VOGTLE ELECTRIC GENERATING PLANT

GEORGIA POWER COMPANY

DIESEL FUEL OIL STORAGE TANK PUMPHOUSE

DESIGN REPORT

Prepared

by

Bechtel Power Corporation, Los Angeles, California

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PDR ADOCK 05000424
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VEGP-DIESEL FUEL OIL STORAGE TANK PUMPHOUSE
DESIGN REPORT

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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 diesel fuel oil storage tank pumphouse (DFOSTPH), 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 governing design forces.

2.0 DESCRIPTION OF STRUCTURE

2.1 GENERAL DESCRIPTION

The DFOSTPH (one for each unit) is a one-story reinforced concrete box type structure. It is substantially buried, with only the roof and access area projecting above grade. The purpose of this structure is to provide access to and workspace around the diesel fuel oil pumps mounted on the buried diesel fuel oil tanks. There are two train-oriented diesel fuel oil storage tanks per unit. Each diesel fuel oil storage tank pumphouse is divided into three compartments. Independent train-oriented compartments are provided for each tank with a common entry area between them.

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".

Each DFOSTPH is located in the Category 1 yard area near the east-west centerline of the plant. It is located approximately 11 feet 6 inches from the diesel generator building (see figures 1 and 2). There are no other structures adjacent to the DFOSTPH.

With the exception of the common entry area, the structure is substantially buried. It is supported on continuous wall footings 2 feet thick which are located approximately 9 feet below grade (see figure 3). The footings are founded on approximately 80 feet of Category 1 backfill placed on the marl bearing stratum. The DFOSTPH is located approximately 50 feet above the high water table.

2.3 GEOMETRY AND DIMENSIONS

The overall plan dimensions for each DFOSTPH are 118 feet by 30 feet. The height above the basemat is 10 feet 6 inches for the pumphouse compartments and 20 feet 6 inches for the entry area. Structure plans and sections are shown in figure 3.

2.4 KEY STRUCTURAL ELEMENTS

The DFOSTPH is analyzed and designed as a shear wall structure. The shear walls spanning the width of the structure are also analyzed and designed as deep beams. The key structural elements are the wall footings, the shear walls, and the roof diaphragms. All walls and roofs are 2 feet thick. The shear wall systems considered are shown in figures 4 and 5.

2.5 MAJOR EQUIPMENT

The DFOSTPH contains no major equipment.

2.6 SPECIAL FEATURES

Reinforced concrete hatches have been provided in the roof of each pumphouse compartment.

3.0 DESIGN BASES

3.1 CRITERIA

The following documents are applicable to the design of the diesel fuel oil storage tank pumphouse.

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 DFOSTPH is designed for all credible loading conditions. The loads are listed and load terms defined in Appendix A. The loads are further defined as follows.

3.2.1 Normal Loads

3.2.1.1 Dead Loads (D)

• Reinforced concrete	150 pcf
• Piping	50 psf applied to roof and slab at grade as applicable
• Steel framing (roof)	5 psf

3.2.1.2 Live Loads (L)

• Distributed snow load on roofs	30 psf
• Distributed load on roofs	150 psf
• Distributed load on interior slabs	50 psf
• Concentrated load on slabs	5k (applied to maximize moment and shear), to provide design margin for additional support and construction loads
• At-rest lateral soil pressure	$0.7\gamma_m H$ (refer to section 3.4.6)

3.2.1.3 Operating Thermal Loads (T_o)

Not applicable

3.2.1.4 Pipe Reactions (R_o)

There are no significant piping loads applicable to the diesel fuel oil storage tank pumphouse.

3.2.2 <u>Severe Enviromental 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 and roof levels are discussed in the Seismic Analysis Report. The horizontal and vertical in-structure OBE accelerations are provided in table 1.

The OBE damping values, as percentages of critical damping, applicable to the DFOSTPH are as follows:

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

Dynamic lateral earth pressures are developed by the Mononabe-Okabe method of analysis for dynamic earth pressures in dry cohesionless materials. The dynamic incremental soil pressure profile is shown in figure 6.

3.2.2.2 Design Wind (W)

The applicable wind load is the 100-year mean recurrence interval 110 mph wind based on ANSI A58.1-1972 (reference 1). Coefficients are per Exposure C, applicable to flat open country. The wind effective velocity pressure profile is shown in figure 7.

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.

Free-field response spectra and the development of horizontal and vertical floor accelerations and in-structure response spectra at the basemat and roof levels are discussed in the Seismic Analysis Report. The horizontal and vertical in-structure SSE accelerations are given in table 1. The SSE damping values, as a percentage of critical damping, applicable to the DFOSTPH are as follows:

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

Dynamic lateral earth pressures are developed by the Mononabe-Okabe method of analysis for dynamic pressures in dry cohesionless materials. The dynamic incremental soil pressure profile is shown in figure 6.

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 (shown in figure 7) is in accordance with reference 2.

The DFOSTPH is a partially vented structure. Conservatively, all walls and slabs are designed for a tornado pressure effect of ±3 psi.

Tornado loading (W_t) is defined as the worst case of the following combination 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)
$W_t = W_{tq} + 0.5\ W_{tp}$
$W_t = W_{tq} + W_{tm}$
$W_t = W_{tq} + 0.5\ W_{tp} + W_{tm}$

The DFOSTPH is also designed to withstand tornado missile impact effects from airborne objects transported by the tornado. The tornado missile parameters are listed in table 2. 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.

3.2.3.3 Probable Maximum Precipitation, PMP (N)

The load due to probable maximum precipitation is applied to the DFOSTPH entry section roof area.

Special roof scuppers are provided with sufficient capacity to ensure that the depth of ponding water due to the PMP rainfall on this portion of the roof does not exceed 18 inches. This results in an applied PMP load of 94 psf. The lower roof sections have no parapets and, therefore, appreciable ponding will not occur.

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 (P_a, T_a, R_a, Y_r, Y_i, Y_m)

There are no significant abnormal loads applicable to the DFOSTPH.

3.3 LOAD COMBINATIONS AND STRESS/STRENGTH LIMITS

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

3.4 MATERIALS

The following materials and material properties were used in the design of the DFOSTPH.

3.4.1 Concrete

- Compressive strength $f'_c = 4.0$ ksi
- Modulus of elasticity $E_c = 3,605$ ksi
- Shear modulus $G = 1440$ 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 stress $F_{ult} = 90$ ksi
- Minimum elongation 7-9% in 8 inches

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 Structual Bolts

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

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

3.4.5 Anchor Bolts and Headed Anchor Studs

3.4.5.1 ASTM A36

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

3.4.5.2 ASTM A108

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

3.4.5.3 ASTM A307

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

3.4.6 Foundation Media

The DFOSTPH is founded on Category 1 backfill. The design parameters of the Category 1 backfill are as follows:

3.4.6.1 General Description

See section 2.2

3.4.6.2 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, — ϕ = 34°
- Cohesion — C = O

3.4.6.3 Net Bearing Capacities

- Ultimate — 81.9 ksf
- Allowable static — 27.3 ksf
- Allowable dynamic — 41.0 ksf

4.0 STRUCTURAL ANALYSIS AND DESIGN

This section provides the methodologies employed to analyze the DFOSTPH 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 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 requirements are satisfied.

The structural analysis and design are primarily performed by manual calculations. The building structure is considered as an assemblage of slabs, beams, walls, and footings. The analysis is performed using standard structural analysis techniques. The analysis techniques, boundary conditions, and application of loads are provided to illustrate the methods of analysis. In addition, 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 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 DFOSTPH.

All other load combinations, including the effects of abnormal loads and tornado loads, are evaluated where applicable on a local area basis, i.e., section 5.2. 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 DFOSTPH consist of concrete roof slabs that support the applied vertical loads, the walls and deep beams that support the roof slabs, and the wall footings which transmit the loads from the walls to the foundation medium. Representative vertical load carrying elements are identified in figures 4 and 5.

The analysis of the building for vertical loads begins at the roof slab and proceeds down through the deep beams and walls to the wall footings. Slabs are analyzed for the vertical loads applied to them. The total vertical load on a wall or deep beam is computed based on its self weight and the vertical loads from the roof slab tributary areas.

4.3 LATERAL LOAD ANALYSIS

The lateral load carrying elements of the DFOSTPH consist of concrete roof slabs acting as rigid diaphragms, the shear walls which transmit the loads from the roof diaphragms to the wall footings, and the wall footings which transmit the loads from the walls to the foundation medium. Representative lateral load carrying elements are identified in figures 4 and 5.

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 statically. The design horizontal earthquake load (story shear load) at the roof level is obtained by multiplying the lumped roof story mass by the maximum roof acceleration. The design horizontal soil pressure components acting on the structure below grade are included in the lateral load analysis. The roof story shear load is distributed to the shear walls in proportion to their relative rigidities.

To account for the torsion caused by seismic wave propagation effects, the inherent building eccentricity between the center of mass and center of rigidity 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 roof story shear. 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 roof story shear (direct shear) and shear due to torsional moment (torsional shear) are combined 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 either 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 SLABS

4.5.1 Analysis and Design Methodology

A layout of the roof slab panels of the DFOSTPH is presented in figure 3. Based on the panel configuration, 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 formulae.

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

Slab panels are selected for design on the basis of the controlling combination of design load intensity, span, panel configuration and support condition. The structural design is primarily based on strength considerations and consists of sizing and detailing the reinforcing steel to meet the ACI 318 Code requirements. Design results are shown in table 3, and design details are presented in figure 8. 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 roof adjacent to large openings.

4.6 SHEAR WALLS

4.6.1 Analysis and Design Methodology

The location of shear walls are identified in figures 4 and 5.

The details of the analysis methodology used to compute the total in-plane design loads of a shear wall are described under lateral load analysis 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 are considered using the soil pressure loads on the exterior walls and the inertia loads on the walls due to the structural acceleration caused by the design earthquake. Soil pressure loads are applied as triangular and uniform pressure loads. The seismic inertia loads are applied as uniform pressure loads.

Conventional beam analysis is used to compute the bending moment and 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 3).

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 requirements for the out-of-plane loads are determined and combined with the requirements for the in-plane loads.

Uniformly distributed roof loads are converted to equivalent uniform loads using the tributary load method. The design vertical earthquake load for the deep beams is obtained by multiplying the tributary mass from the applied loading (including the wall's own mass) by the roof acceleration.

The east-west shear walls are also analyzed and designed as deep beams spanning between the north-south walls. The effective deep beam section selected is the continuous region of the wall, uninterrupted by openings. Conservative support boundary conditions are selected to maximize the internal design forces of the deep beam. The analysis and design are based on strength considerations. In general, additional tension steel is added to that required by the in-plane shear analysis.

Design results are shown in table 3, and design details are presented in figure 8.

4.7 WALL FOOTINGS

4.7.1 Analysis of Design Methodology

The magnitude and distribution of the soil reaction loads are derived by applying statics to the overall DFOSTPH structure, and summing equilibrium forces at the bottom of the wall footings. The result is a linearly varying soil reaction pressure profile. The wall footings are sized to limit the maximum soil pressure surcharge to the allowable values specified by the diesel fuel oil storage tank supplier.

The wall footings are analyzed by statically applying the soil reaction pressure profile. The walls behave as support points for the footings.

The footing is analyzed and designed as a cantilever beam extending perpendicular to the direction of the wall. The peak soil reaction intensity which occurs along the length of the footing is applied as a uniform load. The footing design is primarily based on strength requirements and consists of proportioning and detailing the reinforcing steel in accordance with the ACI 318 Code. Design results are shown in table 3, and design details are presented in figure 8.

5.0 MISCELLANEOUS ANALYSIS AND DESIGN

As described in section 4.1, the DFOSTPH is evaluated for the effects of tornado loads on a local area basis. In addition, the overall stability of the DFOSTPH is evaluated to ensure that an adequate factor of safety against instability is provided. This section describes these analyses.

5.1 STABILITY

Overall safety factors for stability are not calculated for the DFOSTPH as the structure is substantially buried and significant sliding or overturning cannot occur under design loading conditions. Also, since the foundation level (the lowest foundation elevation is elevation 209'-6") is above the high water table (elevation 165'-0"), the DFOSTPH is not subjected to flotation effects.

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. 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 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 4.

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 FOUNDATION BEARING PRESSURE

The maximum calculated bearing pressures under the governing design load conditions are provided in table 5.

6.0 CONCLUSION

The analysis and design of the diesel fuel oil storage tank pumphouse includes all credible loading conditions and complies with all applicable design requirements.

7.0 REFERENCES

1. "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.

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

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

TABLE 1

DIESEL FUEL OIL STORAGE TANK
PUMPHOUSE SEISMIC ACCELERATION VALUES(1)

Operating Basis Earthquake
Horizontal = 0.15g
Vertical = 0.15g
Safe Shutdown Earthquake
Horizontal = 0.25g
Vertical = 0.25g

(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 Projected Area)	4000	0	75	60

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

TABLE 3

DESIGN RESULTS

Item	Governing Load Combination Equation	Design Force		A_s Required ($in.^2$)	A_s Provided ($in.^2$)
		Mu (ft-k)	Vu (k)		
Wall 1	3 Appendix B	3198	376.2	0.36/ft	1.0 in./ft
Walls 1, 5 Deep girders	3 Appendix B	426	59	0.94	1.76
2	3 Appendix B	802	22/ft	1.34	1.76
See Fig. 4 3	3 Appendix B	1250	23.8/ft	2.09	3.52
4	3 Appendix B	1250	23.8/ft	2.09	2.09
Roof	3 Appendix B	60.5	9.58	0.82/ft E-W	1.00 /ft E-W
				0.44/ft N-S (1)	0.44 /ft N-S
Footings	3 Appendix B	10.3	6.1/ft	0.44/ft (1)	0.44/ft

(1) Governed by minimum code reinforcement requirements.

TABLE 4

TORNADO MISSILE ANALYSIS RESULTS[1]

Panel Description and Location	Panel Size Length (ft)	Width (ft)	Thickness (ft)	Computed Ductility	Allowable Ductility
Roof (center section)	25	24	2	5.5	10
Wall (center section)	25	12	2	2.0	10

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

TABLE 5

MAXIMUM FOUNDATION BEARING PRESSURES(1)

Gross Static (ksf)	Net Static (ksf)	Gross Dynamic (ksf)	Net Dynamic (ksf)	Allowable Net(2) Bearing Capacity		Computed(3) Factor of Safety	
				Static (ksf)	Dynamic (ksf)	Static	Dynamic
1.7	0.4	2.5	1.2	27.3	41.0	204.8	68.3

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

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

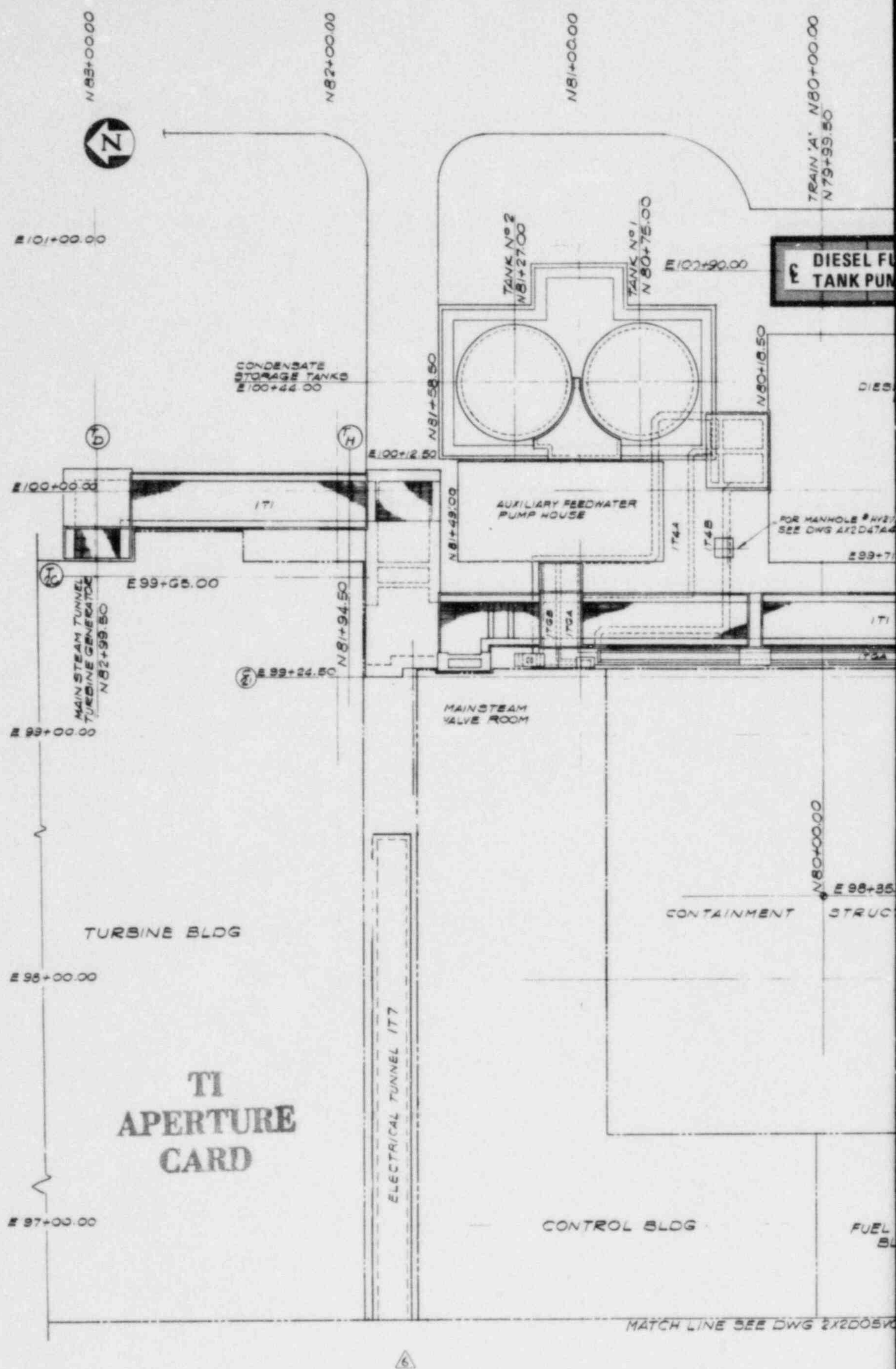
Net Static = The static pressure in excess of the overburden pressure at the base of the structure.

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.



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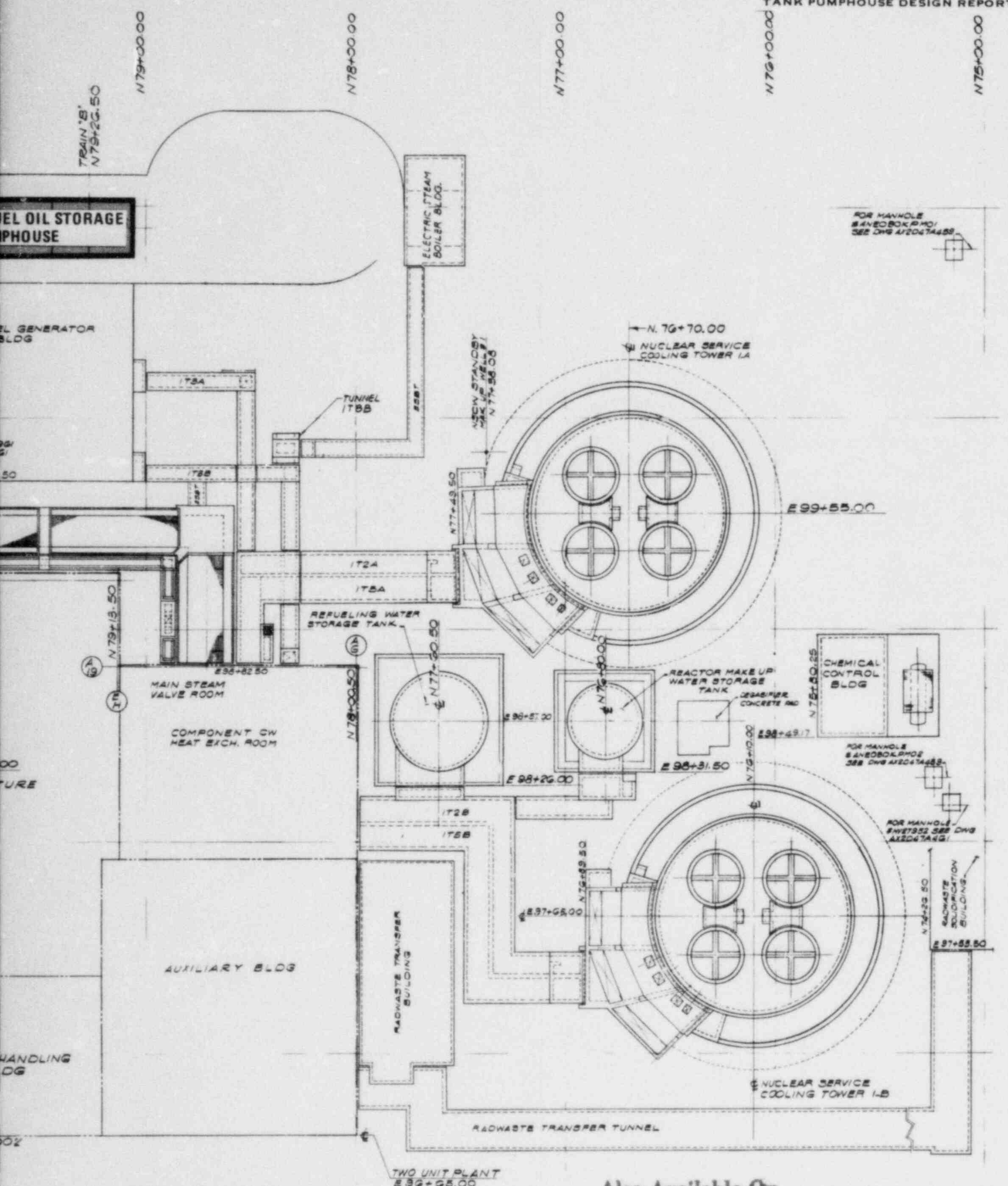
CARD



Also Available On
Aperture Card

Figure 1
LOCATION OF DIESEL FUEL OIL
STORAGE TANK PUMPHOUSE – UNIT 1

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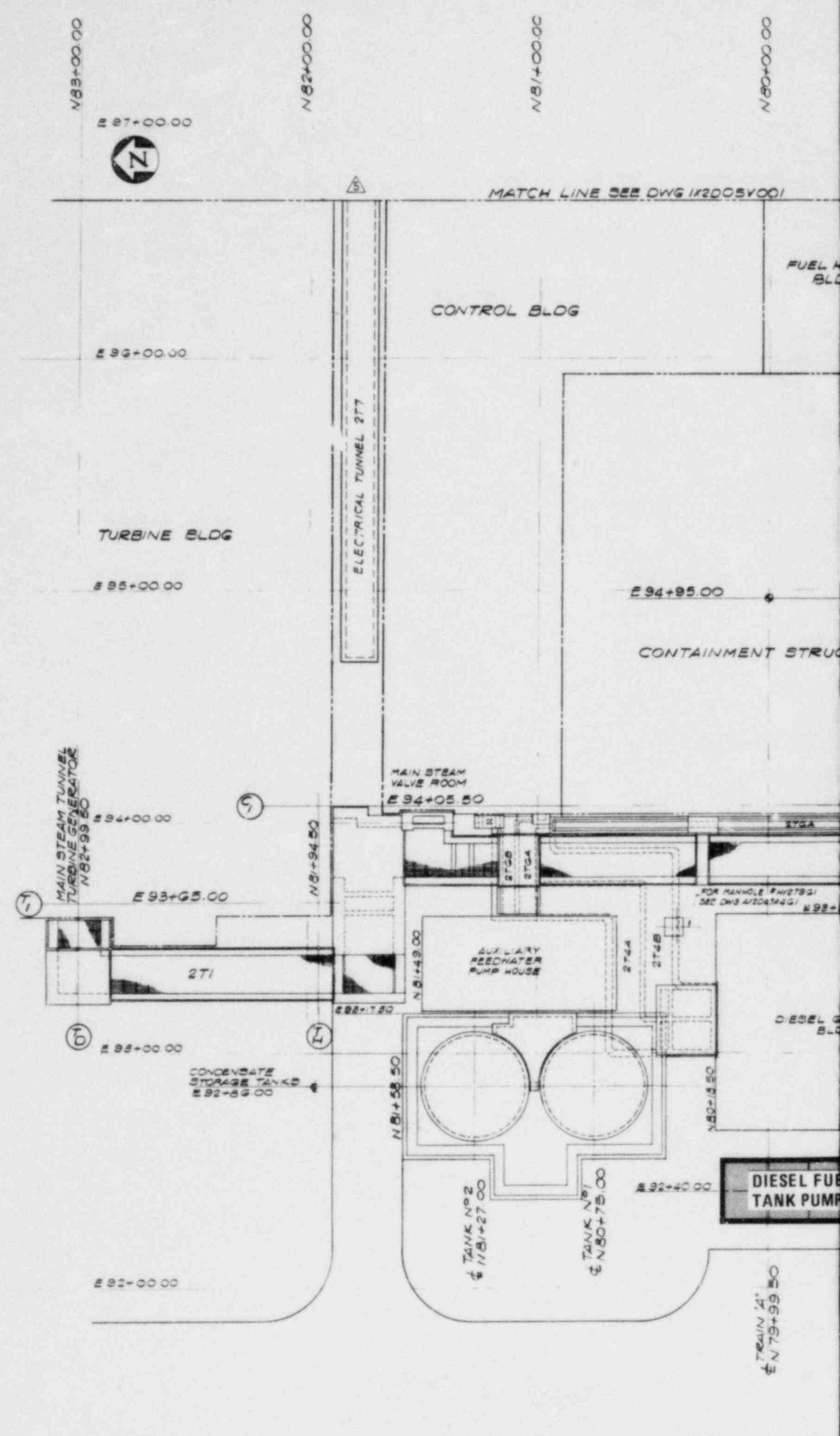
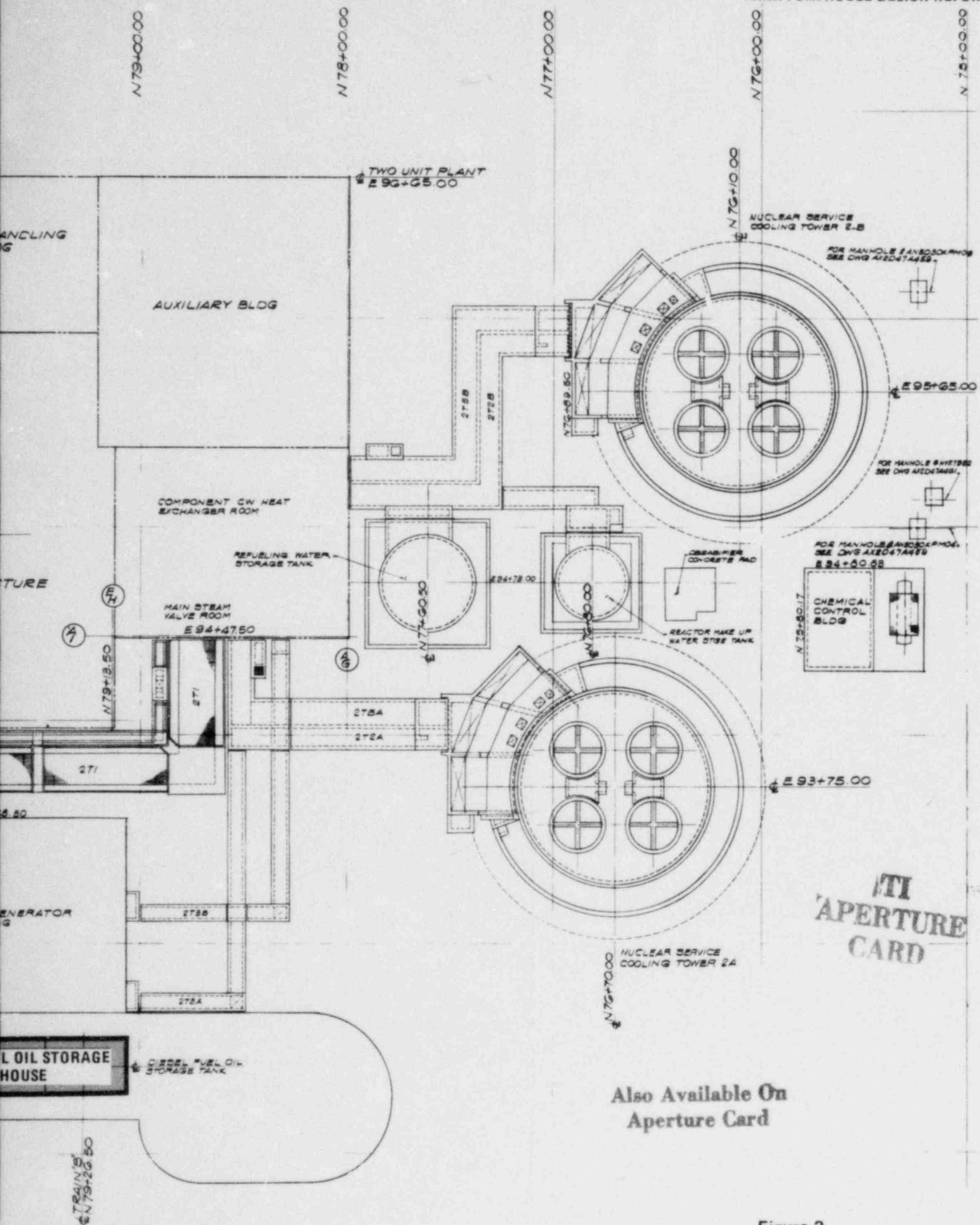

N83+00.00
N82+00.00
N81+00.00
N80+00.00
E 97+00.00
MATCH LINE SEE DWG 1X2D05V001
FUEL H
BLD
CONTROL BLDG
E 96+00.00
ELECTRICAL TUNNEL 2T7
TURBINE BLDG
E 95+00.00
E 94+95.00
CONTAINMENT STRUC
MAIN STEAM VALVE ROOM
E 94+05.50
MAIN STEAM TUNNEL TURBINE GENERATOR N82+99.50
E 94+00.00
N81+94.50
E 93+65.00
2T6B
2T6A
2T6A
FOR MANHOLE
2T4A
2T4B
AUXILIARY FEEDWATER PUMP HOUSE
N81+49.00
2T1
E 93+17.50
E 93+00.00
DIESEL G
BLD
CONDENSATE STORAGE TANKS E 92+66.00
N81+58.50
N80+18.50
E 92+40.00
DIESEL FUE
TANK PUMP
℄ TANK N°2 N81+27.00
℄ TANK N°1 N80+75.00
E 92+00.00
℄ TRAIN 'A' ℄ N 79+99.50



APERTURE CARD

Also Available On
Aperture Card

Figure 2
LOCATION OF DIESEL FUEL OIL
STORAGE TANK PUMPHOUSE – UNIT 2

8411050212-02

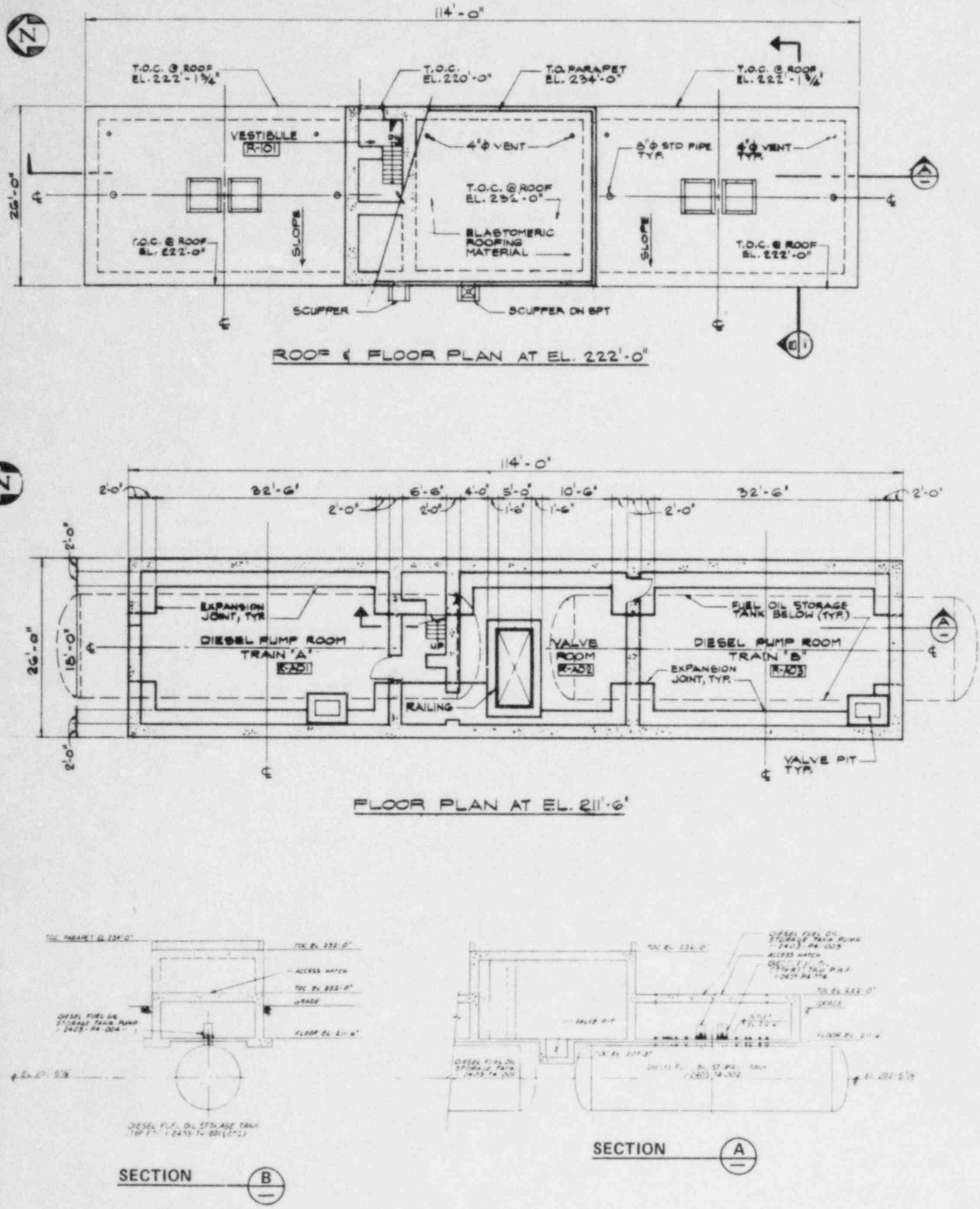
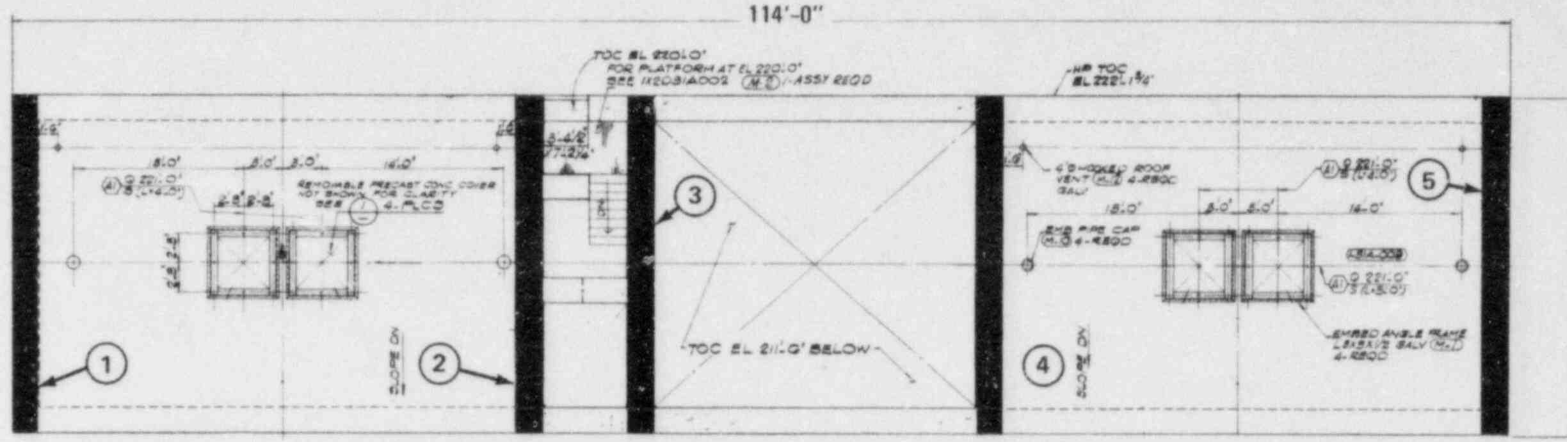


Figure 3
DIESEL FUEL OIL STORAGE TANK
PUMPHOUSE PLAN AND SECTIONS

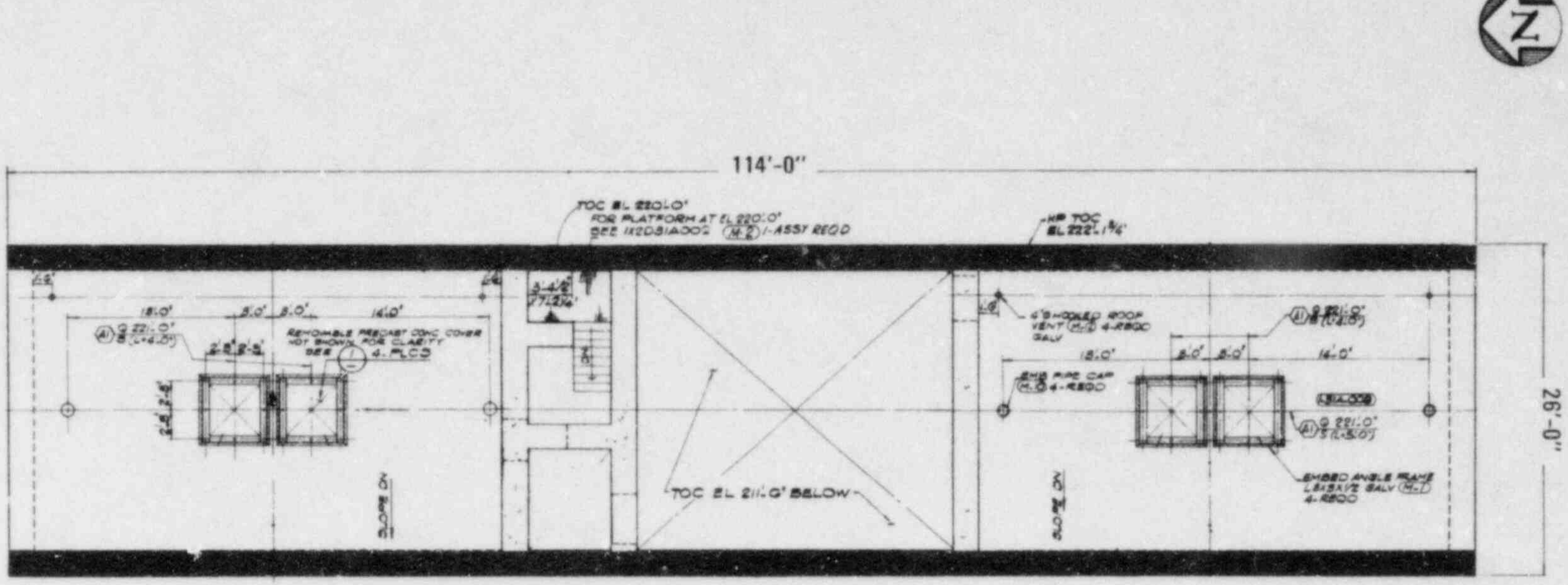


LEGEND

= SHEAR WALLS

= WALL NUMBER

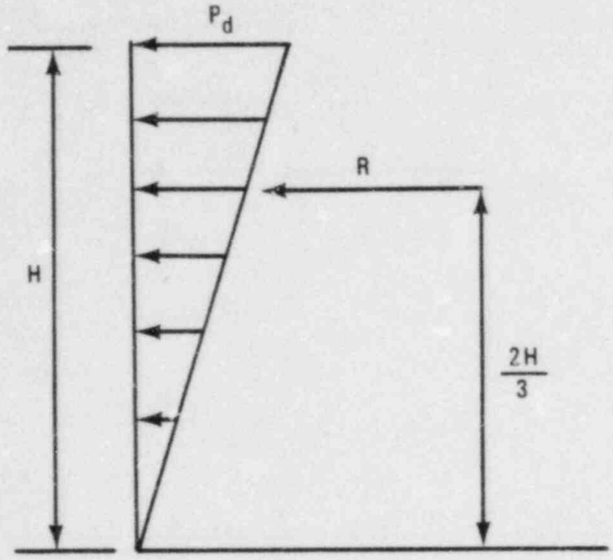
Figure 4
LOCATION OF EAST-WEST SHEAR WALLS



LEGEND

= SHEAR WALLS

Figure 5
LOCATION OF NORTH-SOUTH SHEAR WALLS



H: HEIGHT FROM BASE OF STRUCTURE TO SOIL SURFACE

P_d = DYNAMIC INCREMENTAL SOIL PRESSURE

R: RESULTANT FORCE

$R = .075\, \gamma_m H^2$ (SEE)*

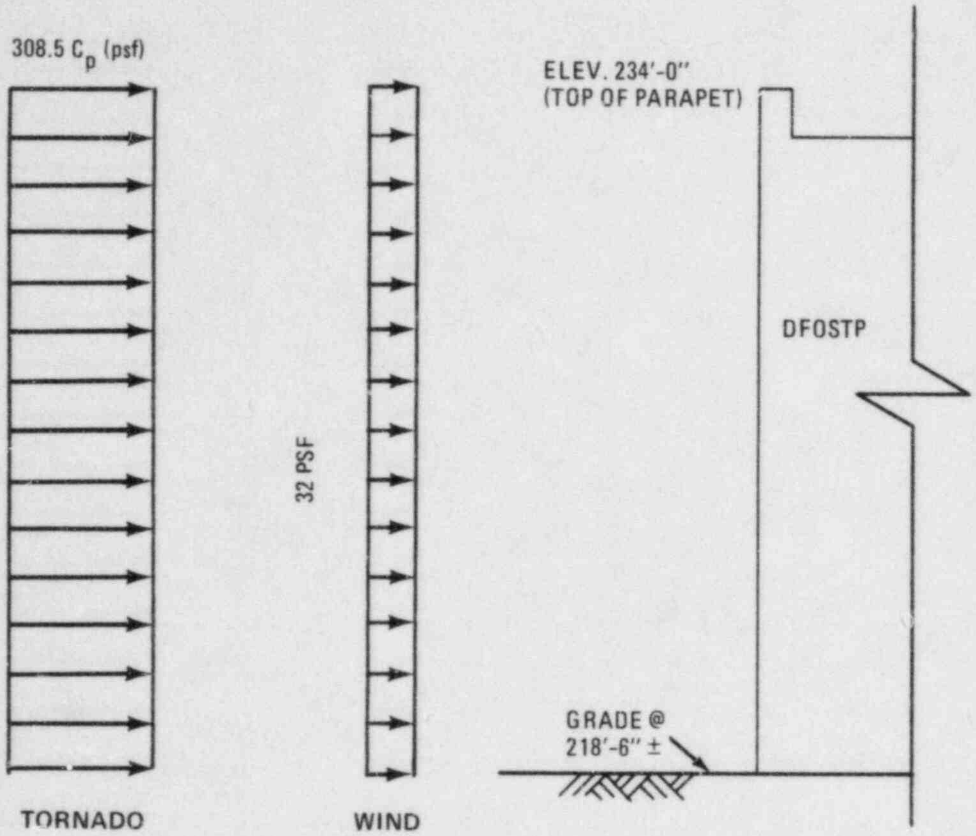
$.045\, \gamma_m H^2$ (OBE)*

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

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 6
DYNAMIC INCREMENTAL
SOIL PRESSURE PROFILE



$P = C_s P_{max} C_p$

WHERE:

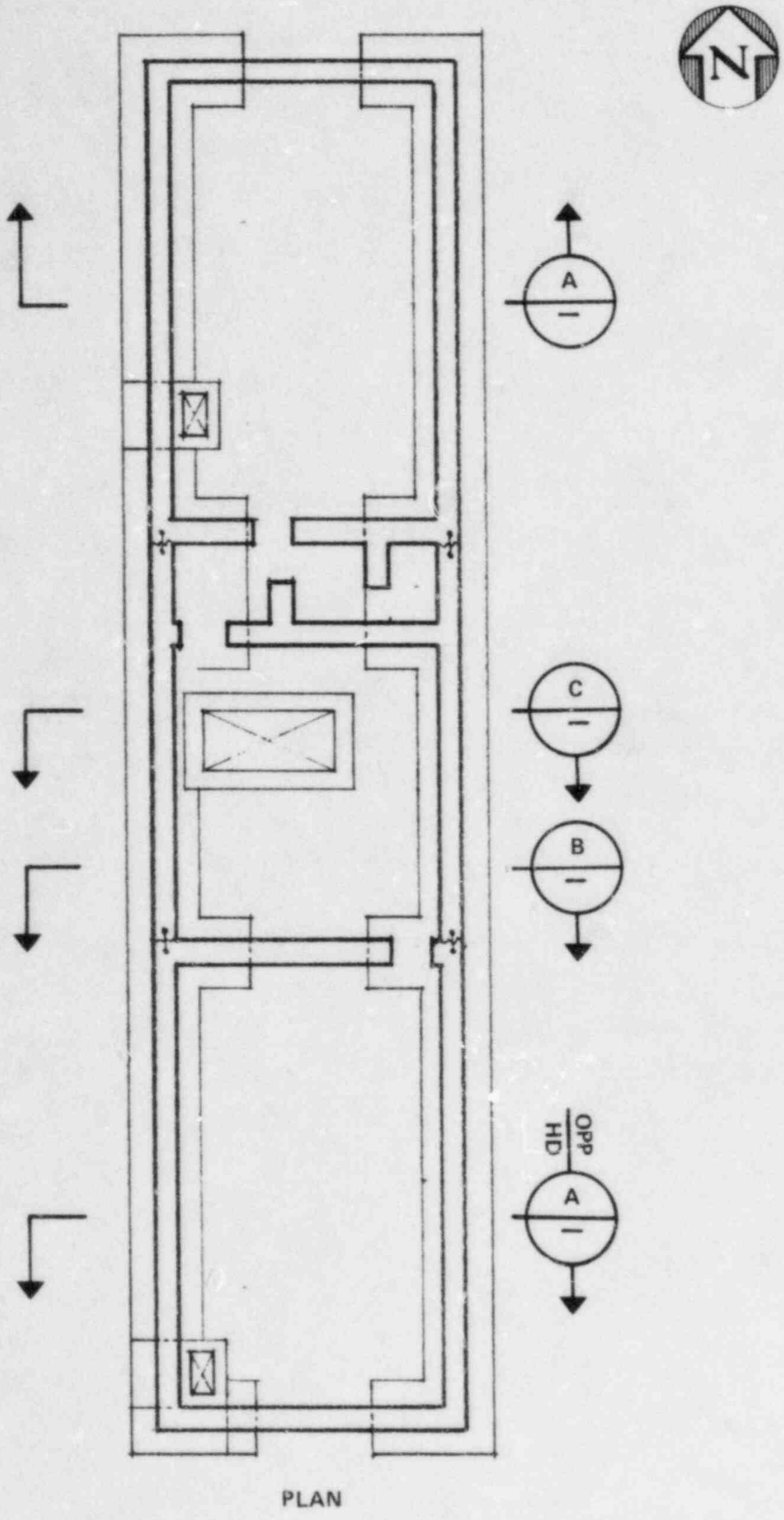
C_s = SIZE COEFFICIENT
= .93

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

C_p = EFFECTIVE EXTERNAL PRESSURE COEFFICIENT

$P = (.93)\ (332\ \text{psf})\ C_p$
$= 308.5\ C_p$ (psf)

Figure 7
WIND AND TORNADO EFFECTIVE VELOCITY
PRESSURE PROFILES



PLAN

Figure 8
REINFORCING DETAILS
(Sheet 1 of 4)

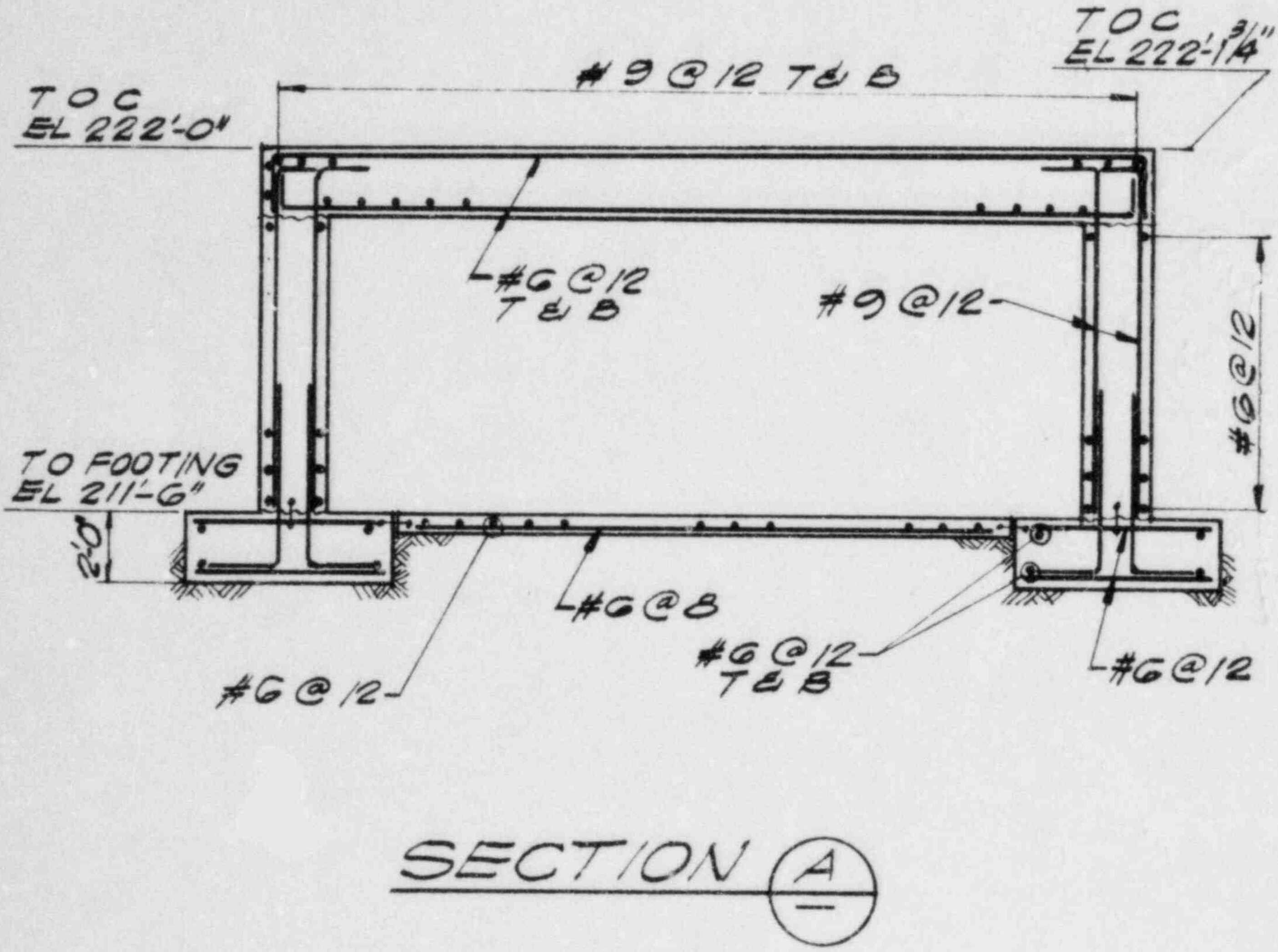


Figure 8
REINFORCING DETAILS
(Sheet 2 of 4)

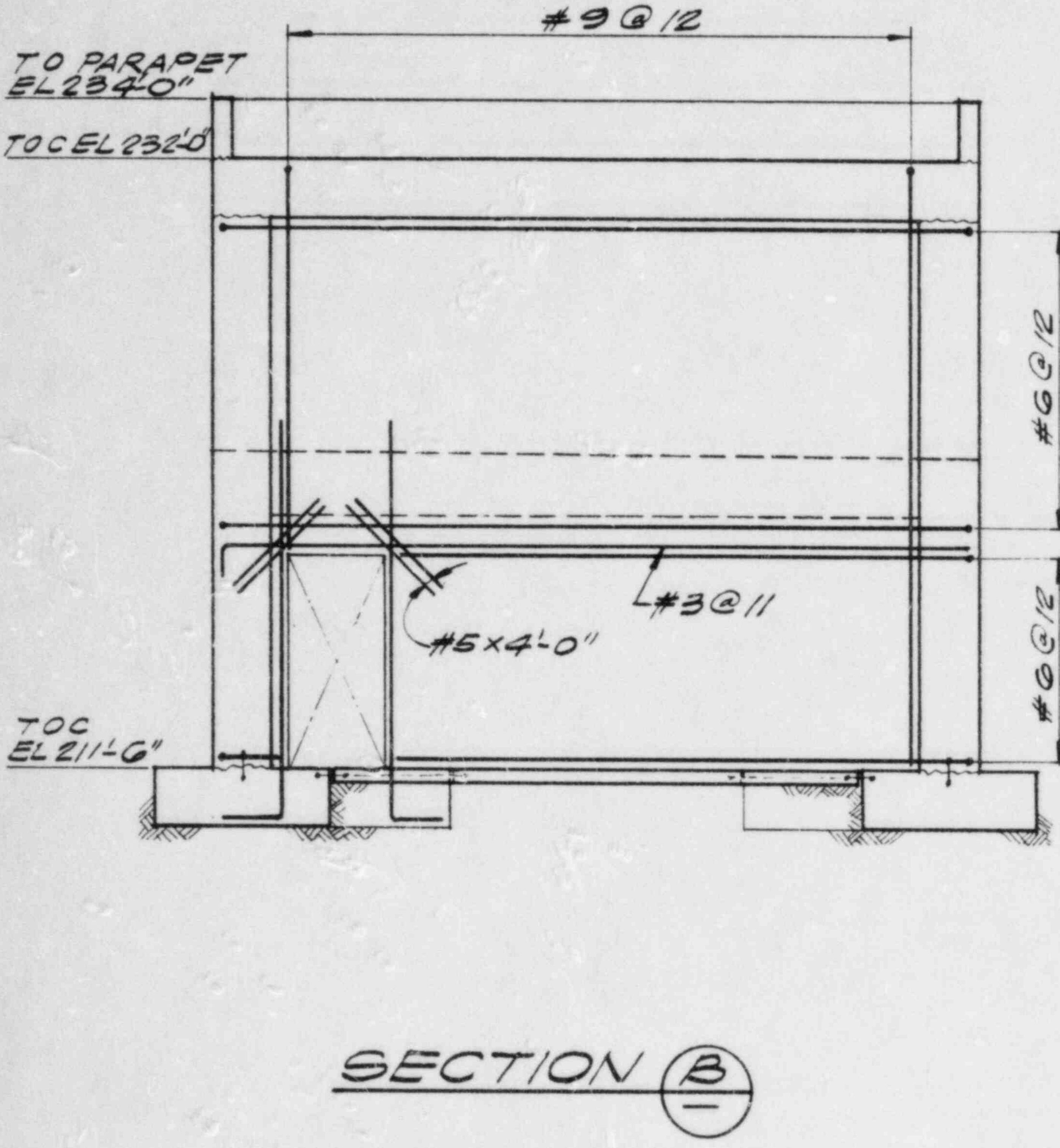


Figure 8
REINFORCING DETAILS
(Sheet 3 of 4)

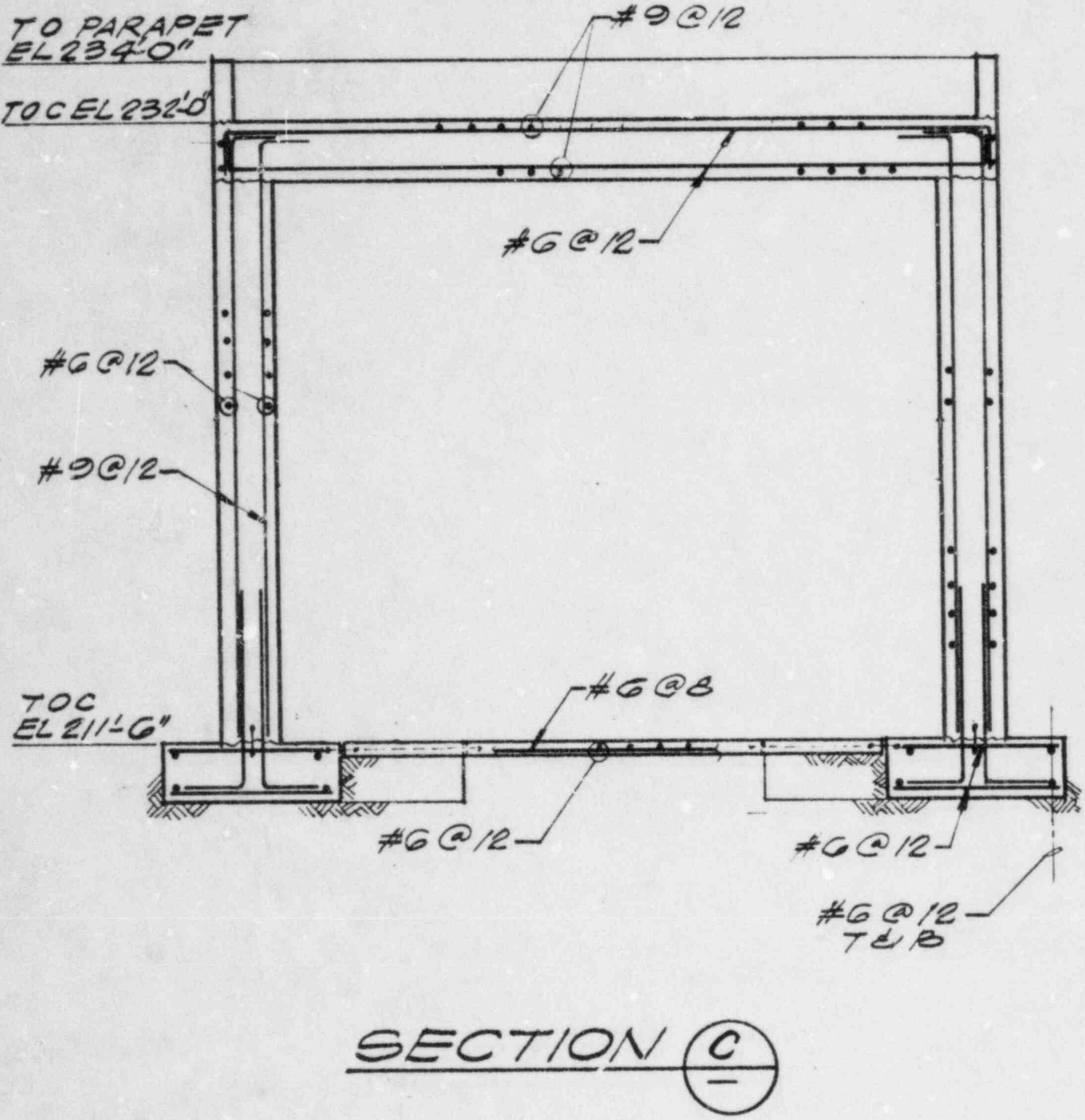


Figure 8
REINFORCING DETAILS
(Sheet 4 of 4)

VEGP-DIESEL FUEL OIL STORAGE TANK PUMPHOUSE
DESIGN REPORT

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-s^2/ft).

V_s = missile striking velocity (ft/s).

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

---

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," CRNL 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.

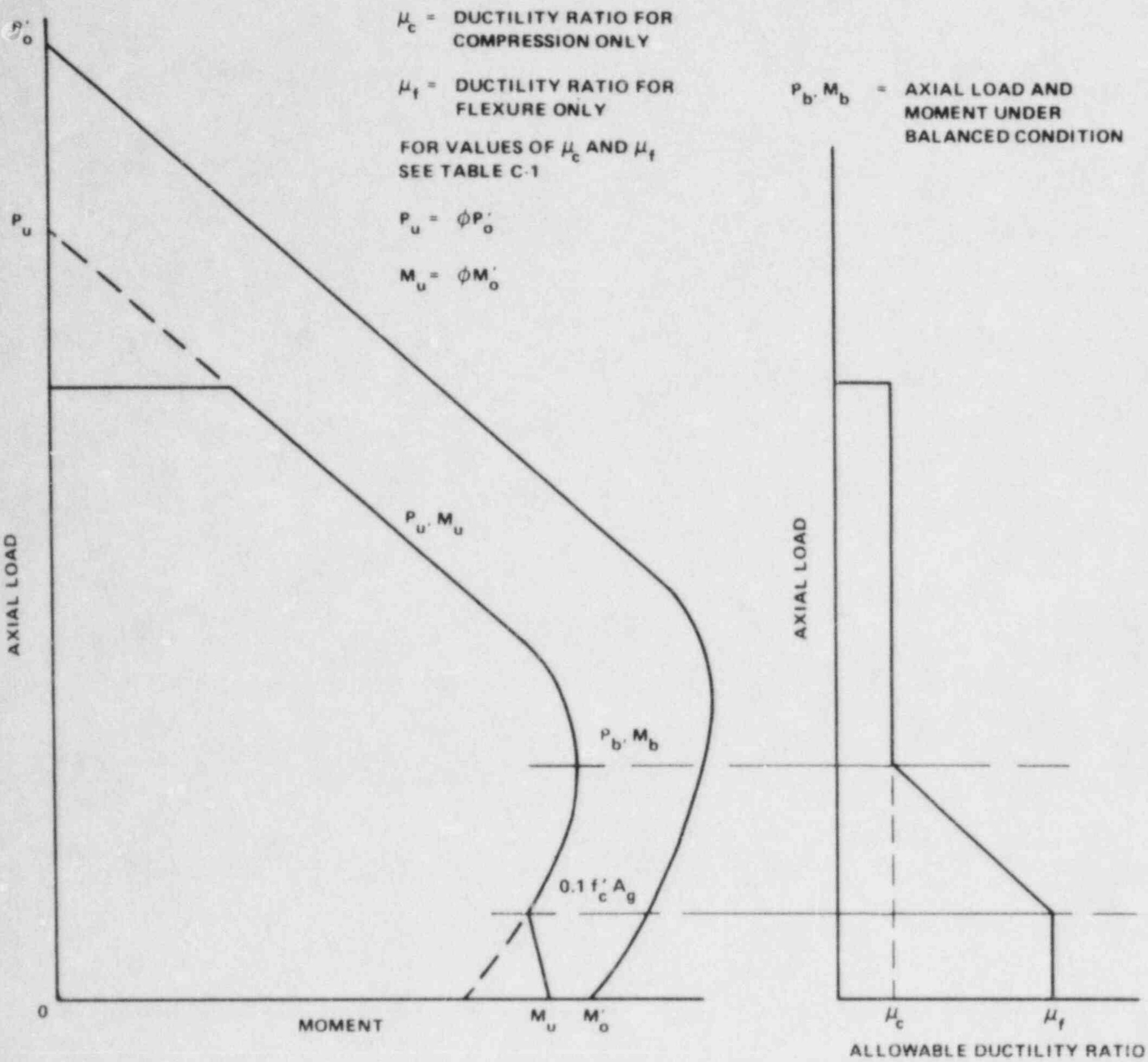
(3) Ductility ratio up to 10 can be used without an angular rotation check.

(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