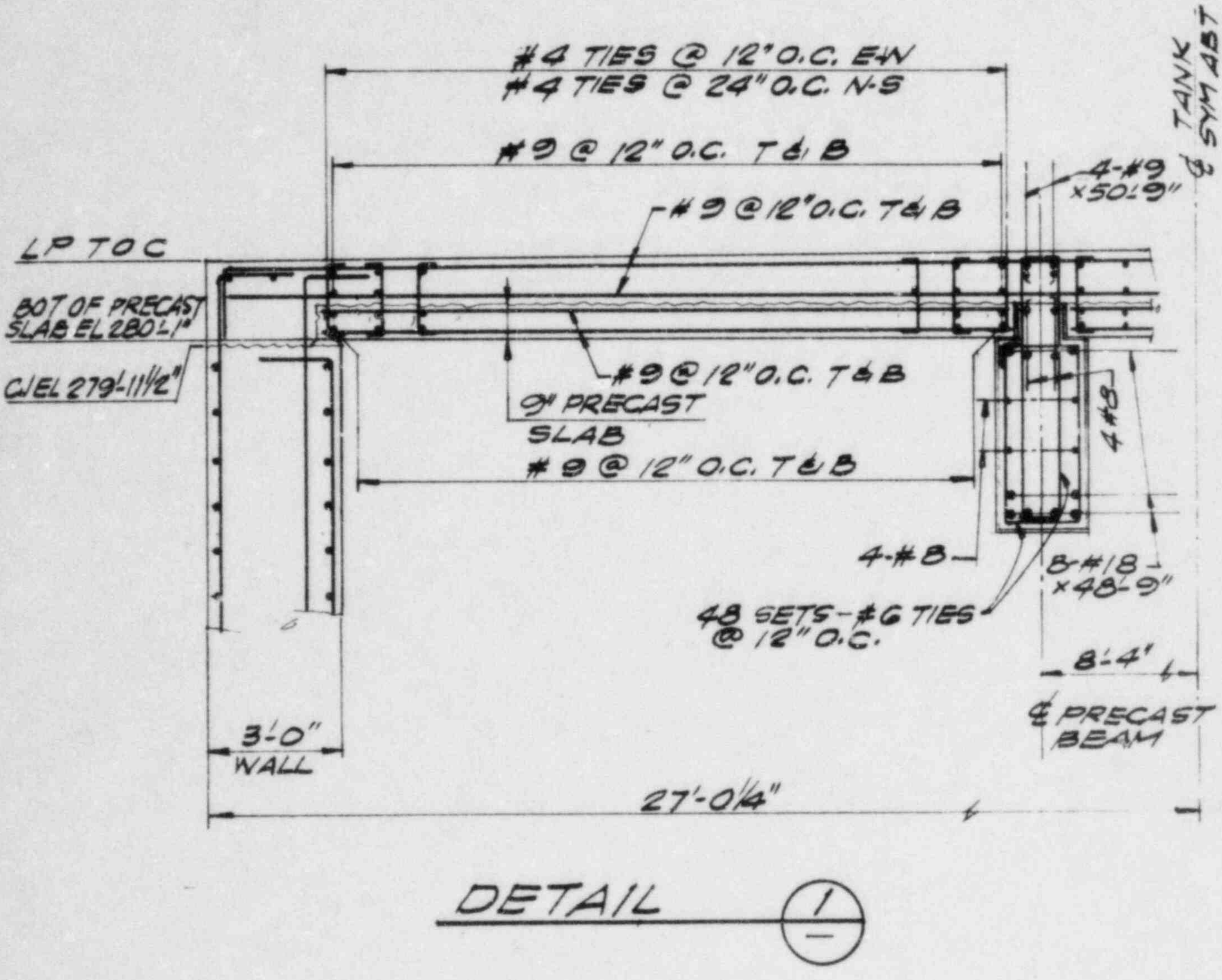
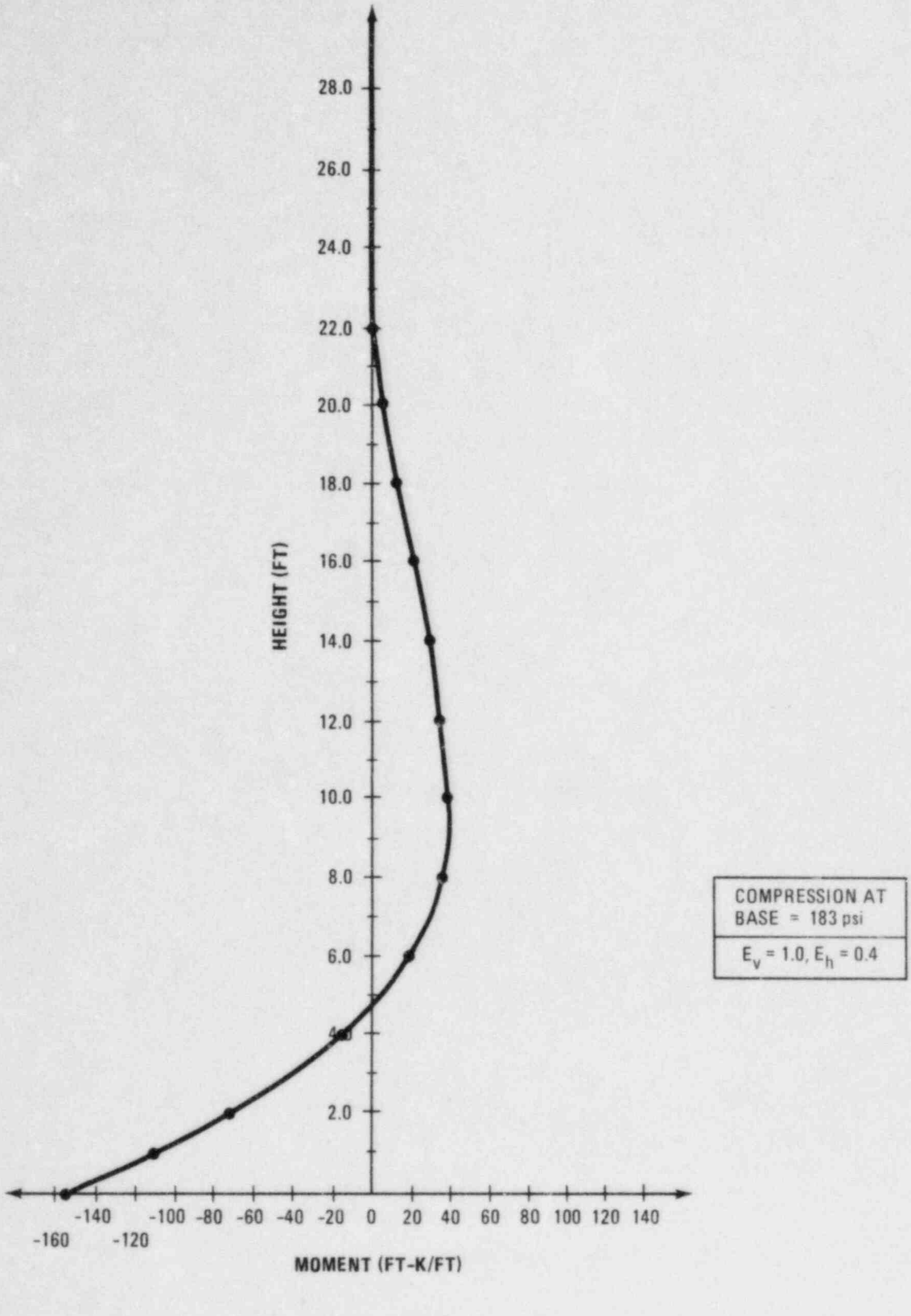
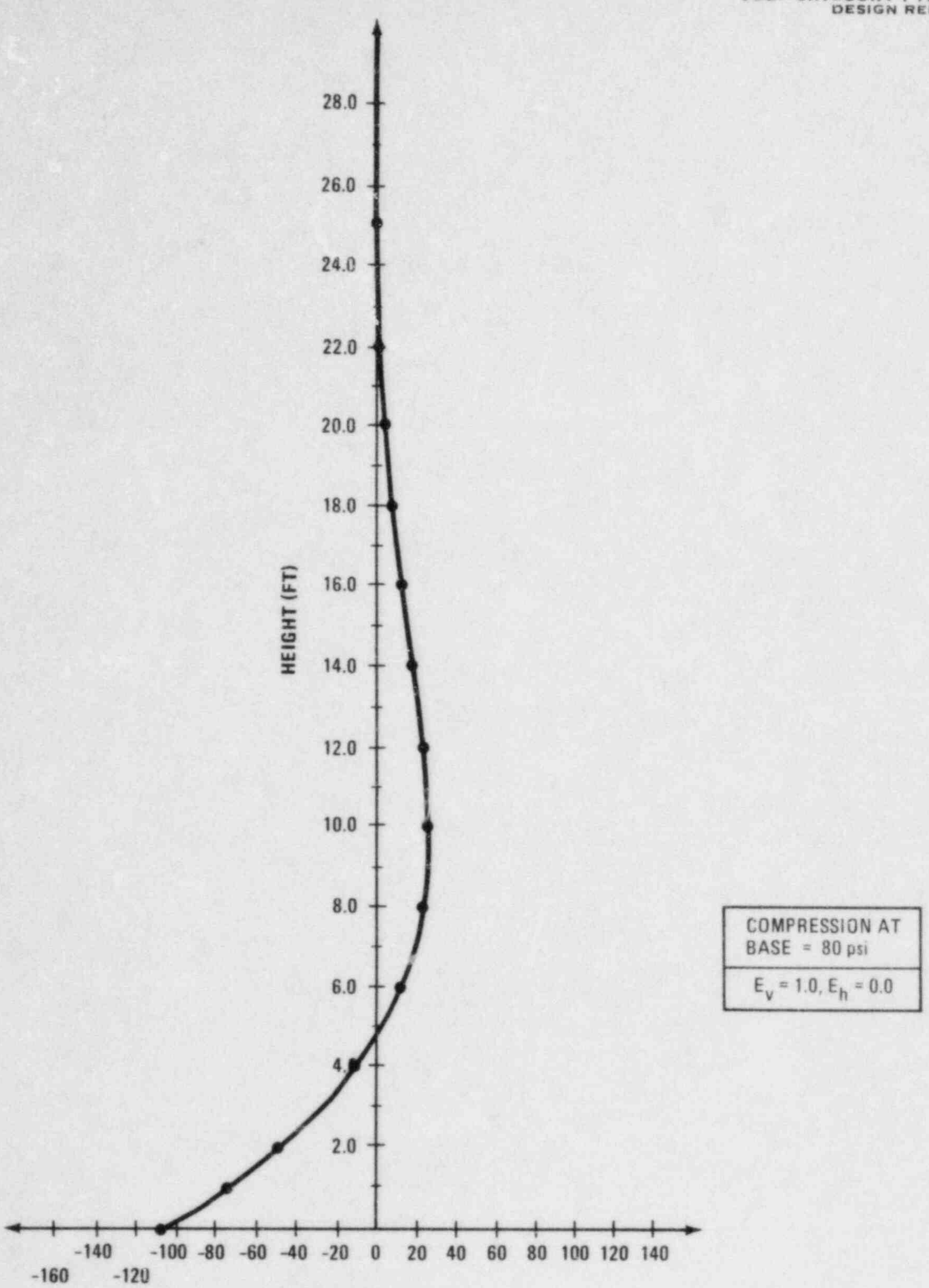



Figure 13
REFUELING WATER STORAGE TANK
REINFORCING DETAILS
(Sheet 3 of 3)



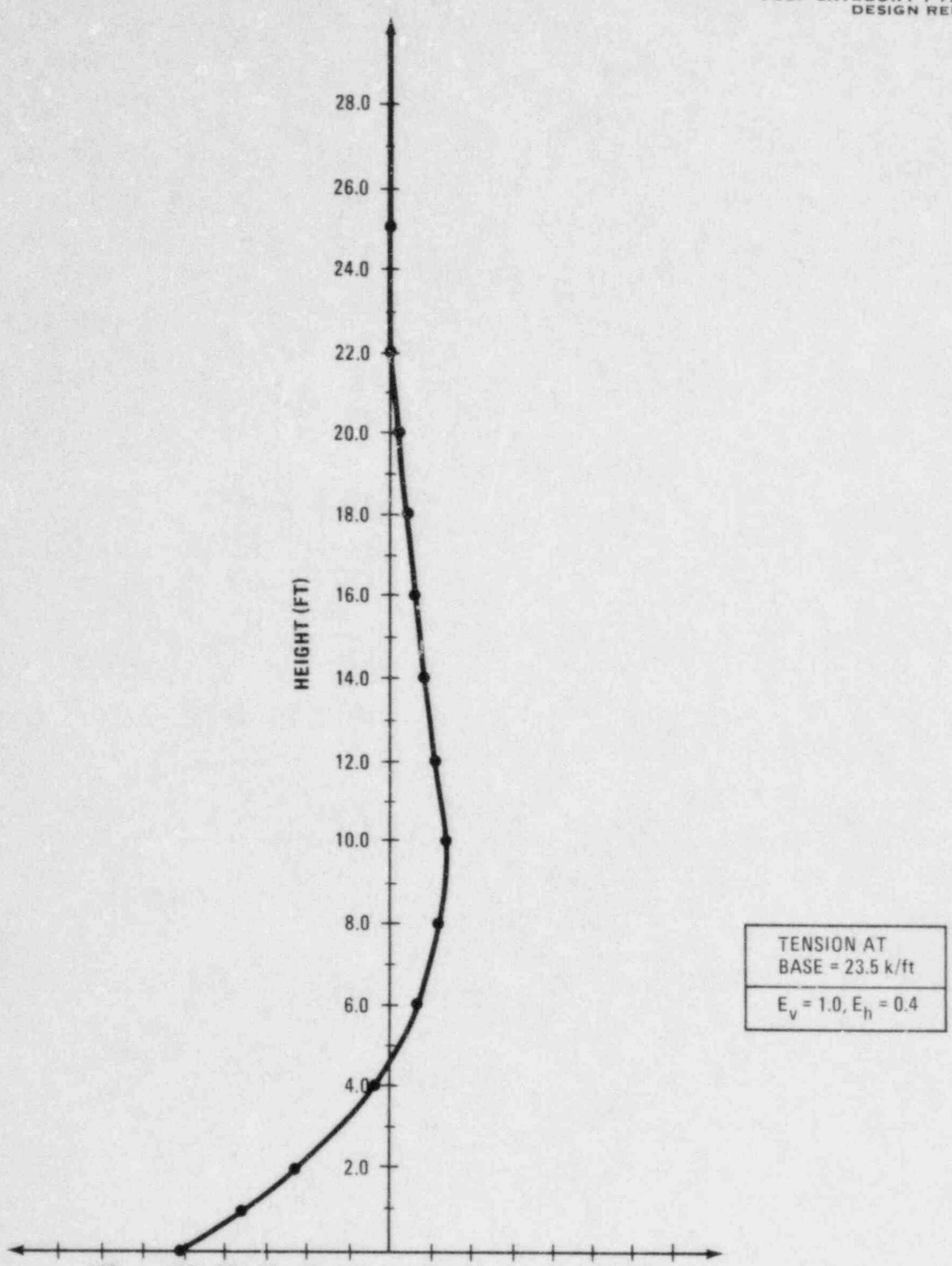

MAXIMUM MOMENT UNDER COMPRESSION

Figure 14
REFUELING WATER STORAGE TANK VERTICAL
WALL MOMENT PROFILES
(Sheet 1 of 4)



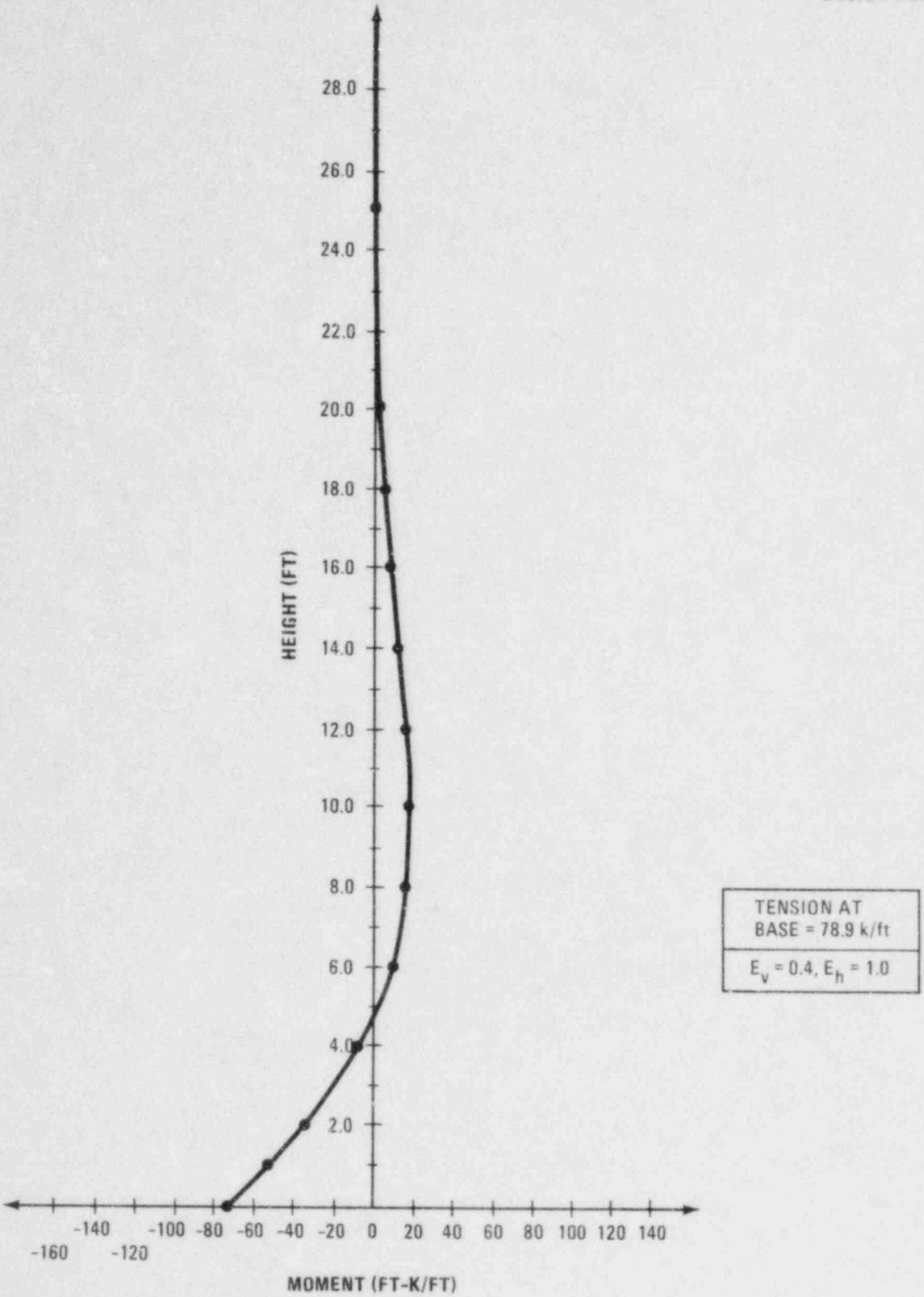

MAXIMUM MOMENT UNDER MINIMUM COMPRESSION

Figure 14
REFUELING WATER STORAGE TANK VERTICAL
WALL MOMENT PROFILES
(Sheet 2 of 4)



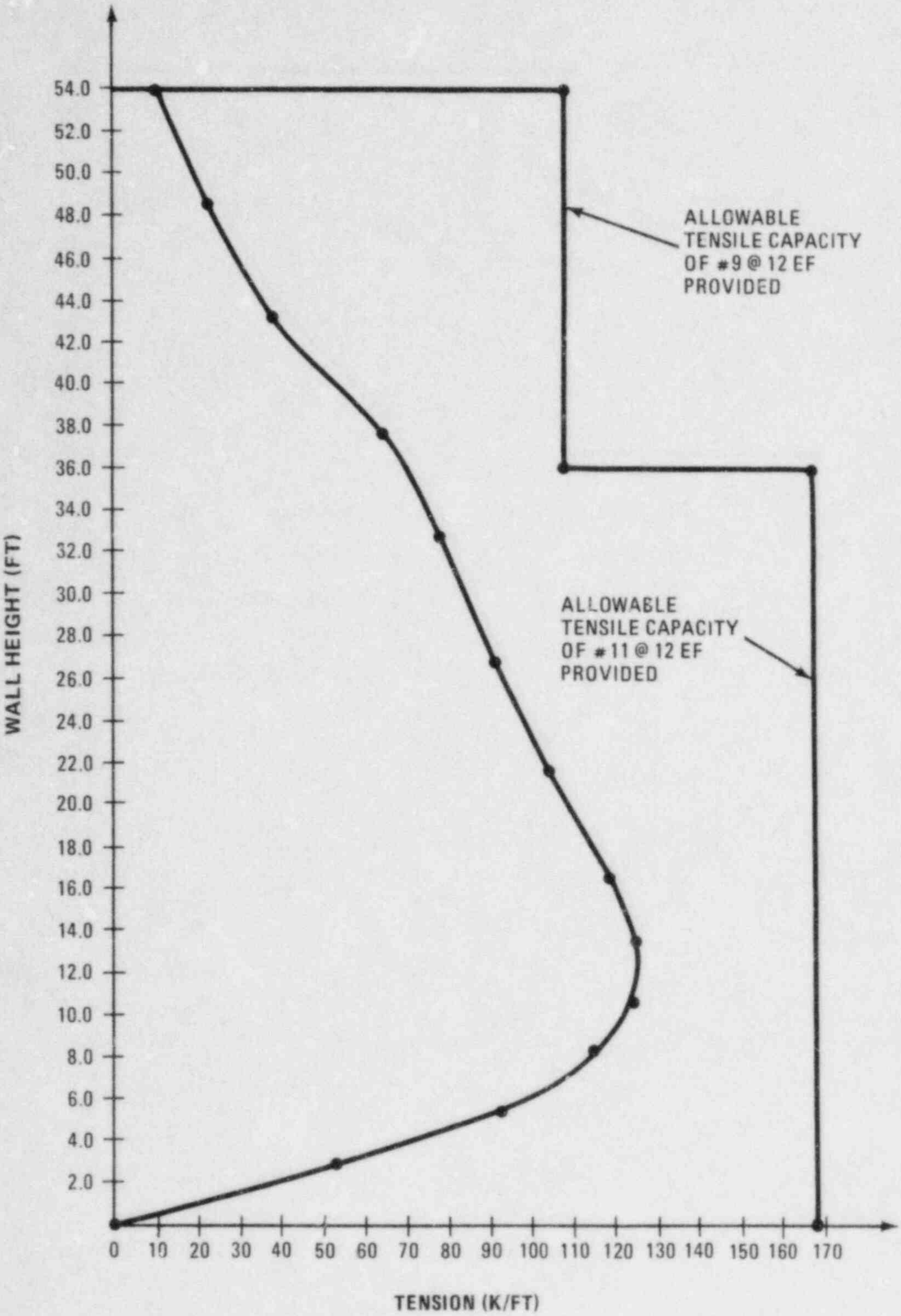

MAXIMUM MOMENT UNDER TENSION

Figure 14
REFUELING WATER STORAGE TANK VERTICAL
WALL MOMENT PROFILES
(Sheet 3 of 4)




MAXIMUM MOMENT UNDER MAXIMUM TENSION

Figure 14
REFUELING WATER STORAGE TANK VERTICAL
WALL MOMENT PROFILES
(Sheet 4 of 4)




MAXIMUM HOOP TENSION ENVELOPE

Figure 15
REFUELING WATER STORAGE TANK
HOOP TENSION PROFILE

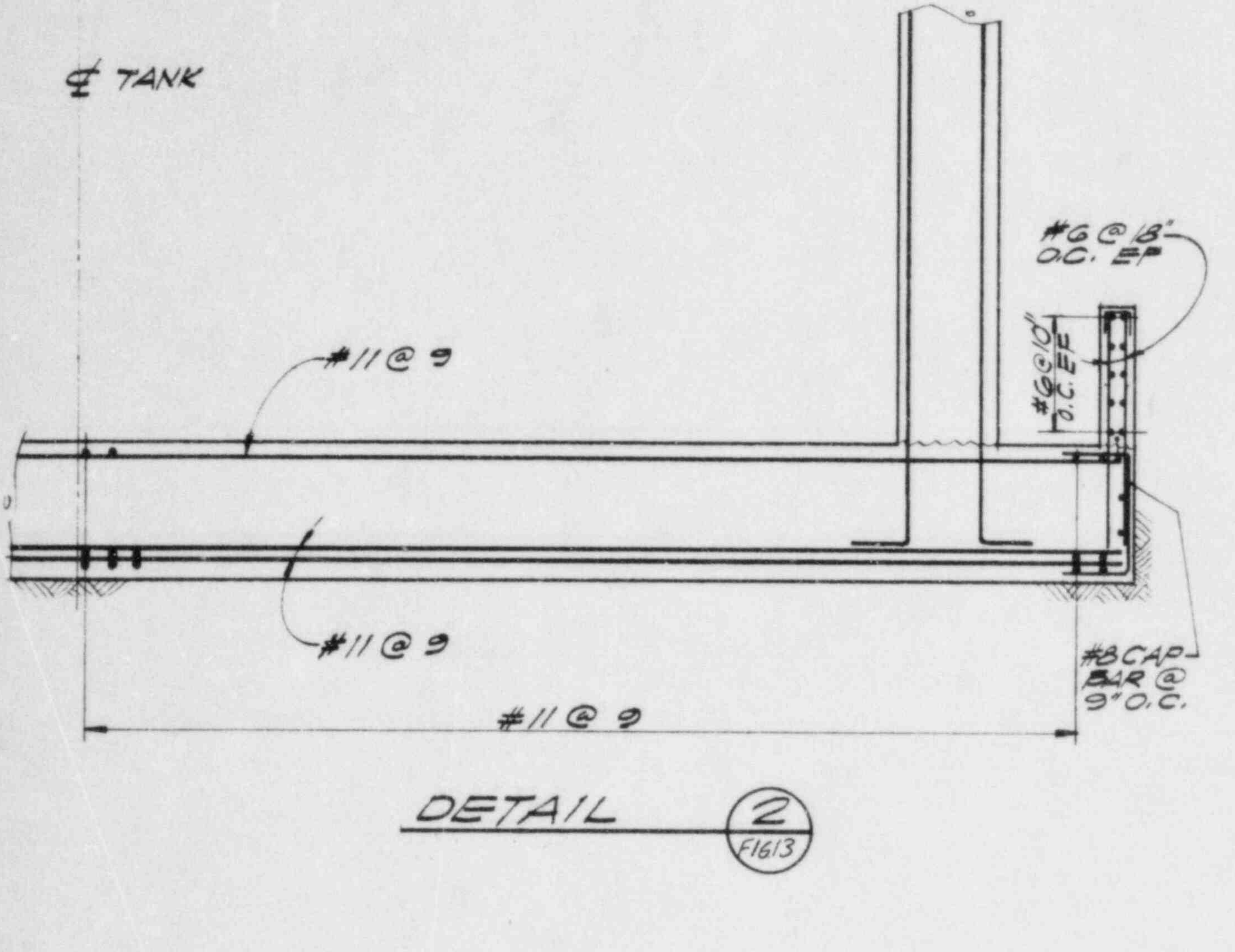



Figure 16
REFUELING WATER STORAGE TANK
BASEMAT REINFORCING DETAILS

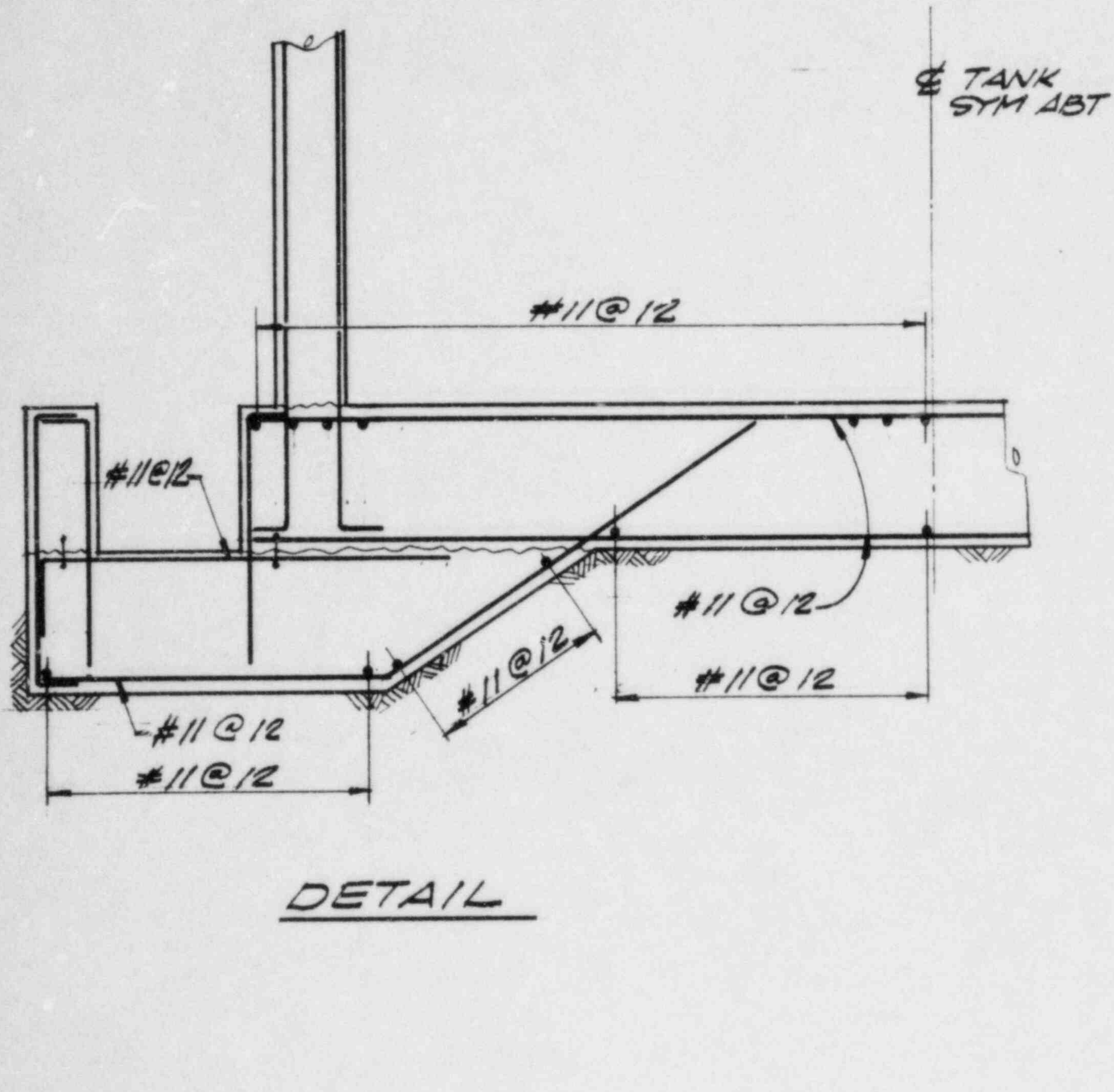



Figure 17
REACTOR MAKEUP WATER STORAGE TANK
BASEMAT REINFORCING DETAILS

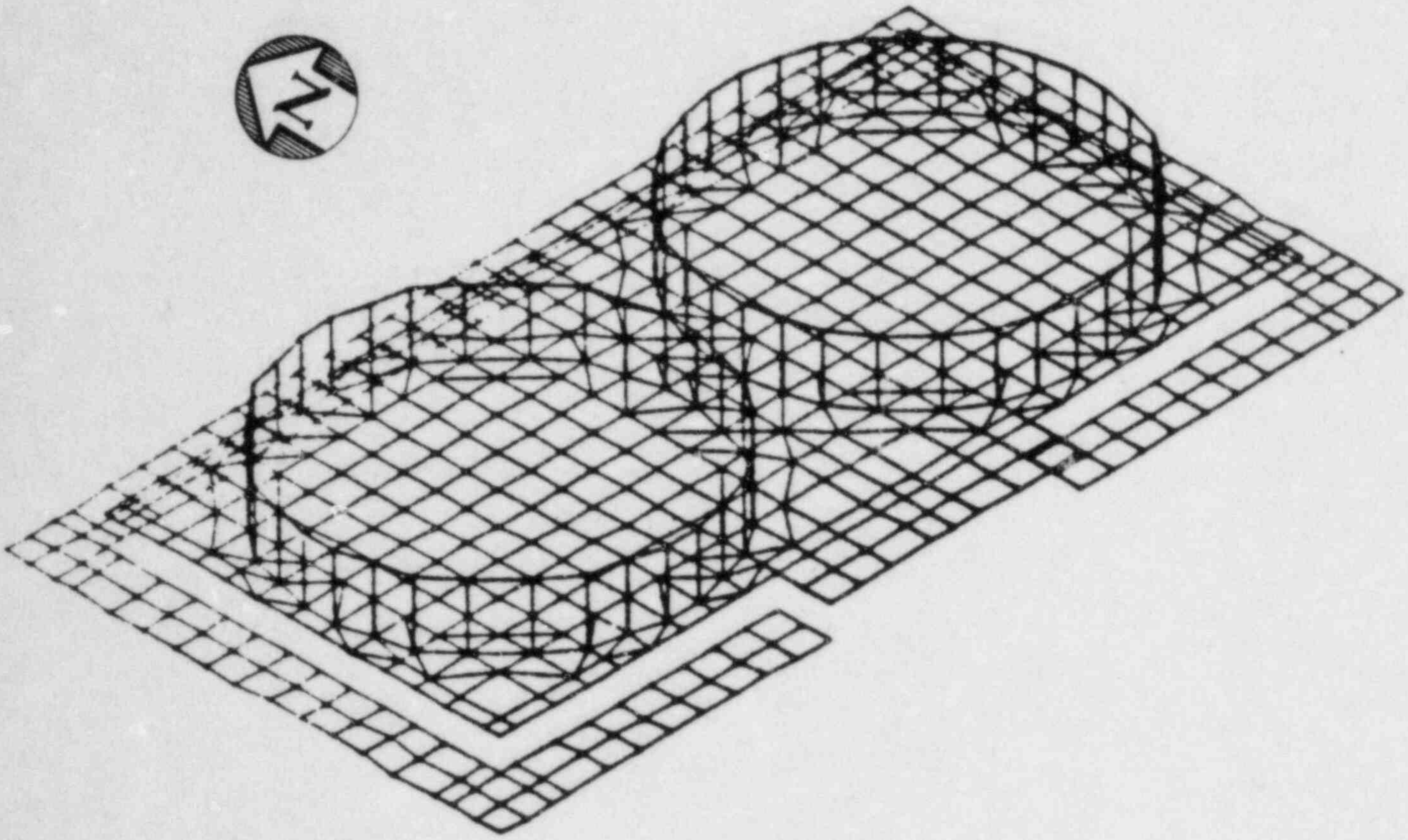
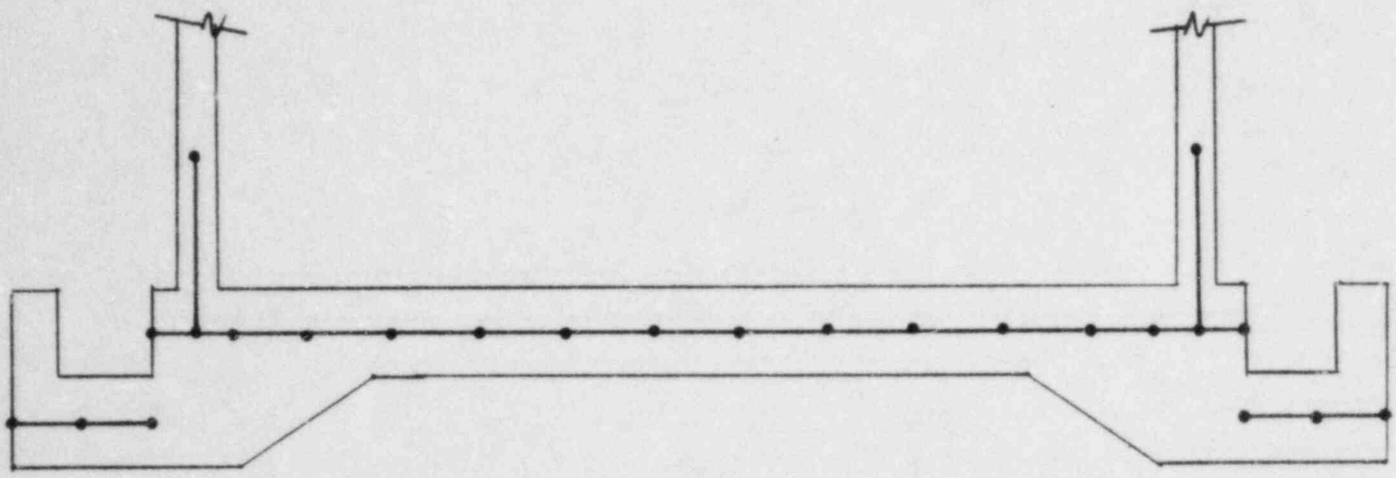



Figure 18
CONDENSATE STORAGE TANKS
BASEMAT FINITE ELEMENT MODEL
(Sheet 1 of 2)



SECTION LOOKING NORTH

Figure 18
CONDENSATE STORAGE TANKS
BASEMAT FINITE ELEMENT MODEL
(Sheet 2 of 2)

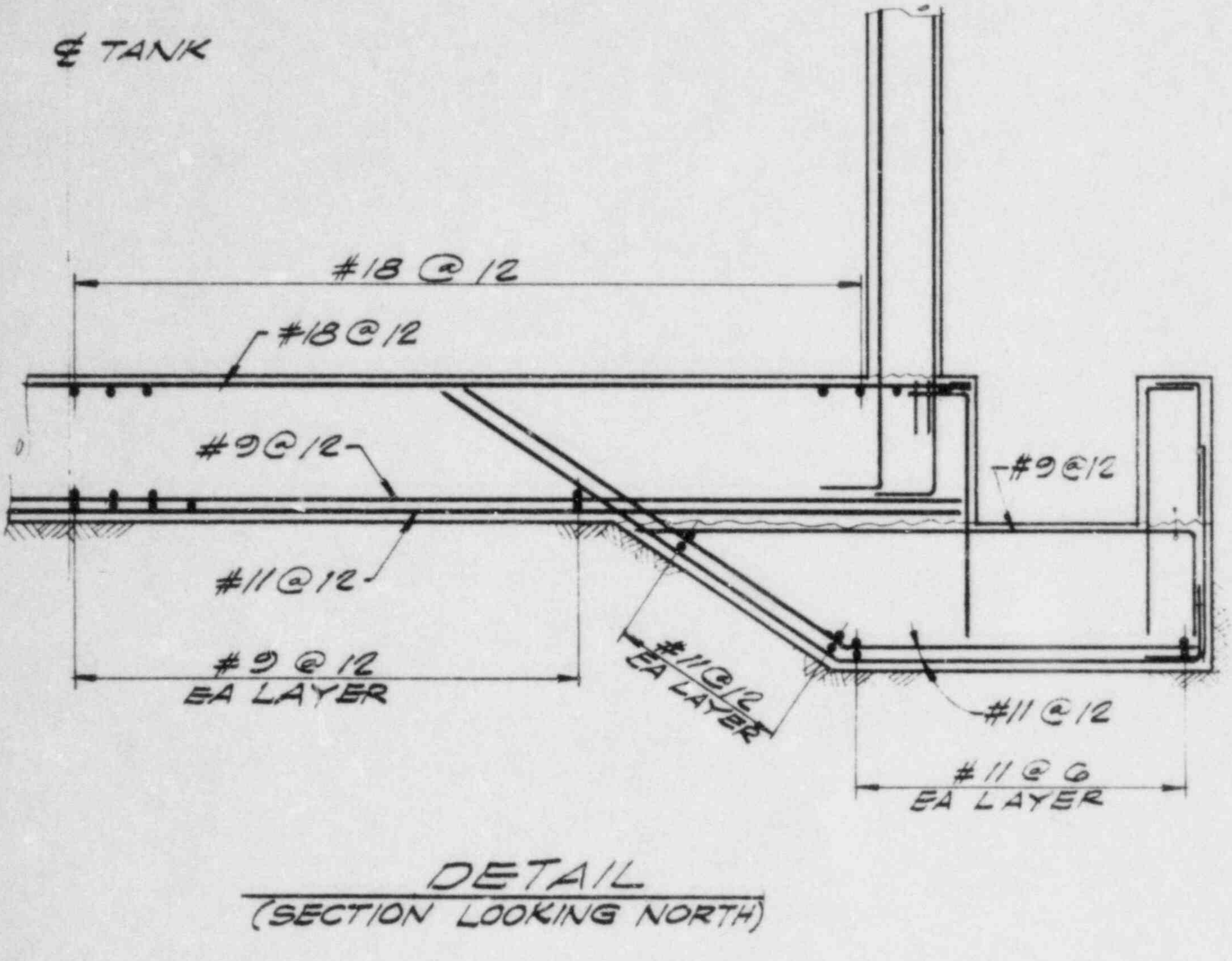



Figure 19
CONDENSATE STORAGE TANK
BASEMAT REINFORCING DETAILS

# 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 |

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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-$s^2$/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.

$$t_p = 1.25\ T_p \qquad (2\text{-}2)$$

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 General

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 ($\mu$) |
|---|---|---|
| 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.

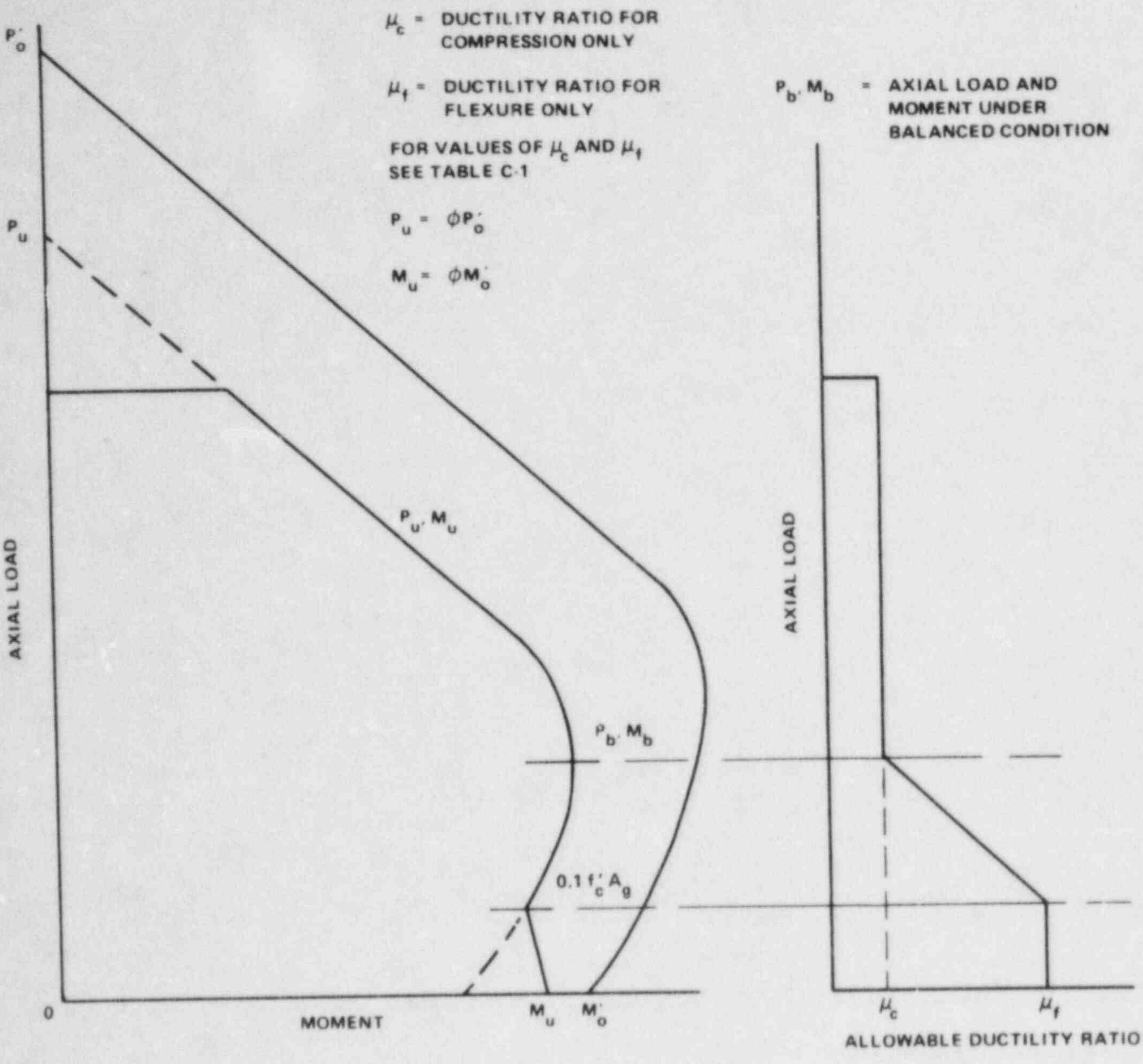
(3) Ductility ratio up to 10 can be used without an angular rotation check.

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

(5) $\ell/r$ 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 $\mu$ VS P

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