VOGTLE ELECTRIC GENERATING PLANT

GEORGIA POWER COMPANY

NSCW TOWER

AND

VALVE HOUSE

DESIGN REPORT

Prepared

by

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DESIGN REPORT

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1.0 INTRODUCTION

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

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

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

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

The purpose of this design report is to provide the Nuclear Regulatory Commission with specific design and construction information for the nuclear service cooling water (NSCW) towers and valve houses, 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 structures and their 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

The NSCW structures house components of the NSCW system whose primary function is to provide cooling water to safety-related equipment, and to transfer the heat loads to the atmosphere and to the storage basin of the NSCW tower which serves as the ultimate heat sink. As shown in figure 1, there are four NSCW towers and valve houses (two each per unit) labeled 1A, 1B, 2A, and 2B. For a general arrangement drawing of a typical tower and valve house, see figure 2.

Each NSCW tower consists of a mechanical draft cooling tower superstructure and a subterranean storage basin which contains the ultimate heat sink water. The primary function of the superstructure is to support and protect the components necessary to cool the incoming hot water and to minimize vapor loss to the atmosphere. The superstructure is divided into four functionally identical cells, each of which contains the necessary system components to operate independently. The basin functions as a cooling water storage supply and is sized to provide an emergency water supply for each reactor unit for shutdown and cooldown under the worst meteorological conditions with no makeup water supply.

Each NSCW tower has a corresponding valve house which adjoins the tower on the north side and serves as a transition structure which protects the piping, valves, and electrical supply as they exit from the NSCW tunnel below grade and disperse into the tower just above grade. To protect these items as they traverse outside the valve house across the 12-foot distance to the tower, a series of tornado missile protection concrete barriers extend from the valve house over the top of this area.

All of the NSCW towers and valve houses are constructed of reinforced concrete. The layout of the 1A and 2A structures is identical to the 1B and 2B structures, respectively; however, the configuration of the Unit 2 structures is opposite hand of that of Unit 1. Each NSCW tower and corresponding valve house are separated by a 5-1/2-inch seismic gap.

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 NSCW structures are located on the south side of the plant within the main power block area as shown in figure 1. The towers are embedded (approximately 90 feet) in Category 1 backfill with the bottom of the storage basin being 37 feet below the design high groundwater table which is located at elevation 165'-0". The base of the tower, which consists of a thick concrete mat foundation, is founded directly on the marl bearing stratum. The NSCW valve houses are partially embedded in the Category 1 backfill (approximately 20 feet) with the base founded directly on Category 1 backfill well above the groundwater table. The valve house base is also a reinforced concrete mat foundation.

The finished grade elevation around the NSCW structures varies slightly to facilitate drainage, but for design purposes it can be considered to be level with an average elevation of 218'-6".

2.3 GEOMETRY AND DIMENSIONS

2.3.1 NSCW Towers

The NSCW towers are cylindrical in shape (88 feet inside diameter) with a flat roof and basemat. The storage basin shell wall is 3 feet thick with the portion below elevation 155'-5 1/2" uniformly thickened to 5 feet. There is also a portion of the shell wall below grade on one side of the basin locally thickened, as shown in figure 3, to 13 feet 10-1/2 inches. The basemat of the tower consists of a 100-foot-diameter, 9-foot-thick mat foundation with a bottom elevation of 128'-0".

The cooling tower superstructure rises 33 feet above grade level to the top of the roof. The 2-foot-thick roof, also called the fan deck, supports the cooling tower fan stacks which rise an additional 13-1/2 feet into the air to an elevation of 264'-5". Air intake for the cooling tower is at grade level through large rectangular openings in the shell wall. The 12-foot-high openings have an average width of 8 feet and are uniformly spaced around the perimeter of the tower except near the valve house where the spacing and width vary slightly to accommodate the routing of piping.

Inside the superstructure, two perpendicular separation walls, 2 feet 3 inches thick, called the crosswalls, symmetrically divide the cylindrical tower into four separate cells. These walls extend from elevation 209'-9" (about 9 feet below grade) up to the roof, and span across the basin to opposite sides of the exterior shell wall. Covering each of the four internal cells is the fan deck roof slab. A large circular opening (approximately 25 feet in diameter) has been provided in the slab in each quadrant for air discharge. Surrounding each of these openings is a concrete fan stack. Each of the fan stacks is approximately hyperbolic in shape to enhance fan and airflow performance, and the concrete thickness varies slightly with an average thickness of about 2 feet (see figure 2).

Within the tower superstructure are several levels of concrete beams which support equipment, piping, and other apparatus. Immediately above the air intake openings lies the fill level. Here, a grid of five main load carrying beams (5 feet 5 inches deep by 1 foot 2 inches wide) running north-south, and a series of smaller perpendicular lateral support beams (3 feet 11 inches deep by 8 inches wide), enclose the fill material in each cell. Directly above the fill level and below the fan deck is the eliminator level. Here, a beam grid identical to that at the fill level, but with smaller beam sizes (3 feet 7 inches deep by 1 foot 2 inches wide main beams and 2 feet 10-1/2 inches deep by 8 inches wide lateral support beams), is used to support the drift eliminators. Both the fill and eliminator beam grids span across each of the four cells of the tower superstructure to the exterior shell wall and crosswalls. At the fan deck level, a set of fan deck beams (4 feet 9 inches deep by 2 feet wide) span mutually perpendicular across the circular air discharge openings and become monolithic with the fan deck slab, and are continuous at their ends with the exterior shell wall and crosswalls.

2.3.2 NSCW Valve Houses

The NSCW valve house is an irregularly shaped reinforced concrete structure whose roof is approximately 14 feet above grade, and whose basemat is 13-1/2 feet below grade to match that of the NSCW tunnel. In plan view, the 2-foot-thick valve house wall next to the tower is contoured to follow the circular outline of the tower. The other walls of the valve house (also 2 feet thick) are rectilinear with the northern walls angling obliquely. The walls rest on a 6-foot-thick mat foundation (bottom elevation of 198'-7"). The basemat extends 12 feet beyond the exterior side walls and the back walls (the northern walls farthest away from the NSCW tower). Within the valve house, a mezzanine at elevation 218'-6" is situated along the curved wall next to the tower. Extending from the roof of the valve house are the tornado missile protection shields which rise an additional 16 feet above the roof to an elevation just below the fan deck of the tower. There are a total of three 2-foot-thick shield slabs which are arranged to act in conjunction with the valve house roof to prevent direct missile strikes on the Category 1 items between the tower and the valve house at grade, and yet at the same time allow for proper air flow for intake into the tower.

2.4 KEY STRUCTURAL ELEMENTS

2.4.1 NSCW Towers

Being a cylindrical structure, the shell is the main structural element of the NSCW tower. It functions as the container for the storage basin water, the prime vertical load carrying element, and the key lateral load resisting element. The vertical continuity of the cylindrical shell wall is interrupted by the location of the air intake openings which are uniformly distributed around the circumference, resulting in an upper and lower shell structure that is joined together structurally by the columns (typically 6 feet wide by 3 feet thick) that are formed between adjacent openings. These columns provide direct vertical support for the upper shell wall and are also a part of the lateral load resisting system. The crosswalls also resist lateral loads by means of shear wall action. Structurally, they also function as deep beams for vertical support of the fan deck and all of the internal appurtenances.

The fan deck slab and fan support beams function as vertical support for the fan stacks and equipment located at this level. Inside the tower, the fill and eliminator beams also provide support for major equipment. Besides carrying gravity loads, they function as stiffening elements for the circular shell wall and, as such, are a part of the lateral load resisting system. There are a total of ten north-south main load carrying beams and four grid lines of smaller east-west lateral support beams for each level.

In addition, there is a locally thickened zone of the shell wall whose center lies approximately 40 degrees from the north direction on one side of the basin shell wall and extends from grade level down to the basemat. The concrete in this thickened zone supports and encases the pumping equipment required for circulation of the storage basin water. There is one thickened zone per tower, and it is an integral part of the shell wall below grade.

See figure 3 for the location of the key structural elements.

2.4.2 NSCW Valve Houses

Being somewhat boxlike in shape, the valve house is basically a shear wall structure. However, frame action also plays a part in the lateral load resisting system since pilasters are provided in the north and south exterior walls below the vertical wall supports for the missile shield slabs above the roof. These pilasters act as columns in conjunction with the missile shield interior wall supports which act as stiff beams to form two single-bay frames. The walls and pilasters also support vertical loads which are carried down to the concrete mat foundation where the loads are distributed to the soil. The extensions on the basemat beyond the exterior walls are provided to stabilize the structure against overturning in the event of significant lateral loading. As mentioned earlier, the series of concrete barriers which extend from the valve house roof function to protect Category 1 items from direct tornado missile strikes.

See figure 3 for the location of the key structural elements.

2.5 MAJOR EQUIPMENT

2.5.1 NSCW Towers

The NSCW tower's structure houses all of the necessary equipment and appurtenances to cool the NSCW system's water and to reduce vapor plumage. The cooling water is removed from the storage basin by the NSCW and transfer pumps. These pumps are located outside of the tower at grade on top of the thickened zone of the shell wall. This thickened zone serves as a protective concrete casing for the pump shafts and impellers.

The incoming water is cooled within the superstructure by direct contact with the ambient air which enters the tower at grade through the large rectangular air intake openings. The air flow through the tower is mechanically induced by four 22-foot-diameter fans located within the concrete fan stacks. The fans are driven by motors, housed and protected against tornado missiles by two, small, boxlike concrete compartments located between the fan stacks.

The heat transfer from the water to the air takes place at the fill level which consists of an assemblage of corrugated sheets of asbestos cement board. These fill bundles span between all of the concrete fill beams and thus cover the plan area of each cell. The returning NSCW system coolant is distributed uniformly over the fill by the spray system manifolds which rest directly on top of the main fill beams.

It is the function of the drift eliminators located immediately above the fill level to reduce water loss from the NSCW system. The eliminator blades are made of lightweight asbestos cement board assemblies which span between all of the concrete eliminator beams in an arrangement similar to the fill level.

See figure 3 for the location of the major NSCW tower equipment.

2.5.2 NSCW Valve Houses

There is no major equipment in the valve house other than the piping, valves, and electrical supply which passes through the valve house from the tunnel and into the tower.

2.6 SPECIAL FEATURES

In order to reduce coolant loss from the system at the falling water zone, a concrete splash ring surrounds the tower at grade adjacent to the air intake openings. The splash ring consists of a 12-foot-wide pad which slopes gently in toward the tower, a 2-foot-thick wall at the outside edge of the pad which rises 2 feet 3 inches above the top of the air intake openings, and a 10-foot-wide footing which functions to stabilize the wall. The splash ring forms an air inlet labyrinth which minimizes water loss by preventing direct lateral wind gusts from entering the openings which could cause significant splashing. If any splashing out through the openings does occur, the water will drain off the splash wall and splash pad back into the storage basin. The splash ring is actually a separate structure detached from the tower by a 1/2-inch gap filled with elastomeric joint sealant. The splash ring's continuity around the tower is interrupted by the thickened zone of the shell wall and the circular wall of the valve house. Each of these other elements functionally serves as a replacement for the splash ring section which it interrupts. The valve house is separated from the splash ring (as well as the tunnel and thickened portion of the tower shell wall) by a 5-1/2-inch seismic gap.

3.0 DESIGN BASES

3.1 CRITERIA

The design of the NSCW towers and valve houses is in accordance with the applicable sections of the following documents.

3.1.1 Codes and Specifications

- 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 those issued by the American Society for Testing and Materials (ASTM), the American Concrete Institute, and the American Iron and Steel Institute (AISI), are used to specify material properties, testing procedures, fabrication, and construction methods.

3.2 LOADS

The NSCW towers and valve houses are designed for all credible loading conditions. The loads considered credible are listed and defined in Appendix A and are further discussed below.

3.2.1 Normal Loads

3.2.1.1 Dead Loads (D)

Generally, the dead loads include the weight of all concrete elements, miscellaneous steel, major piping, and permanent equipment. Static vertical earth pressures on the valve house basemat extensions and hydrostatic pressures of the water in the storage basin are also considered dead loads.

The NSCW pump and transfer pump (see figure 3) weigh 27,600 pounds and 7,210 pounds, respectively. The equipment weight at the fan deck level of the tower is identified in figure 4. The wet weight of the fill material is 60 psf of plan area, and the wet weight of the drift eliminators is 12 psf. The spray system piping weight is 100 plf for each of the main north-south supporting beams. To account for miscellaneous piping and small equipment, a 50-100 psf load is considered where applicable.

3.2.1.2 Live Loads (L)

Design live loads include floor occupancy loads, platform and roof live loads, normal operating equipment reactions (excluding gravity loads), static lateral earth pressures (including ground water and surcharge load effects), and precipitation loads. A 30 psf design live load applied to the roofs of the structures envelops the effects of occupancy, snow, and 100-year rainwater ponding loads. A 100 psf live load, which is applied to floor areas and platforms, represents the effects of occupancy, movable equipment loads, and loads temporarily supported by the structure during maintenance. The equipment reactions considered at the fan deck level are identified in figure 4. The distribution of the static lateral earth pressures on the tower and valve house are shown in figures 10 and 11.

The surcharge loads on the soil surrounding the tower consist of a 250 psf design live load, and the surcharge effects of the nearby buildings. The static surcharge loads from nearby buildings are summarized in figure 5. All of the building surcharge loads shown are considered in the NSCW tower analysis, while only those from the reactor makeup and refueling water storage tanks are considered significant for the valve house design.

3.2.1.3 Operating Thermal Loads (T_o)

The NSCW towers are subject to various thermal profiles depending on the outside ambient conditions and tower operating conditions. The outside air temperature range of a minimum of 17°F to a maximum of 120°F is considered. The soil temperature is assumed to remain constant at elevation 164'-0" and below, and to vary linearly from the outside air temperature at grade down to 60°F at elevation 164'-0". The design operating temperatures corresponding to the extreme outside air temperatures are summarized in figure 6.

There is no significant variation in temperature across the NSCW valve house exterior walls and roof, and therefore the effect on the structure is neglected.

3.2.1.4 Operating Pipe Reactions (R_o)

The local effects of individual normal pipe reactions at anchor points on the structure are considered as applicable.

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 structure accelerations and in-structure response spectra at selected elevations of the structures are discussed in the Seismic Analysis Report. Tables 1 and 2 provide the OBE horizontal and vertical structure accelerations, respectively, for the NSCW tower and the valve house. The basic input horizontal and vertical OBE design spectra curves used for the valve house analysis are shown in figure 7.

Loads due to the OBE include structure inertia loads, seismic induced piping and equipment reactions, hydrodynamic effects of the water in the storage basin, and incremental dynamic lateral soil pressures on the buried walls.

The OBE damping values, as percentages of critical, applicable to the tower and valve house design are as follows:

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

The hydrodynamic effects of the storage basin water are determined based on Housner's method as given in Chapter 6 and Appendix F of reference 1. The vertical distribution of the hydrodynamic pressure on the storage basin wall is given in figure 10.

Dynamic incremental lateral soil pressures are based upon the Mononobe-Okabe analysis of dynamic pressures in dry cohesionless materials with simplifications for the active condition as derived by Seed and Whitman in reference 2. The peak free-field horizontal and vertical ground acceleration of 0.12g is used as the basis for calculating the magnitude of these dynamic pressures. The vertical distribution of the dynamic incremental pressure on the tower and valve house walls is given in figures 10 and 11.

The dynamic effects of the building surcharge loads as they affect the lateral soil pressures are considered also. Soil bearing pressure diagrams for the nearby buildings for the OBE seismic case are given in figure 5.

3.2.2.2 Design Wind (W)

The design wind effective velocity pressure profiles for the NSCW tower and valve house are given in figure 9. They correspond to a design wind velocity of 110 mph, based on an annual extreme fastest mile speed 30 feet above the ground, with a 100-year mean recurrence interval. The wind pressure distributions on the structures are calculated for Exposure C (flat open country) conditions and wind pressure coefficients in accordance with references 3, 4, and 5.

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 structure accelerations and in-structure response spectra are discussed in the Seismic Analysis Report. Tables 1 and 2 provide the SSE horizontal and vertical structure accelerations, respectively, for the NSCW tower and the valve house. The input SSE design spectra curves used for the valve house analysis are shown in figure 8.

Loads due to the SSE include structure inertia loads, seismic induced piping and equipment reactions, hydrodynamic effects of the water in the storage basin, and incremental dynamic lateral soil pressures on the buried walls. The soil bearing pressure diagrams for the nearby buildings for the SSE case are given in figure 5.

The SSE damping values, as percentages of critical, applicable to the tower and valve house design are as follows:

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

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 ft
• Atmospheric pressure differential	-3 psi
• Rate of pressure differential change	2 psi/sec

References 3, 4, and 5 are used to determine the tornado wind pressures on the tower and valve house. The resultant effective velocity pressure profiles (with structure size effects included) corresponding to the above parameters are given in figure 9.

The tower and valve house are both considered to be partially vented structures which experience a pressure drop at the onslaught of the tornado which causes a bursting-type load, and later an atmospheric pressure recovery case in which the normal outside atmospheric pressure combined with the lowered internal pressure produce an inward vacuum-type load. The tower and valve house are both conservatively designed for a ± 3 psi pressure differential across all interior walls and slabs and all exterior walls and roofs to account for these tornado atmospheric effects.

The tornado missile parameters are listed in table 3. For missile trajectories up to and including 45 degrees off the horizontal, the listed horizontal velocities are used. For trajectories greater than 45 degrees, the vertical velocities are used.

The 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)

The load due to probable maximum precipitation is applied to the NSCW tower and valve house roof areas. Special roof scuppers are provided with sufficient capacity to ensure that the depth of ponding water due to the PMP rainfall does not exceed 18 inches. This results in an applied PMP load of 94 psf.

PMP loads are not considered applicable on the valve house missile shield slabs since their top surface is sloped and their edges do not contain parapets or curbs.

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 applied overpressure of 2 psi (acting inwards or outwards) applied as a static load to all exterior surfaces.

3.2.4 Abnormal Loads

The NSCW tower is designed for the thermal effects (T_a) due to NSCW system temperatures during a postulated plant accident condition. These accident design temperatures are summarized in figure 6 for corresponding outside air ambient temperatures of 17°F minimum and 120°F maximum.

There are no other significant abnormal loads applicable to the tower. There are no significant abnormal loads applicable to the valve house.

3.3 LOAD COMBINATIONS AND STRESS/STRENGTH LIMITS

The load combinations and stress/strength limits which have been considered in the reinforced concrete design and miscellaneous structural steel design of the NSCW towers and valve houses are provided in Appendix B.

3.4 MATERIALS

The following materials and corresponding properties have been used in the design and construction of the NSCW towers and valve houses.

3.4.1 Concrete

- Compressive strength $f'_c = 4$ ksi
- Modulus of elasticity $E_c = 3,640$ ksi
- Shear modulus $G = 1,544$ ksi
- Poisson's ratio $\nu = 0.17 - 0.25$

3.4.2 Reinforcement - ASTM A615 Grade 60

- Minimum yield stress — f_y = 60 ksi
- Minimum tensile strength — f_{ult} = 90 ksi
- Minimum elongation — 7-9% in 8 inches

3.4.3 Structural Steel - 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 Stainless Steel Hardware

ASTM A276, Type 304L, stainless steel has been used for miscellaneous structural steel components within the tower which are exposed to the corrosive effects of the NSCW coolant.

- Minimum yield stress — f_y = 25 ksi
- Minimum tensile strength — f_{ult} = 70 ksi

3.4.5 Structural Bolts

ASTM A325 bolts have been used in AISC Type N standard connections outside of the NSCW tower (which includes the valve house). ASTM A307 bolts have been used in miscellaneous concrete anchorage connections outside of the tower. Inside of the tower, stainless steel connections and anchorage have been made with ASTM A276, Type 304L, field fabricated bolts and ASTM A320, Grade B8 premanufactured stainless steel bolts.

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 — γ_m = 126 pcf
- Saturated unit weight — γ_t = 132 pcf

- Shear modulus

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

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

3.4.6.3 Modulus of Subgrade Reaction

- Static
  NSCW tower - 100 kcf
  Valve house - 60 kcf
- Dynamic
  NSCW tower - 275 kcf
  Valve house - 175 kcf

3.4.6.4 Net Bearing Capacities

- Ultimate
  NSCW tower - 61.7 ksf
  Valve house - 133.5 ksf
- Allowable static
  NSCW tower - 20.6 ksf
  Valve house - 44.5 ksf
- Allowable dynamic
  NSCW tower - 30.9 ksf
  Valve house - 66.8 ksf

4.0 STRUCTURAL ANALYSIS OF NSCW TOWER

This section provides the methodologies employed to analyze the NSCW tower in order to determine the design forces on its key structural elements, using the applicable loads and load combinations specified in section 3.0. The structural analysis to obtain these forces is performed using a computer model and conventional analysis techniques. A discussion of the analysis and modeling techniques, selection and application of critical loads, and a description of the computer model and boundary conditions are provided to illustrate the overall method of analysis. In addition, representative analysis results are provided to illustrate the response of key structural elements for governing load combinations.

A preliminary proportioning of key structural elements was done 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.

4.1 SELECTION OF GOVERNING LOAD CASES

An evaluation of design load magnitudes, load factors, and load combinations is performed to determine a set of governing load cases to include in the computer analysis of the overall structural response. It is determined that from all of the load types discussed in section 3.2, only D, L, E, W, W_{tq}, W_{tp}, N, and B loads need be included in this analysis. Appropriate directions are selected for the application of E, W, W_{tq}, and B loads, as discussed in section 4.3, and included as separate load cases in the analysis.

SSE loads, E', are eliminated based on a comparison of all of the SSE loads with the OBE loads, after the appropriate load factors have been applied in accordance with Table B.2 in Appendix B. This comparison shows that the factored OBE loads are consistently larger than the factored SSE loads, and thus the OBE case governs.

The effects of R_o and W_{tm} loads are evaluated, where applicable, on a local area basis (see section 8.2). The localized response is combined with the analysis results of the overall structural response, as applicable, to confirm that design integrity is maintained.

The thermal effects on the tower under operating conditions are investigated using the methodology described in section 8.3.

4.2 ANALYSIS METHODOLOGY AND COMPUTER MODEL

Because of the unique structural configuration of the NSCW tower and the number of load cases and combinations to consider, the finite element technique of mathematical modeling was chosen for the structural analysis. The NSCW tower is analyzed with the Bechtel Structural Analysis Program (BSAP), which is a general purpose computer program for linear-type finite element analyses. This program uses the direct stiffness approach to perform linear elastic analyses of three-dimensional structural models.

The static analysis method of BSAP was used for analyzing the effects of all primary loads. The primary loads consist of a set of BSAP load cases which are representative of the overall structural effects of the design loads as defined in section 3.2. In general, these loads include gravity loads (D and L), seismic loads (E), and atmospheric loads (W, W_{tq}, W_{tp}, N, and B). The application of these loads to the finite element model is discussed in section 4.3.

Dead loads, by their nature, are static loads, and since there are no significant impactive design live loads, all of the live loads may be considered to be applied statically. The seismic maximum structure accelerations (see table 2) obtained from the dynamic seismic analysis of the NSCW tower are used to compute structure inertia loads, which are then applied statically as seismic loads to the finite element model. Since the height of the tower above grade is small when compared to the diameter, and the structure is not wind-sensitive, the wind pressures are also applied statically.

Due to nonsymmetries of the NSCW tower structure and the unsymmetric nature of many of the applied loadings, a complete three-dimensional finite element model was chosen to represent the structure. The structure is modeled using shell, plate, brick, beam, and boundary elements from the BSAP finite element library. Concrete wall and slab thicknesses, and other structural member sizes were selected based on preliminary structural calculations and other nonstructural considerations. The three-dimensional computer model, which is based on the Unit 1 tower configuration, is shown in an expanded isometric view in figure 12. All of the major structural elements of the tower are modeled resulting in a total of 3,804 BSAP elements and 3,397 nodes having 16,613 static degrees of freedom.

The fan deck, fan stacks, crosswalls, and 3-foot-thick and 5-foot-thick sections of the exterior shell wall are modeled with thin shell elements. These are elements that have membrane and bending properties in accordance with small deformation, thin plate theory in which the membrane and bending effects are computed separately and the results superimposed. The grids are made of mostly quadrilateral elements as shown in figure 12, with a limited number of triangular elements used for meshing at grid transition points.

The thickened portion of the shell wall below grade, which supports and encases the basin pumping equipment, is modeled with brick elements. These are eight-node hexahedron, isotropic solid elements having membrane and bending properties in accordance with an isoparametric formulation with three translational degrees of freedom per node. The grid of elements follows the outline of the actual concrete surfaces, with the four pump wells accounted for by omitting elements in the grid at these locations.

Plate elements are used to model the basemat. These are similar to the BSAP thin shell elements but possess a consistent load vector formulated to produce more accurate stresses for flat plates.

Vertical boundary (spring-type) elements are attached to each of the basemat nodes to characterize the foundation media as a set of elastic soil springs. The stiffness of each spring is determined by multiplying the nodal tributary area by the coefficient of vertical subgrade reaction.

All of the tower's concrete beams, which consist of those at the fill level, eliminator level, and fan deck level, are modeled with beam elements. The columnar portions of the shell wall separating the air intake openings at grade are also modeled with beam elements. These beam elements are all located at the centerline of each of the structural elements with the exception of the fill and eliminator level lateral support beams. The centerlines of these beams are assumed to be the same as the main north-south vertical load carrying beams. Since these differences are small, this approximation has no adverse effects on the results.

The horizontal translation of the model is fixed at the basemat level by a series of stiff boundary elements located around the perimeter of the basemat underneath the exterior shell wall. The effects of the soil at the sides of the structure are accounted for by including statically applied lateral soil pressures in the analysis.

After the computer model was assembled, a series of test loading cases were run using BSAP, and the results reviewed to ensure proper model behavior. The design loads are applied as described in section 4.3, and analyzed in a series of BSAP runs to obtain the design forces for all of the applicable primary load cases.

The combination of co-directional responses due to three component earthquake effects are 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.,

$$R = \pm 1.0\ R_i \pm 0.4\ R_j \pm 0.4\ R_k$$
$$R = \pm 0.4\ R_i \pm 1.0\ R_j \pm 0.4\ R_k$$
$$R = \pm 0.4\ R_i \pm 0.4\ R_j \pm 1.0\ R_k$$

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. The use of these methods, as they apply to the NSCW tower analysis, is further discussed in section 4.3. The combination of torsion effects due to seismic wave propagation is also discussed in section 4.3.

4.3 APPLICATION OF LOADS

The dead load weight of the concrete structural elements of the tower is input into BSAP by using a static gravity acceleration load applied to beam, brick, plate, and shell elements having the proper thicknesses and a mass density equivalent to 150 pcf. The mass of any concrete which is not modeled with BSAP elements is input as nodal masses (e.g., roof parapet, fan motor missile protection walls and slabs, separation walls on top of the buttress, equipment pedestals). The weights of miscellaneous steel, equipment, and major piping are also input as nodal masses with an applied static gravity load.

The design uniform live loads are applied as element pressure loads on the shell elements representing the fan deck, and on the brick elements at the top of the thickened zone of the shell wall supporting the basin pumping equipment. Calculated live load reactions from the eliminator level platforms, which are not modeled, are applied as nodal forces at appropriate points. Equipment operating live load reactions of the fans and fan motors are also applied as nodal forces.

The hydrostatic pressures of the storage basin water on the inside face of exterior shell wall and on the top face of the basemat are applied as element pressure loads. The pressure load on each shell wall element is applied as a uniform element pressure and is computed by averaging the hydrostatic pressure along the height of the element. In general, this procedure is used for all of the applied element pressure loading cases which are a result of varying pressure diagrams (e.g., lateral soil pressures and wind pressures on the cylindrical wall). In the analysis, two extreme cases are considered, i.e., (1) the storage basin is full to the design high water line at elevation 217'-9" and (2) the basin is completely empty. Hydrostatic pressures are not applied to the submerged bottom portion of the crosswalls when the basin is full, since the pressures on each face of the walls are equal and therefore cancel each other.

Static lateral earth pressures on the exterior wall below grade are calculated by the equivalent-fluid method. Because of the stiffening effects due to ring action of the cylindrical shell, and because of the stiffening effect of the crosswalls and beams spanning across the basin within the tower superstructure, the basin exterior wall is considered to be an unyielding wall. Thus, the "at-rest" condition is used to determine the lateral soil pressure based on an earth pressure coefficient, K_o, of 0.7 for heavily compacted backfill. Included as part of the at-rest static pressures on the outside face of the wall is the effect of the hydrostatic pressure below the high groundwater table at elevation 165'-0". Also, an upward buoyant force is applied to the bottom face of the basemat plate elements.

The surcharge loads on top of the soil due to the adjacent structures near the towers all create an effective increase in lateral pressure on the storage basin wall. The procedure used to determine the magnitude and distribution of these pressures is based on an elastic-type analysis of the soil. Each of the adjacent building basemat areas is divided into a grid of much smaller "sub-area" elements (see figure 5). The building soil bearing pressure diagram is then approximated as a series of concentrated point loads at each of the grid line intersections, which are calculated by multiplying the tributary area times the pressure at that point. Using the Boussinesq solution for stresses in a semi-infinite elastic medium due to a point load applied at its surface, the lateral soil pressure is determined for all of the point load surcharges at the centroid of each of the storage basin wall elements within the range of influence of the surcharge load. Since the wall is considered to be a rigid unyielding wall, the horizontal Boussinesq soil stress is doubled to give the effective lateral pressure on the wall (see reference 6). The total surcharge lateral pressure for each of the wall elements is determined by summing the effects of each point load of each of the nearby buildings. Because of the complexity of this load transformation, a computer program was written and used to perform the actual calculations. The analysis was performed for both towers 1A and 1B, and an examination of the results revealed that since the valve house effect was much greater than the other buildings, and since this effect was the same for both towers 1A and 1B, it is sufficient to envelop the results of both cases and only input one static surcharge lateral earth pressure case for the tower's stress analysis.

The static lateral earth pressure due to the 250 psf uniform live load on the adjacent soil at grade was also considered. This pressure was obtained simply by multiplying the uniform surcharge load by the static at-rest lateral earth pressure coefficient, $K_o = 0.7$, and applying this as a uniform pressure to each of the storage basin wall elements.

The seismic structure inertia loads were included in the finite element model as static acceleration loads applied to the model masses. The BSAP model was divided into enough element groups so that different accelerations could be applied at each level as shown in table 1. The static gravity acceleration for a given level is applied to all of the element masses tributary to that level.

Since these accelerations are applied to the BSAP finite element model as static gravity loads, any torsion effects due to actual eccentricities between the center of rigidity of each level and the center of mass are accounted for automatically in the BSAP stress analysis. In addition, to account for the torsional motion imparted to the structure due to the effects of seismic wave propagation, an additional torsional moment, equal to the building's story shear times an eccentricity of 5 percent of the maximum plan dimension at that level, is considered in the analysis.

This additional torsion to be applied to the BSAP model is calculated for elevations identified in table 1. These twisting moments are calculated by multiplying the equivalent horizontal seismic force at each level by 5 percent of the maximum plan dimension at that level. The resulting twisting moments are applied to the BSAP model as a series of concentrated nodal forces around the perimeter of the shell wall at each level. The direction of the twisting moments is the same for each level in order to maximize the shear due to torsion at the base. Also, the direction of the applied twisting moments is selected such that they are additive to the twisting moments resulting from the eccentricities between the actual center of gravity and center of rigidity.

The hydrodynamic pressures due to horizontal seismic ground motion, which are applied to the BSAP storage basin wall shell and brick elements, are calculated in accordance with Housner's method (see reference 1). By this method, the hydrodynamic effects are separated into impulsive (rigid) and convective (sloshing) parts. The impulsive effects are based on the maximum structure acceleration at elevation 180'-0" (the location of the impulsive mass), and the convective effects are based on the maximum structure acceleration at elevation 200'-0" (the location of the convective mass) for one-half percent damping, corresponding to a calculated sloshing period of 5.4 seconds. The resulting pressure diagram, whose vertical distribution is shown in figure 10, has a peak value at the front and back portion of the wall directly in the line of action of the seismic force, and tapers off to zero at the sides of the cylinder. Housner's method also gives the hydrodynamic pressures on the basemat due to horizontal seismic ground motion. The calculated pressures, which vary from upward at one edge of the basemat to downward at the opposite edge, are applied to the BSAP model basemat plate elements.

For vertical seismic effects, the hydrodynamic pressures, which are assumed to be directly proportional to the hydrostatic pressures, are calculated based on the maximum vertical structure acceleration of the basemat level.

In determining the lateral dynamic earth pressure distribution on a cylindrical structure for a given direction of seismic ground motion, it is assumed that those wall elements whose faces are perpendicular to the line of action of the seismic force receive the brunt of the force, whereas those whose faces are at some other angle to the line of action receive a smaller portion of the load. This results in a horizontal pressure distribution similar to that for the hydrodynamic pressures. The vertical peak pressure profiles for the dynamic incremental lateral soil pressure are given in figure 10.

Four different horizontal directions were considered for the line of action of seismic loads (south, east, southwest, and southeast), so that after combining the two horizontal perpendicular directions, two cases will result, i.e., (1) seismic loads applied parallel to the BSAP model global axes through the crosswalls and (2) seismic loads applied diagonally at 45 degrees to these axes.

The torsion effects due to seismic wave propagation are considered for the horizontal earthquake components before the SRSS method is applied. Since the inherent center of mass to center of rigidity eccentricity components along each of the two mutually perpendicular directions are to be augmented by the extra 5 percent of the maximum building dimension, the effects of the corresponding applied twisting moments must be considered additive to the effects of each of the horizontal seismic load cases. Since the torsional component may either be clockwise (as input into BSAP), or counterclockwise, the absolute values of the torsional seismic load case stress components are added to the absolute values of the horizontal seismic load case stresses. These absolute value sums for each of the two perpendicular horizontal components are then combined with the vertical component using the SRSS method to obtain the total seismic structural response. This is done for both the case of the storage basin being full and the case with it empty.

During an earthquake, the lateral earth pressure effects on the storage basin wall due to the surcharge loads of the adjacent buildings are different from those corresponding to the static at-rest condition. The lateral seismic inertia forces of each of these structures generate overturning moments which, when combined with their vertical loads, create linearly varying bearing pressure distributions on the soil (assuming rigid basemats). Lateral earth pressures on the wall due to these surcharges are calculated using the same procedure as for the static condition (i.e., Boussinesq elastic analysis). In order to reduce the total number of cases to analyze, it is conservatively assumed that all of the adjacent structures are tending to overturn toward the towers at the same time. It is also assumed that the worst case is when the vertical seismic acceleration component for each of the structures is acting downward. The component factor method is used when determining the soil bearing pressure diagrams with 1.0 times the downward seismic force combined with 0.4 times the lateral forces. Since the dead loads and live loads are included in the bearing pressure diagrams for each structure, the lateral pressures on the tower wall from this calculation are not incremental, but instead, replace the static at-rest surcharge pressure effects in all of the loading combinations involving seismic loads. As was the case for the static condition, the lateral earth pressures are determined separately for each of towers 1A and 1B, and the results enveloped to give only one set of pressures to be applied in the BSAP analysis. Also, these pressures are considered additive to the effects of the SRSS combination of the tower's structure inertia loads, hydrodynamic pressures, and the Mononobe-Okabe dynamic incremental soil pressures.

The wind pressures corresponding to the design wind are calculated using the methodology and wind pressure coefficients as given in references 3, 4, and 5. No reductions to the effective velocity pressures are made for the effects of direct shielding provided by the valve house or other nearby buildings. The wind pressure coefficients used to determine the pressure distribution applicable for the tower are given in figure 13. Two different directions are considered in the BSAP stress analysis: wind acting southward, and wind acting southeast. Also two different possible values are considered for the internal pressure coefficient; one corresponding to the fans operating, and one for the fans not operating (see figure 13).

The tornado wind velocity pressures are applied using procedures paralleling those for the severe environmental design wind, the primary difference being the treatment of the tornado horizontal and vertical pressure profiles. The tornado velocity pressures are assumed not to vary with height. Instead, the velocity pressures vary with horizontal distance from the center of the tornado, with the peak at the radius of the tornado. For design purposes, an average value of wind velocity is used rather than the maximum wind velocity for the entire structure. The magnitude of the average value is dependent on the size of the structure versus the design radius of the tornado, and is determined in accordance with reference 3. The resultant average velocity is shown in figure 9.

As described in section 3.2.3, the tornado atmospheric pressure differential effects are represented by either a 3 psi outward bursting-type pressure, or by a 3 psi inward vacuum-type load. For each of these effects, two separate cases are considered: one with a large tornado over the whole tower such that the pressure differential is across all of the exterior shell wall, and one with a small radius tornado centered over one fan stack such that the pressure differential is applied across the walls surrounding only one cell. Also considered are pressure loadings on the internal structural components of each cell which are the result of directional airflow induced by a pressure differential from the fan stack openings to the air inlet openings at grade.

Tornado missile impact loads are not included in the overall BSAP stress analysis of the structure, but their local effects are examined separately as discussed in section 8.2.

The design blast load (refer to section 3.2.3) is a static pressure of ±2 psi. Since the tower walls and slabs are designed to withstand a statically applied load of ±3 psi due to a tornado atmospheric pressure differential, the direct effects of the blast load on each individual structural element are not considered. However, in order to assess the ability of supporting structural elements to carry the loads transmitted from the directly loaded exterior surfaces, a load case is included in the BSAP analysis which has the 2 psi pressure applied only to the north half of the exterior wall. This case, which is representative of the blast overpressure on the structure after the incident blast wave has traversed over only half of the structure, results in the maximum net horizontal resultant blast force to be applied to the structure. This, in turn, maximizes the lateral load carried by the crosswalls and columns, and also maximizes the overturning effects due to the blast.

The PMP water ponding load on the fan deck is simply applied as downward element pressures on the representative shell elements.

4.4 ANALYSIS RESULTS

After the primary load cases are analyzed by BSAP, and the seismic load components are combined using the COMBINE module of the BSAP-POST computer program, the results are reviewed for correctness. As a part of this process, stress contour plots are made using the BSAP-POST program to verify that reasonable results are obtained.

Representative analysis results for governing load combinations are presented in figures 14 through 19 for key structural elements.

5.0 STRUCTURAL DESIGN OF NSCW TOWER

This section provides the methodologies used to design the key structural elements of the NSCW tower for the results of the structural analysis described in section 4.0. The structural elements are designed either manually or by computer in accordance with the applicable sections of the codes listed in section 3.1.1. A discussion of the design procedures, selection of critical load combinations, and sample design results and design details are provided to illustrate the overall design process.

5.1 SELECTION OF GOVERNING LOAD COMBINATIONS

The BSAP analysis results for all of the primary load cases, selected as described in section 4.1, are reviewed to attain a thorough understanding of the structural behavior corresponding to each of the different load types. A special enveloping subroutine, written and compiled into the COMBINE module of the BSAP-POST computer program, is used to find the maximum positive and negative stress components and corresponding load cases for all of the elements in the finite element model. Stress contour plots, generated by the BSAP-POST program, highlight the location of critical elements. The load combination equations of Appendix B are then systematically evaluated to determine a set of governing combination cases.

For the design of the key structural concrete elements of the NSCW tower, load combination equations 7 and 11 (Appendix B, Table B.2) involving SSE loading are eliminated based on considerations discussed in section 4.1. Equations 9 and 10 involving abnormal loads effects are examined separately as discussed in section 8.3. Equations 4, 5, and 6 involving operating temperature effects are also examined separately using the design methodology described in section 8.3. The remaining six general critical load combination equations (i.e., equations 1, 2, 3, 8, 12, and 13) are listed in table 4.

In applying these general equations, due consideration is given to:

A. The possibility of the storage basin being full or empty of water

B. The possibility of live load magnitudes varying from zero to their full design value

C. The different possible directions to maximize the effects of applied wind and seismic loads

D. The possibility of the fans operating or not operating with regard to its effects on the wind pressures

E. The possibility of either an outward bursting-type or inward vacuum-type load when considering tornado atmospheric effects, along with the possible cases of the pressure differential occurring across all of the exterior wall and slab surfaces simultaneously or else only across the walls and fan deck portion surrounding one of the four cells

F. The different possible combinations of tornado effects as given in section 3.2.3.

Only load cases which are judged to be significant are included in the design computations.

For each of the governing load combination cases involving OBE loads, permutations of the equation are made which consider possible combinations of plus or minus seismic stresses.

5.2 DESIGN METHODOLOGY

The reinforced concrete design of the key structural elements of the NSCW tower is primarily done using the OPTCON module of the BSAP-POST computer program, in accordance with the strength design method of the ACI 318 Code.

BSAP-POST is a general-purpose, post-processor program for the BSAP finite element analysis program, which consists of a collection of modules that perform specific independent post-analysis tasks. BSAP-POST reads computed BSAP results 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 specified stress and strain limitations. Any load combination whose design axial force and corresponding moment (load set) falls within the envelope of the interaction diagram indicates all stress and strain code criteria are satisfied.

The minimum area of flexural steel required to satisfy all of the critical load combination equations is determined for each of the key structural elements of the tower by running OPTCON for all of the corresponding finite elements and the appropriate associated BSAP stress results. For each of the structural elements, the concrete cross-sectional dimensions are input along with a minimum trial area of tension (A_s) and compression (A_s') reinforcement. The OPTCON program then evaluates the concrete section for each load combination load set, and iteratively increments the A_s and/or A_s' values, sweeping all possible solutions, until an optimum solution (i.e., minimal total reinforcement) is found. For slabs and walls, the required area of steel is calculated on a per-foot-width basis, for each of the two principle orthogonal directions, with the membrane force and bending moment stress components input accordingly.

The special load combination processor of OPTCON is used to consider the possible combinations of plus or minus seismic stresses by varying sub-combinations of the design axial force and design bending moment.

The minimum flexural reinforcing steel computed for each of the key structural elements is summarized in the form of contour plots on the wall and slab surfaces. These plots are generated using the plotting module of the BSAP-POST program.

5.3 DESIGN RESULTS

The design results for governing load combinations are presented in figures 14 through 19 for representative key structural elements of the NSCW towers.

5.4 DESIGN DETAILS

Representative concrete reinforcing design details are provided in figure 20.

6.0 STRUCTURAL ANALYSIS OF NSCW VALVE HOUSE

This section provides the methodologies employed to analyze the NSCW valve house in order to determine the design forces on its key structural elements, using the applicable loads and load combinations specified in section 3.0.

Proportioning of the key structural elements is done as described in section 4.0 for the NSCW tower.

The majority of the structural analysis is performed by computer analyses in which the valve house is modeled as an assemblage of finite elements, and the analysis performed using the standard finite element method. For these analyses, the modeling techniques, application of loads, and description of the computer model and boundary conditions are provided herein.

Some of the structural analysis also involved manual calculations using standard analysis techniques. For this manual portion, the analysis techniques, assumptions, and application of loads are discussed to illustrate the methodology.

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

6.1 SELECTION OF GOVERNING LOAD CASES

The procedure used to determine the governing load cases to be used in the analyses is similar to that described in section 4.1 for the NSCW tower. The OBE load case governs over the SSE load case based on a comparison of the valve house OBE and SSE design spectra curves given in figures 7 and 8, respectively. This comparison demonstrates that the applicable 4 percent damping OBE spectra curve, after its acceleration values are increased by a 1.9 concrete design load factor, envelops the applicable 7 percent damping SSE curve multiplied by a 1.0 design load factor, for both the horizontal and vertical cases.

It is determined that for the superstructure analysis, only load types D, L, E, W, W_{tq}, W_{tp}, and N (see section 3.2) need be considered. By evaluating the results of the superstructure analysis, it is determined that only D, L, and E loads need be considered for both the basemat and the pilaster analysis.

6.2 ANALYSIS METHODOLOGY AND COMPUTER MODEL

The NSCW valve house is analyzed by the finite element technique using the BSAP computer program (see section 4.2 for program description), and supplemented by manual analyses of selected key structural elements. The superstructure (all portions of the valve house above the basemat) and the basemat are analyzed by separate computer analyses, using the same basic model (see figure 21) with appropriate changes made to the boundary conditions respectively. The design forces for the pilasters are obtained manually by a two-dimensional, single-bay frame analysis, assuming frame action as defined in section 2.4.2. Other miscellaneous manual calculations include the tornado missile impact analysis of the missile shield slabs above the roof (see section 8.2).

6.2.1 Computer Model

Due to the irregular shape of the structure, the NSCW valve house is modeled and analyzed using the BSAP program. The three-dimensional model is comprised of quadrilateral and triangular finite elements with grids as shown in figure 21. The elements are either of the shell or plate type with properties as described in section 4.2. There are a total of 1210 nodal points connected by 1213 elements. Typical elements are quadrilateral shapes, 3 to 5 feet on a side.

Doorways, blockouts, and other significant openings in the walls and slabs are accounted for by giving the corresponding elements in the grid a relatively small thickness.

6.2.2 Superstructure Analysis

The superstructure is analyzed using the BSAP computer model with all of the basemat nodes completely fixed.

All of the critical load types discussed in section 6.1, other than the seismic case, are applied statically to the model for the same reasons discussed in section 4.2 for the NSCW towers. To obtain the stresses due to concrete inertia loads, a separate dynamic response spectrum analysis is made using as input the design spectra curves given in figure 7. The eigenvalue analysis of the BSAP finite element model is performed using the Householder-QR kinematic reduction solution method, with mass lumping employed to reduce the model to 255 dynamic degrees of freedom. The combination of modal responses for the 21 modes extracted is done using the SRSS method. The combination of co-directional responses from this analysis due to the three component earthquake effects is performed using the SRSS method as described in section 4.2. To account for the torsion effects due to seismic wave propagation, as discussed in section 4.3, a separate statically applied torsion load case is included in the superstructure's static analysis, and later combined absolutely with the response spectrum analysis stresses. Dynamic incremental lateral earth pressures are also applied statically and later combined with the other seismic effects.

The application of loads for the anlaysis is further discussed in section 6.3.1.

6.2.3 Basemat Analysis

The basemat is analyzed separately using the BSAP computer model described in section 6.2.1, with vertical spring-type boundary elements attached to each of the basemat nodes in the model to represent an elastic foundation media. The stiffnesses of these soil springs is determined by a similar methodology to that discussed in section 4.2 for the NSCW tower model. To prevent rigid body translation, a series of horizontal spring-type boundary elements are attached to appropriate basemat nodes.

For the basemat analysis, all of the load types are applied statically in a BSAP static analysis. The three component earthquake effects are combined using the component factor method described in section 4.2.

6.2.4 Frame Analysis

Because of the modeling technique, appropriate design forces for the pilasters were not directly obtainable from the computer analyses. To obtain these forces, a conservative and simple independent frame analysis is performed manually using conventional analysis techniques.

The interior missile shield support walls are assumed to act in conjunction with the pilasters to form two interior single-bay frames with fixed bottom supports at the basemat. As shown in figure 3, one of the analyzed frames assumes that the small thickened portion of the exterior northern wall acts as the second column of the frame.

The two frames are analyzed for primarily lateral load action. The total lateral design load for each of the frames is determined based on a relative rigidity comparison of the frames and the two "parallel" exterior side walls. After the tributary forces for each of the frames are determined, they are analyzed separately using the moment distribution method to obtain the pilaster design forces.

6.3 APPLICATION OF LOADS

6.3.1 Superstructure Analysis

With the exception of seismic loads where stresses are determined by the response spectrum dynamic analysis method using the appropriate input design spectra curves, all loads are applied statically to the BSAP computer model.

The appropriate magnitudes of the dead loads, D, and live loads, L (including static lateral earth pressures), are applied to the BSAP computer model using the same procedures used for the NSCW tower as described in section 4.3. To determine the magnitudes of the applied soil pressures, the at-rest condition (implying relatively stiff non-yielding walls) is assumed and the pressures calculated accordingly. The effective pressures due to adjacent building surcharge loads are determined based on an elastic Boussinesq approach using the same computer program developed for the NSCW tower analysis.

To account for torsion effects on the walls due to seismic wave propagation, a separate load case is included in the BSAP static analysis consisting of a series of concentrated nodal loads, applied to the valve house model at the roof elevation, which represent a resulting twisting moment equal to the superstructure's lateral design seismic force times 5 percent of the maximum plan dimension at that level. The resulting stresses are subsequently combined with the three component earthquake dynamic response spectrum analysis stresses using the load combination process module of the BSAP-POST program.

Mononobe-Okabe dynamic incremental soil pressures are also applied to the computer model in the BSAP static analysis, and later directly added to the other earthquake effects.

Design wind pressures and tornado wind pressures are determined by assuming a rectangular structure and applying the appropriate wind pressure coefficients of reference 4. Tornado atmospheric pressure effects are also appropriately applied.

6.3.2 Basemat Analysis

Since the analysis of the basemat uses the full finite element computer model, all of the non-seismic loads and the Mononobe-Okabe pressure increments are applied in the same manner as described in the previous section for the superstructure.

The effects of the superstructure seismic inertia loads on the basemat are considered by applying appropriate forces at selected superstructure nodes, where the masses are lumped (see section 6.2.2), based on the seismic acceleration values obtained from the response spectrum analysis. Three separate seismic load cases are prepared corresponding to each of the two horizontal and vertical orthogonal directional components of the design earthquake.

6.3.3 Frame Analysis

The dead loads and live loads for this analysis are determined on a tributary area basis and appropriately applied to the frames. The OBE lateral seismic loads, which are distributed to the frames based on relative stiffnesses, are determined using acceleration values from the superstructure response spectrum analysis.

6.4 ANALYSIS RESULTS

As was done for the NSCW tower, stress contour plots are made of the valve house superstructure and basemat analysis results using the BSAP-POST program for analysis verification purposes.

Representative analysis results for the superstructure, basemat, and frame analyses are given in figures 22 through 26.

7.0 STRUCTURAL DESIGN OF NSCW VALVE HOUSE

This section provides the methodologies used to design the key structural elements of the NSCW valve house for the results of the structural analysis described in section 6.0. The structural elements are designed either manually or by computer in accordance with the applicable sections of the codes listed in section 3.1.1. A discussion of the design procedures, selection of governing load combinations, and sample design results and design details are provided to illustrate the overall design process.

7.1 SELECTION OF GOVERNING LOAD COMBINATIONS

The basis for the selection of governing load combinations to include in the valve house design is the same as that discussed in section 5.1 for the NSCW tower. Load combination equations 1, 2, 3, 8, and 13 of Appendix B, Table B.2, are determined to be significant for the design of the superstructure, and equations 1 and 3 of this table are determined to be significant for the basemat and pilaster design. These equations are summarized in table 5.

7.2 DESIGN METHODOLOGY

For those portions of the NSCW valve house which were analyzed by computer, the reinforced concrete is designed using the OPTCON module of the BSAP-POST computer program. This program is described in section 5.2, and its implementation is basically the same as described in that section for the NSCW tower.

The pilaster reinforced concrete design consists of manually sizing and detailing the main reinforcing steel and lateral column ties in accordance with the requirements of the ACI 318 Code.

7.3 DESIGN RESULTS

The design results for governing load combinations are presented in figures 22 through 26 for representative key structural elements of the NSCW valve house.

7.4 DESIGN DETAILS

Representative concrete reinforcing design details are provided in figure 27.

8.0 MISCELLANEOUS ANALYSIS AND DESIGN

This section provides a summary of the significant miscellaneous analysis and design performed for the NSCW tower and valve house, other than the structural analysis and design of the key structural elements as discussed in sections 4.0, 5.0, 6.0. and 7.0. Important items relating to overall structural effects such as stability considerations are addressed, as well as items relating to local effects such as provisions for tornado missile impact.

8.1 STABILITY ANALYSIS

The overall structural stability of the NSCW tower and valve house is investigated by evaluating the factor of safety against sliding, overturning, and flotation for governing load combinations.

8.1.1 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.

Since seismic loads govern over all other lateral loads, stability calculations are performed for both the NSCW tower and valve house for this case using equivalent lateral inertia forces based on the structure accelerations given in tables 1 and 2. The frictional resistance along the potential sliding failure surface is appropriately reduced to account for the effects of bouyancy and the vertical earthquake effects.

8.1.2 Overturning

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

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

The factor of safety against overturning for 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 appropriately reduced to account for the effects of bouyancy and the vertical component of the design earthquake.

The factor of safety against overturning using the energy balance method is defined as the ratio of the increase in the potential energy at the point of overturning about the critical edge of the structure to the maximum kinetic energy that could be imparted to the structure as a result of earthquake loading. The energy balance analysis methodology is described in reference 7.

Upon examining the geometric configuration and physical characteristics of the NSCW tower, along with the conditions of foundation support, it is obvious that the structure remains stable against overturning with a large margin of safety for all credible loading conditions, since the majority of the structure is deeply embedded (approximately 90 feet) in the densely compacted Category 1 backfill, with its center of gravity well below grade.

8.1.3 Flotation

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

The buoyant force is calculated using the high groundwater level of 165'-0". Since the bottom of the valve house basemat is well above this elevation, a flotation stability analysis is performed only for the NSCW tower.

8.1.4 Analysis Results

The minimum required factors of safety and the calculated factors of safety for stability of the NSCW tower and valve house structures are provided in table 6.

8.2 TORNADO LOAD EFFECTS

This section provides the general procedures used in the analysis and design of the NSCW tower and valve house for the impact effects of the tornado-generated missiles discussed in section 3.2.3 and given in table 3. Provisions are made to ensure that safety-related equipment, systems, and components are protected from damage resulting from these effects. In general, this consists of assessing the adequacy of the structure and its components to withstand the effects of missile impact, and providing missile-resistant barriers where necessary to protect the safety-related items.

8.2.1 Establishment of Missile Barriers

The safety-related equipment, systems, and components housed by the structures are identified, and missile barriers are established which will protect these items against the postulated spectrum of tornado-generated missiles. In general, these barriers comprise portions of the exterior building surfaces as well as shielding elements provided specifically for missile protection.

For the NSCW towers and valve houses, all of the exterior walls and slabs above grade function as missile barriers to protect the safety-related internals. The splash ring wall (see section 2.6) also functions to prevent horizontal missiles from entering the air intake openings.

To protect safety-related items located between the tower and valve house, a special arrangement of concrete barrier missile shielding slabs and walls is provided which projects upward and outward from the valve house roof over this area (see section 2.3.2). A series of short concrete stubs, projecting from the tower and valve house walls surrounding the interface of these two structures, is designed to prevent small missiles from entering this area through the 5-1/2-inch seismic gaps.

The NSCW tower fan stacks protect the fans from damage due to horizontal missile strikes, and concrete barrier walls and cover slabs are provided to protect the fan motors.

Based on a probabilistic study, it is concluded that the probability of tornado missiles disabling the NSCW system is much lower than the acceptance criterion of 10^{-7} per year given in Standard Review Plan 2.2.3. Therefore, no special barriers are required for tornado missile protection of the NSCW tower fans.

8.2.2 Analysis and Design Methodology

The methodology used to analyze and design the structural elements and specially provided barriers to withstand the effects of tornado missiles is given in Appendix C, along with the associated criteria. Appropriate consideration is given to the possible concurrent effects of other tornado loads (see section 3.2.3), as well as the other design loads given in load combination equation 8 of Tables B.1 and B.2 of Appendix B.

The majority of the missile impact structural response analyses performed for the NSCW tower and valve house are by the response chart solution method (see Appendix C, Section C.3.1) using standard resistance functions and response periods.

To check and verify the adequacy of the combined structural response of the irregularly shaped fan stack and integrated fan deck slab and beam support system, an extensive study utilizing the BSAP computer program is performed. A three-dimensional BSAP finite element model of one-quarter of the fan deck level is used to compute the structural response. BSAP static analyses and dynamic time history analyses are performed for critical missile impact forcing functions applied at critical impact locations to investigate the response. Based on these results, the available resistance force and structural response period are defined for each of the critical cases, and the response chart solution method is used to assess the design adequacy.

8.2.3 Analysis Results

The missile impact analyses of the NSCW tower and valve house missile barriers verify that the provided designs are adequate to protect the associated safety-related items from the damaging effects of the postulated missiles. Representative analysis results for critical barrier elements are given in table 7.

8.3 ABNORMAL LOADS EFFECTS

The primary function of the NSCW tower structure is to transfer the heat loads to the atmosphere and to the storage basin of the tower which serves as the ultimate heat sink, and therefore, special consideration is given to the evaluation of thermal effects.

A thermal analysis of the NSCW tower structure is performed to ensure that structural integrity is maintained under the abnormal conditions associated with a plant accident involving the NSCW system, as well as under the normal operating and plant shutdown conditions. The design system temperatures are used along with the outside air and soil temperatures, to obtain the maximum critical temperature gradients across the key structural elements. The thermal analysis consists of evaluating the effects of these gradients acting on the structure in combination with the effects of other design loads.

Load combinations involving thermal loads (refer to Appendix B) are evaluated to determine the critical combination cases to analyze. The basis for this selection involves reviewing the results of the structural analysis as described in section 4.0, and the OPTCON analysis results as described in section 5.0, to determine the controlling load combinations for each of the structural elements. It is determined that only load combinations containing OBE need be evaluated in the thermal analysis of the key structural elements of the NSCW tower.

The OPTCON module of the BSAP-POST computer program (see section 5.2 for general description) is used to perform the thermal analysis of the key structural elements. The concrete reinforcing selected in the structural design phase described in section 5.0, is reanalyzed by the OPTCON program for each of the concrete cross sections to verify structural adequacy when thermal effects are included.

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

The thermal analysis of the NSCW tower's key structural elements verifies that the reinforcing steel selected in the design phase discussed in section 5.0 is adequate for the critical load combinations with thermal effects included.

8.4 FOUNDATION BEARING PRESSURE

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

9.0 CONCLUSION

The analysis and design of the NSCW tower and valve house includes all credible loading conditions and complies with all applicable design requirements.

10.0 REFERENCES

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

2. Seed, H. B. and Whitman, R. V., Design of Earth Retaining Structures for Dynamic Loads, ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth-Retaining Structures, pp. 103-147, 1970.

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

4. "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.

5. Wind Forces on Structures, Transactions of the American Society of Civil Engineers, Vol. 126, Part II, 1962.

6. Newmark, N. M., Influence Charts for Computation of Stresses in Elastic Foundations, University of Illinois Engineering Experiment Station, Bulletin No. 338, Urbana, Illinois, pp. 5-25, 1942.

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

TABLE 1

SEISMIC ACCELERATION VALUES FOR NSCW TOWER

Elevation	Structure Accelerations (g's)(1)				Remarks
	OBE		SSE		
	Horiz.	Vert.	Horiz.	Vert.	
250'-11"	0.16	0.16	0.27	0.26	Fan Deck
242'-5"	0.16	0.16	0.26	0.26	
	0.16	0.24	0.26	0.34	Eliminator Level Beams
230'-9"	0.15	0.16	0.25	0.27	
	0.15	0.15	0.25	0.25	Fill Level Beams
218'-6" (grade level)	0.15	0.16	0.25	0.27	
200'-0"	0.14	0.15	0.23	0.26	
180'-0"	0.13	0.15	0.22	0.25	
137'0"	0.12	0.15	0.19	0.25	Basemat

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

TABLE 2

SEISMIC ACCELERATION VALUES FOR NSCW VALVE HOUSE

Elevation	Structure Accelerations (g's)[1]						Remarks
	OBE			SSE			
	Horiz. (E-W)	Horiz. (N-S)	Vert.	Horiz. (E-W)	Horiz. (N-S)	Vert.	
248'-0"	0.40	0.45	0.32	0.67	0.74	0.53	Upper Missile Shield
241'-6"	0.35	0.36	0.28	0.58	0.59	0.46	Lower Missile Shield
232'-6"	0.29	0.27	0.25	0.48	0.45	0.42	Roof
218'-6" (grade level)	0.20	0.21	0.24	0.33	0.35	0.41	Mezzanine
205'-0"	0.12	0.14	0.16	0.20	0.24	0.26	Basemat

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

TABLE 3

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' Wood Plank	200	216	200	160
3" Ø std x 10' Steel Pipe	78.5	212	200	160
1" Ø x 3' Steel Rod	8	Unlimited	317	254
6" Ø std x 15' Steel Pipe	285	101	160	128
12" Ø std x 15' Steel Pipe	744	46	150	120
13-1/2" Ø x 35' Wood 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 4

GOVERNING LOAD COMBINATIONS
FOR STRUCTURAL ANALYSIS OF NSCW TOWER(1)

EQN	D	L	W	E	W_t	N	B
1	1.4	1.7	-	-	-	-	-
2	1.4	1.7	1.7	-	-	-	-
3	1.4	1.7	-	1.9	-	-	-
4	1.0	1.0	-	-	1.0	-	-
5	1.0	1.0	-	-	-	1.0	-
6	1.0	1.0	-	-	-	-	1.0

(1) See Appendix A for definition of load symbols.

TABLE 5

GOVERNING LOAD COMBINATIONS
FOR STRUCTURAL ANALYSIS OF NSCW VALVE HOUSE(1)

EQN	D	L	W	E	W_t	N
1	1.4	1.7	-	-	-	-
2	1.4	1.7	1.7	-	-	-
3	1.4	1.7	-	1.9	-	-
4	1.0	1.0	-	-	1.0	-
5	1.0	1.0	-	-	-	1.0

(1) See Appendix A for definition of load symbols.

TABLE 6

FACTORS OF SAFETY FOR STRUCTURAL STABILITY

Structure	Load[(1)(2)] Combination	Overturning Factor of Safety			Sliding Factor of Safety		Flotation Factor of Safety	
		Minimum Required	Calculated: Equivalent Static Method	Calculated: Energy Balance Method	Minimum Required	Calculated	Minimum Required	Calculated
NSCW Tower	D + H + E	1.5	-	-	1.5	see note (3)	-	-
	D + H + E'	1.1	-	-	1.1	1.9	-	-
	D + F'	-	-	-	-	-	1.1	2.1
NSCW Valve House	D + H + E	1.5	2.2	see note (3)	1.5	1.7	-	-
	D + H + E'	1.1	1.3	4.5	1.1	1.1	-	-

(1) D = Dead loads
H = Lateral earth pressure
E = OBE loads
E' = SSF loads
F' = Buoyant force

(2) The effects of the design wind, tornado, and blast are less critical than the effects of the design OBE and SSE.

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

TABLE 7

TORNADO MISSILE ANALYSIS RESULTS[1]

Structure	Element	Element Dimensions Length (ft)	Width (ft)	Thickness (ft)	Computed Ductility	Allowable Ductility
NSCW Tower	Fan Stacks	25.0 (4)	14.5	1.25	6.1	10.0 (3)
	Splash Ring	14.0	19.0 (5)	2.0	10.0	10.0
	Fan Motor Enclosure	11.25	5.0	1.75	7.0	10.0
NSCW Valve House	Roof	28.0	19.0	1.75	- (2)	10.0
	Missile Shields	28.0	5.0	2.0	8.6	10.0
	Rear Wall	33.0	28.0	2.0	1.5	10.0
	Side Wall	20.0	14.0	2.0	- (2)	10.0

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

(2) Remains elastic.

(3) Allowable ductility for a two-way reinforced concrete slab is shown for comparison.

(4) Largest side of trapezoidal yield-line pattern.

(5) Base width of tapering cantilever strip.

TABLE 8

MAXIMUM FOUNDATION BEARING PRESSURES[1]

Structure	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
NSCW Tower	8.8	-2.4	34.2	23.1	20.6	30.9	- (4)	2.7
NSCW Valve House	3.2	0.6	12.6	10.0	44.5	66.8	222.5	13.4

(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.

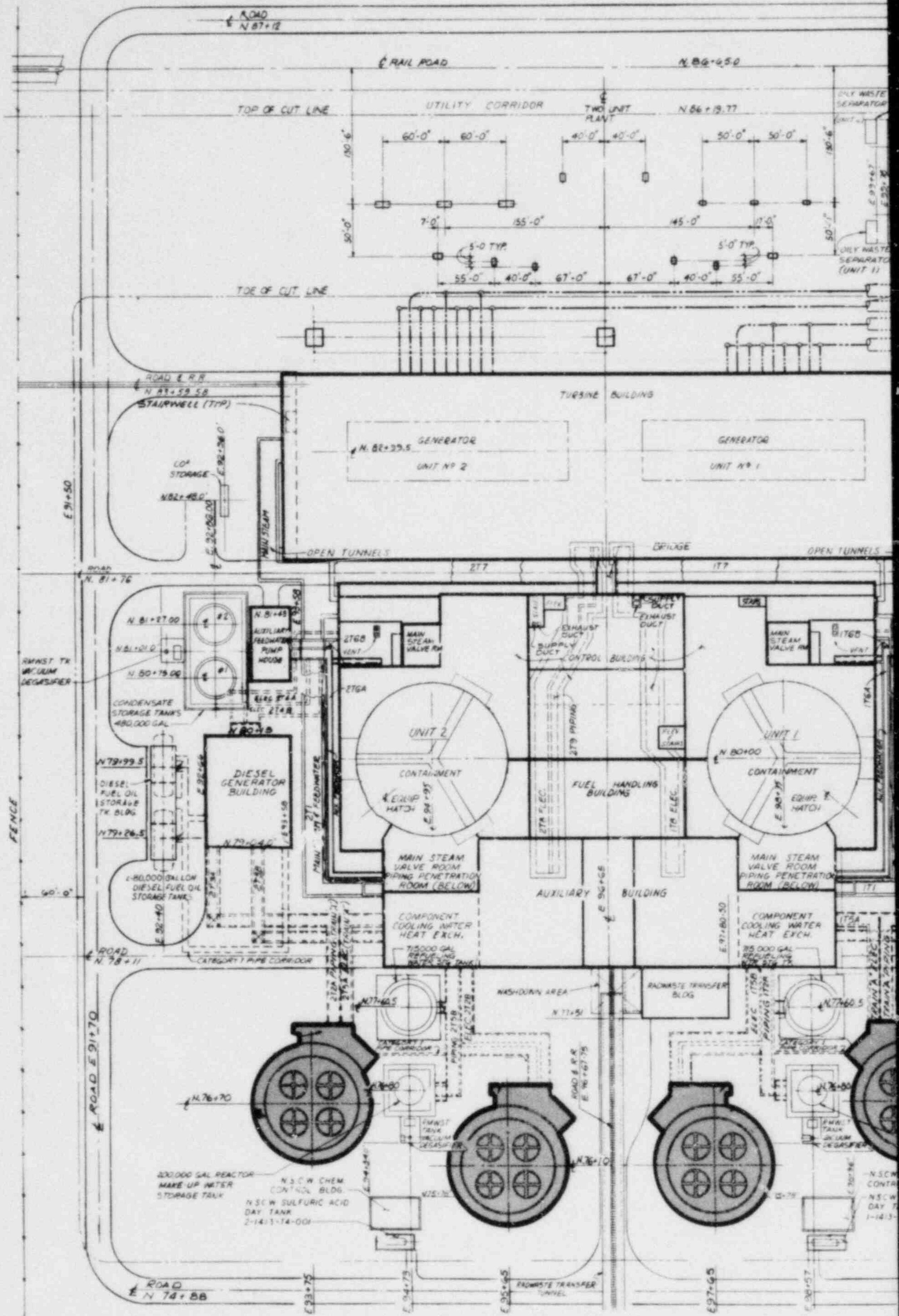
Gross Dynamic = Maximum soil pressure under dynamic loading conditions (i.e., unfactored SSE).

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.

(4) The static factor of safety is not applicable since the net static bearing pressure is negative.

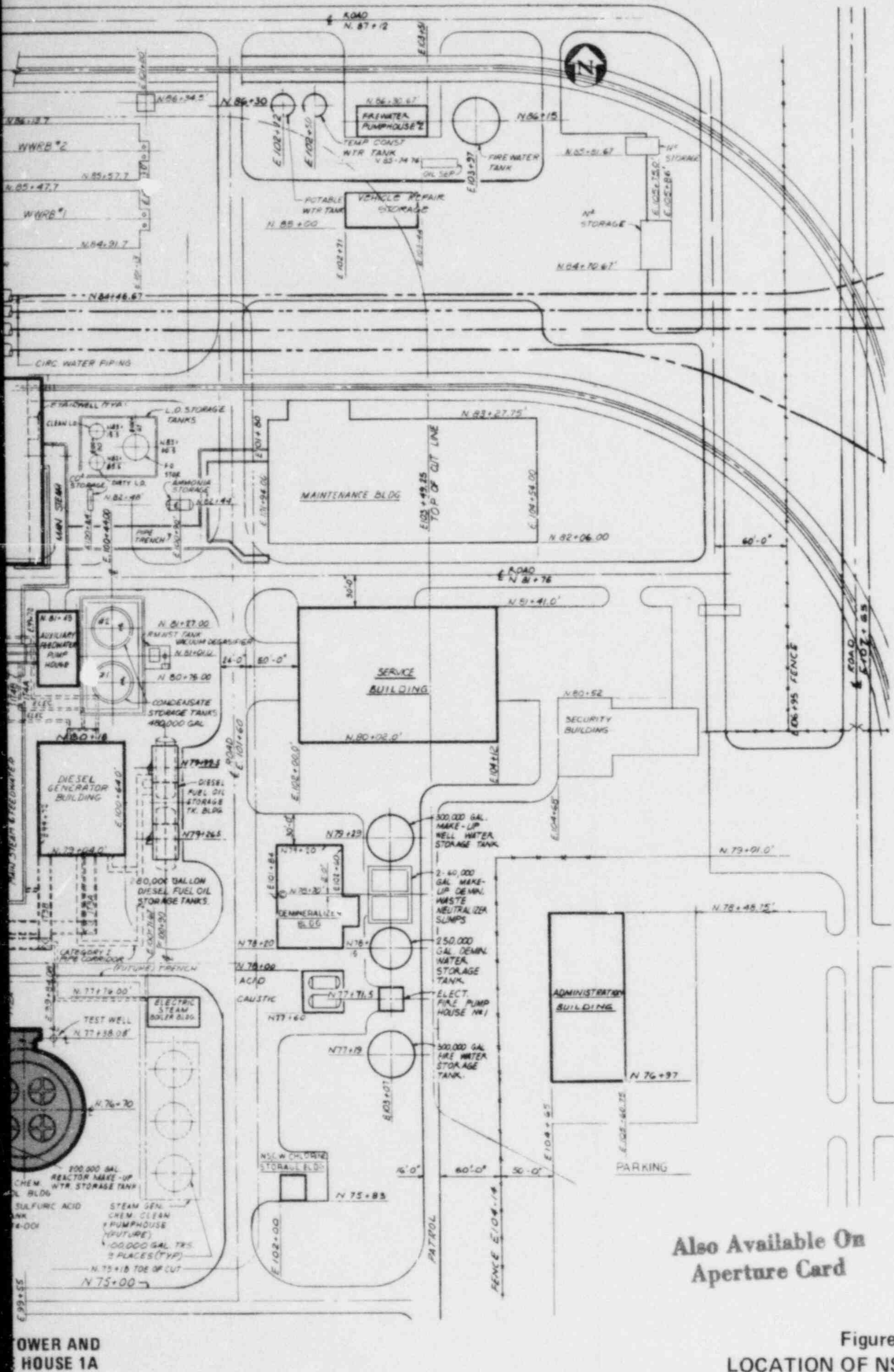


NSCW TOWER AND VALVE HOUSE 2A

NSCW TOWER AND VALVE HOUSE 2B

NSCW TOWER AND VALVE HOUSE 1B

NSCW T[illegible] VALVE[illegible]



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APERTURE
CARD

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Aperture Card

Figure 1
LOCATION OF NSCW TOWERS
AND VALVE HOUSES

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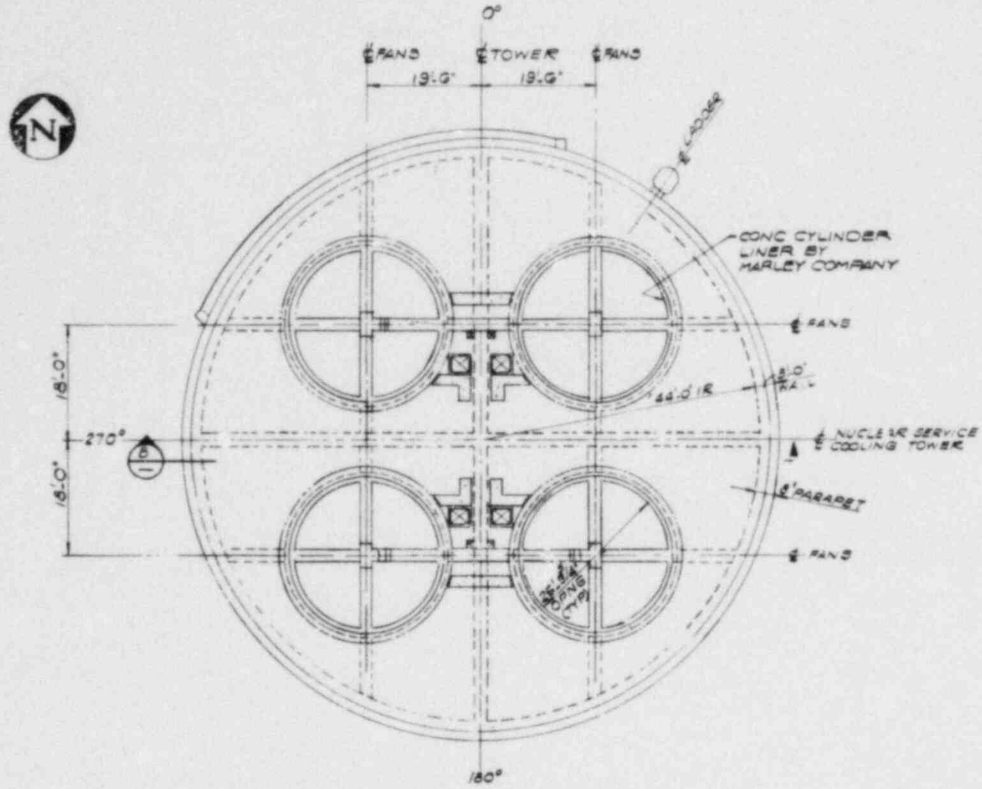
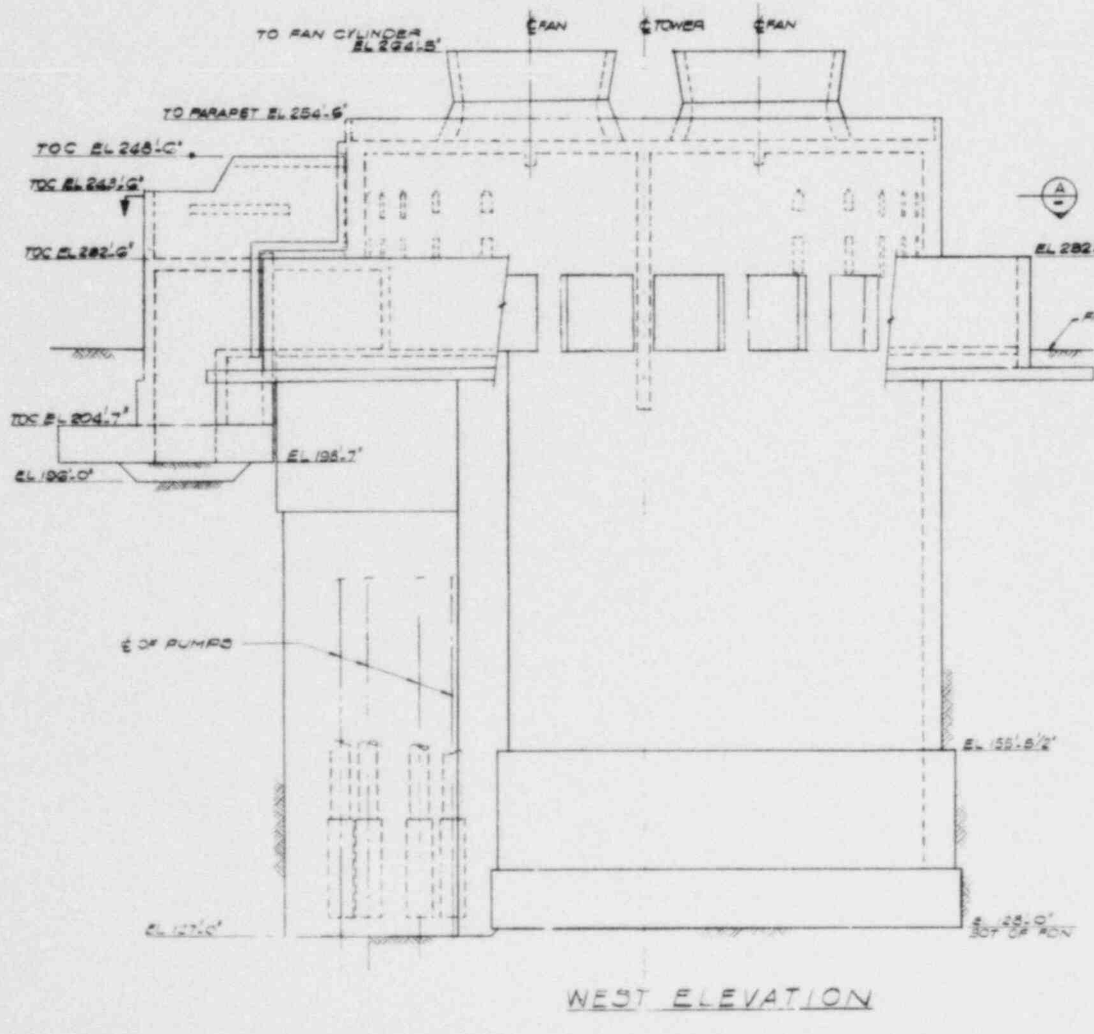
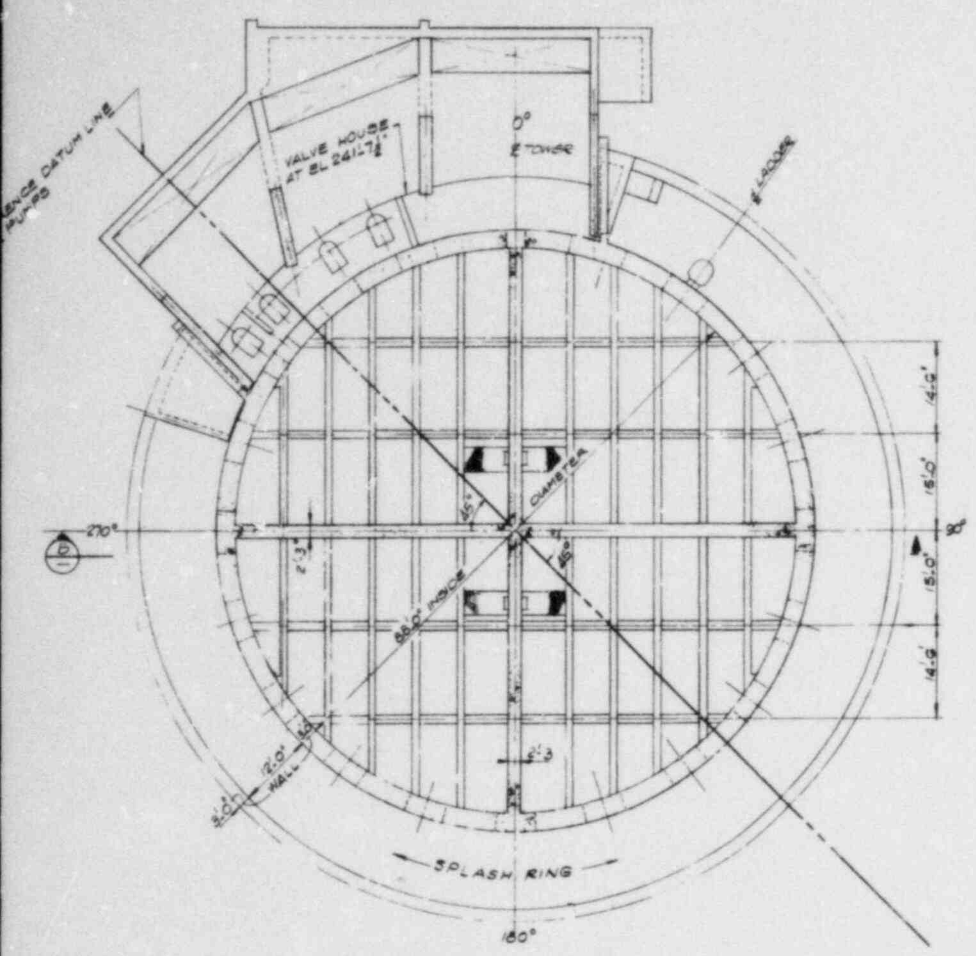


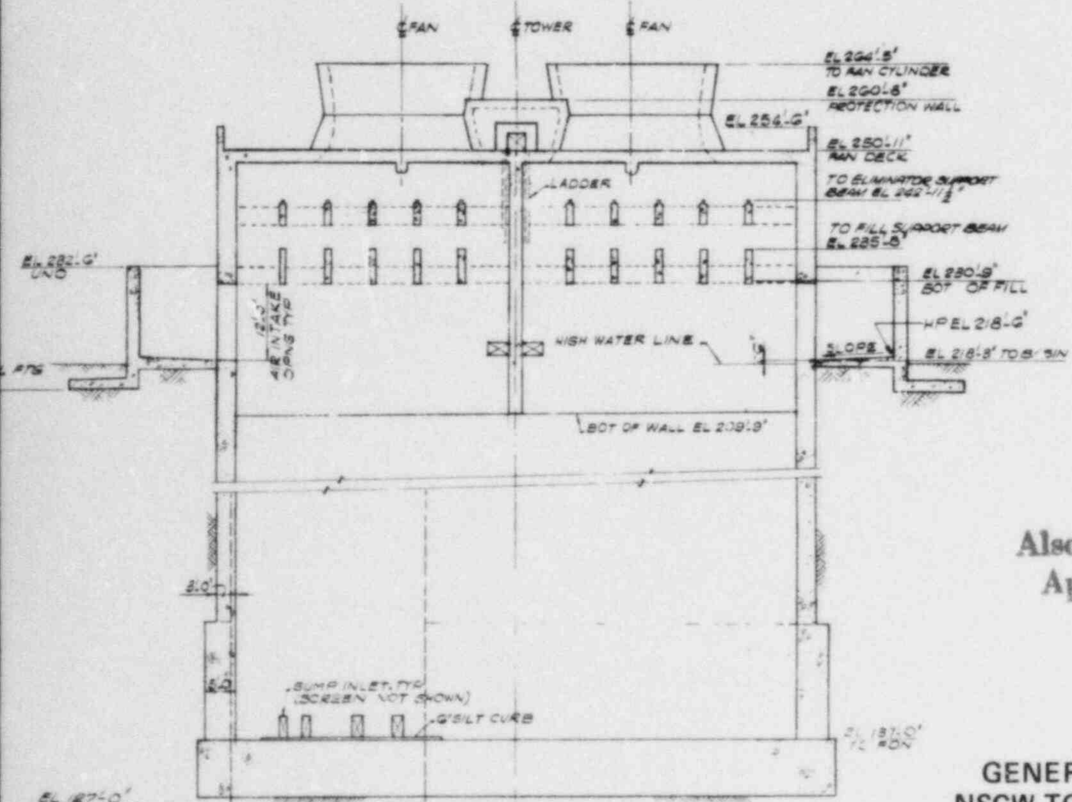
PLAN AT FAN DECK
EL 250-11' AND ABOVE



WEST ELEVATION



PLAN SECTION A
EL 242'-11½" AND BELOW



SECTION B

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Figure 2
GENERAL ARRANGEMENT OF
NSCW TOWER AND VALVE HOUSE
(Sheet 1 of 2)

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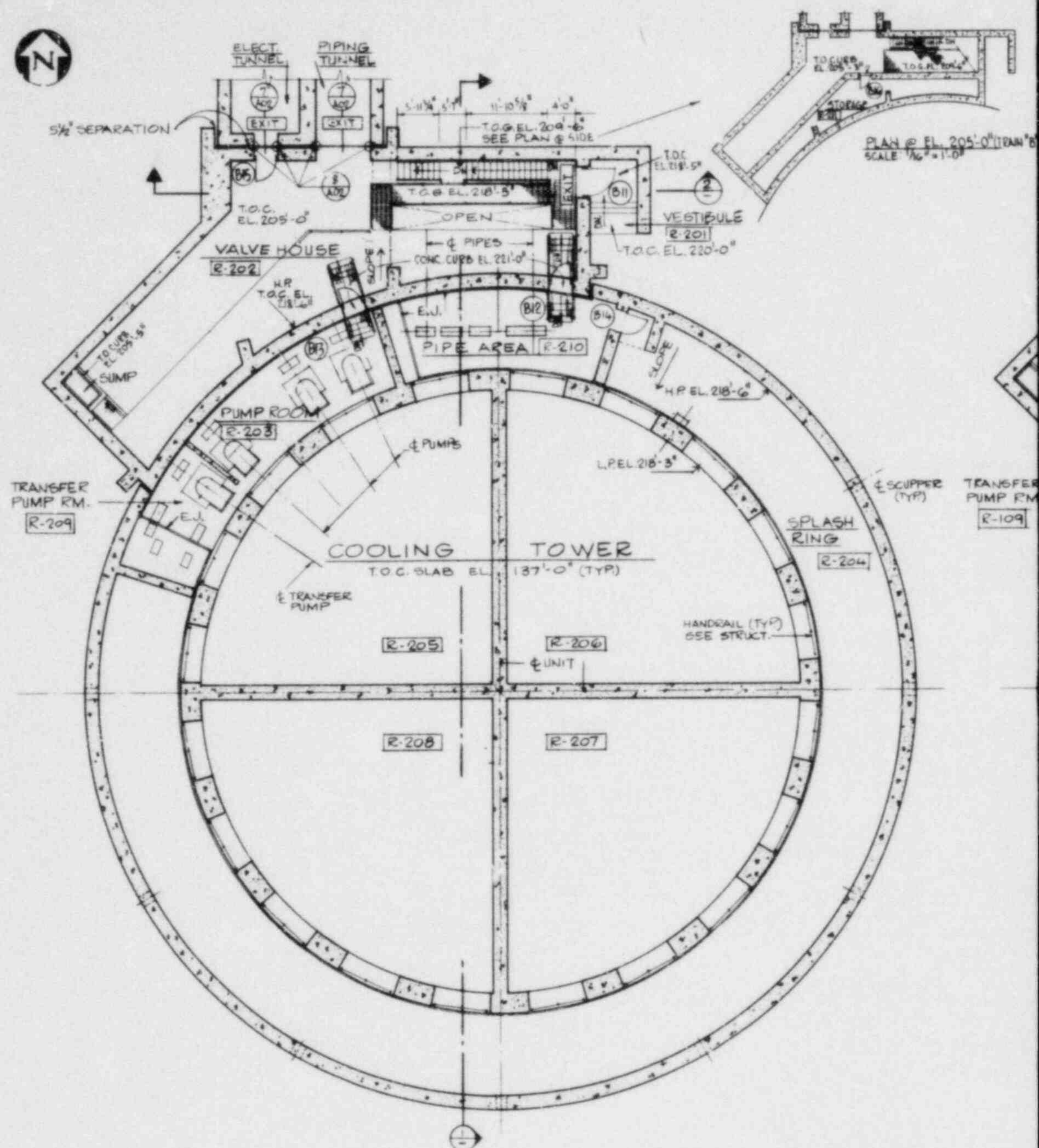
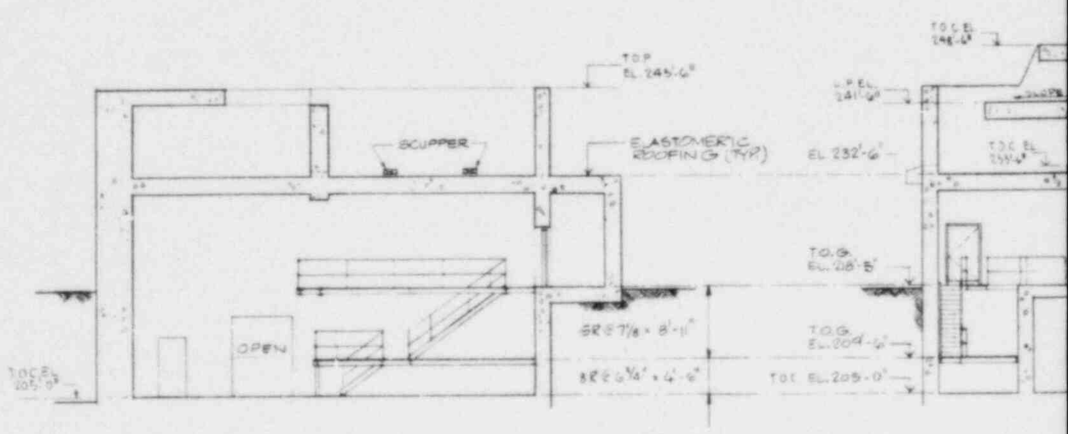


PLAN - GROUND LEVEL - TRAIN "B"



SECTION 2

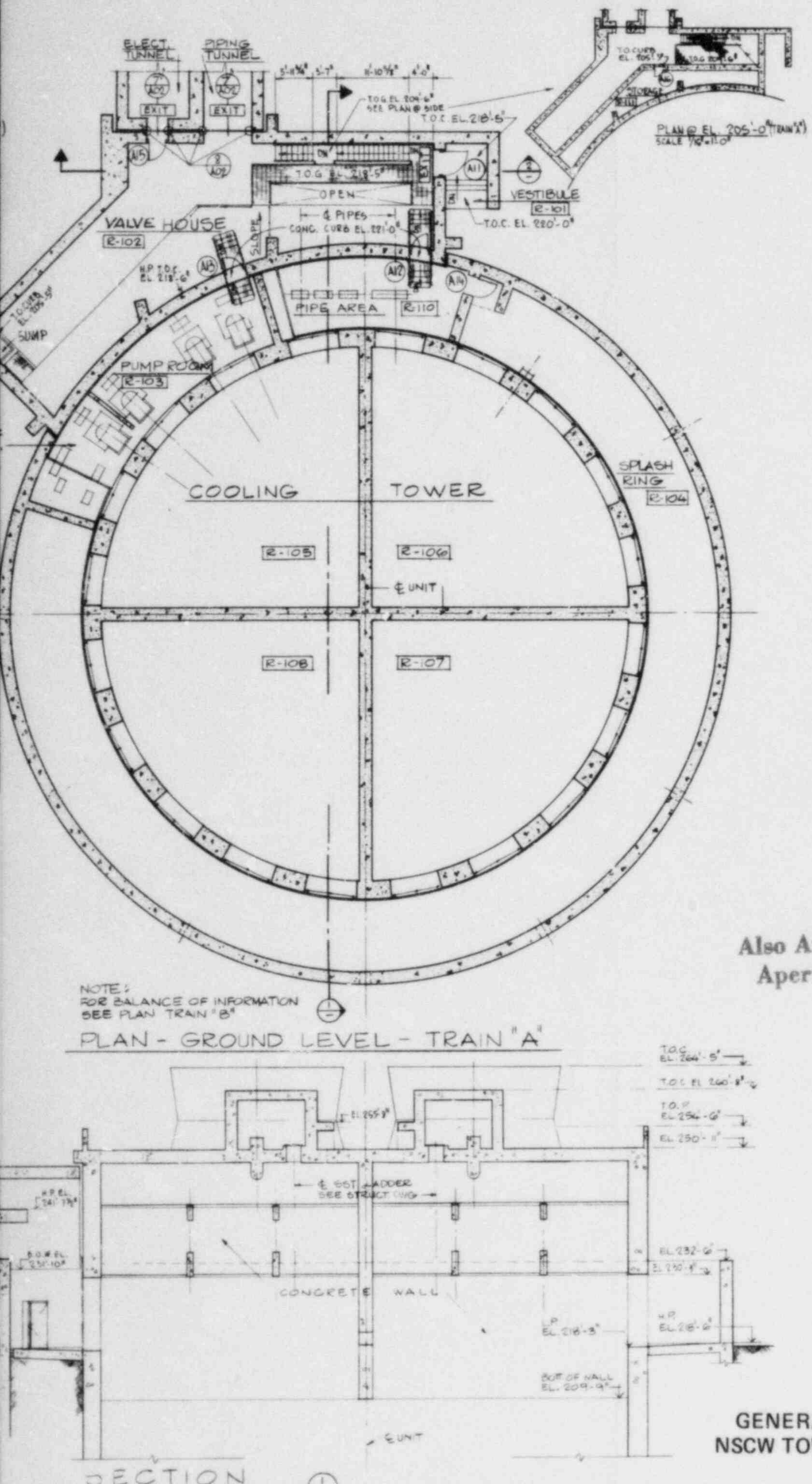


Figure 2
GENERAL ARRANGEMENT OF
NSCW TOWER AND VALVE HOUSE
(Sheet 2 of 2)

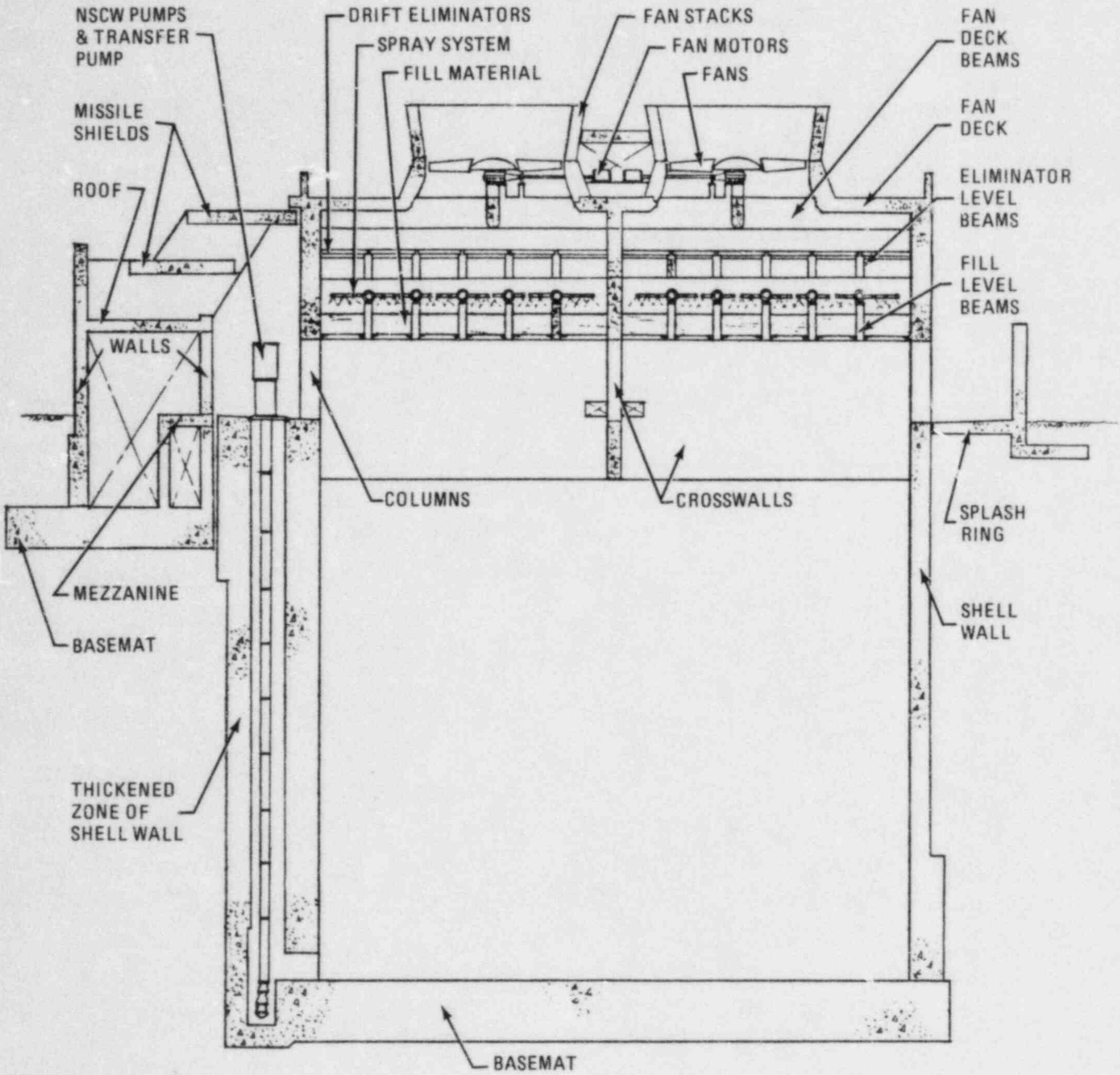


Figure 3
KEY STRUCTURAL ELEMENTS AND
MAJOR EQUIPMENT
(Sheet 1 of 2)

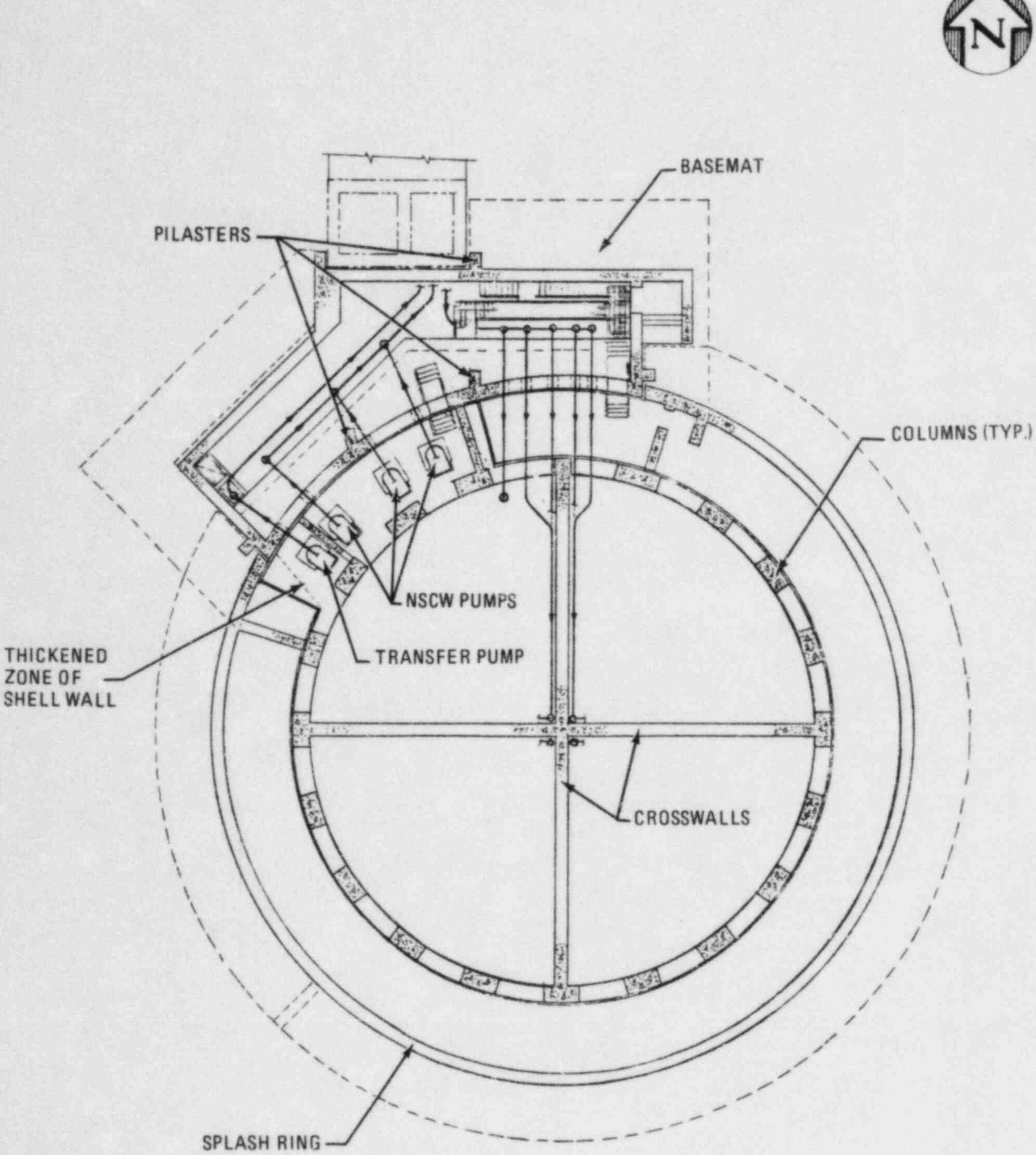


Figure 3
KEY STRUCTURAL ELEMENTS AND
MAJOR EQUIPMENT
(Sheet 2 of 2)

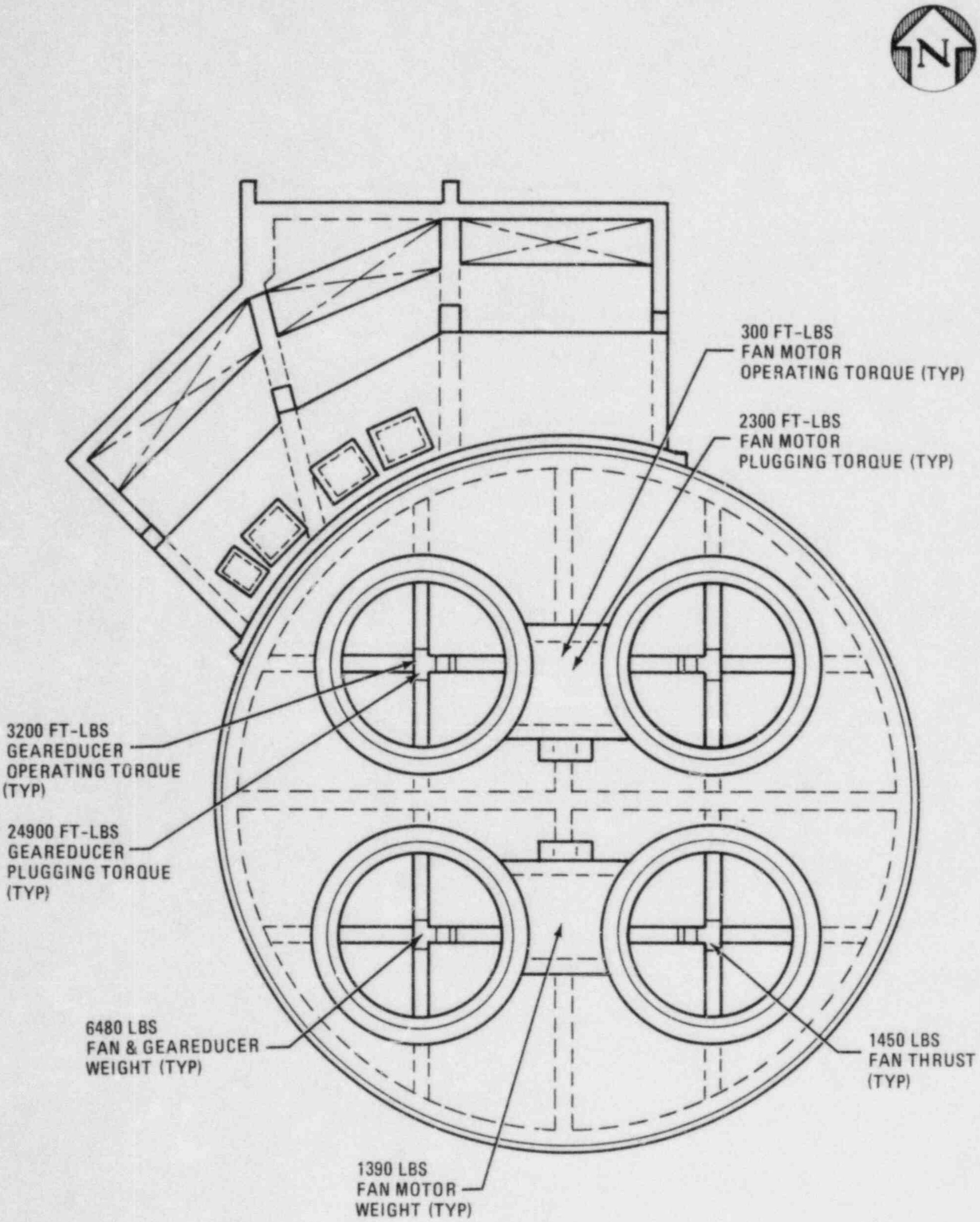
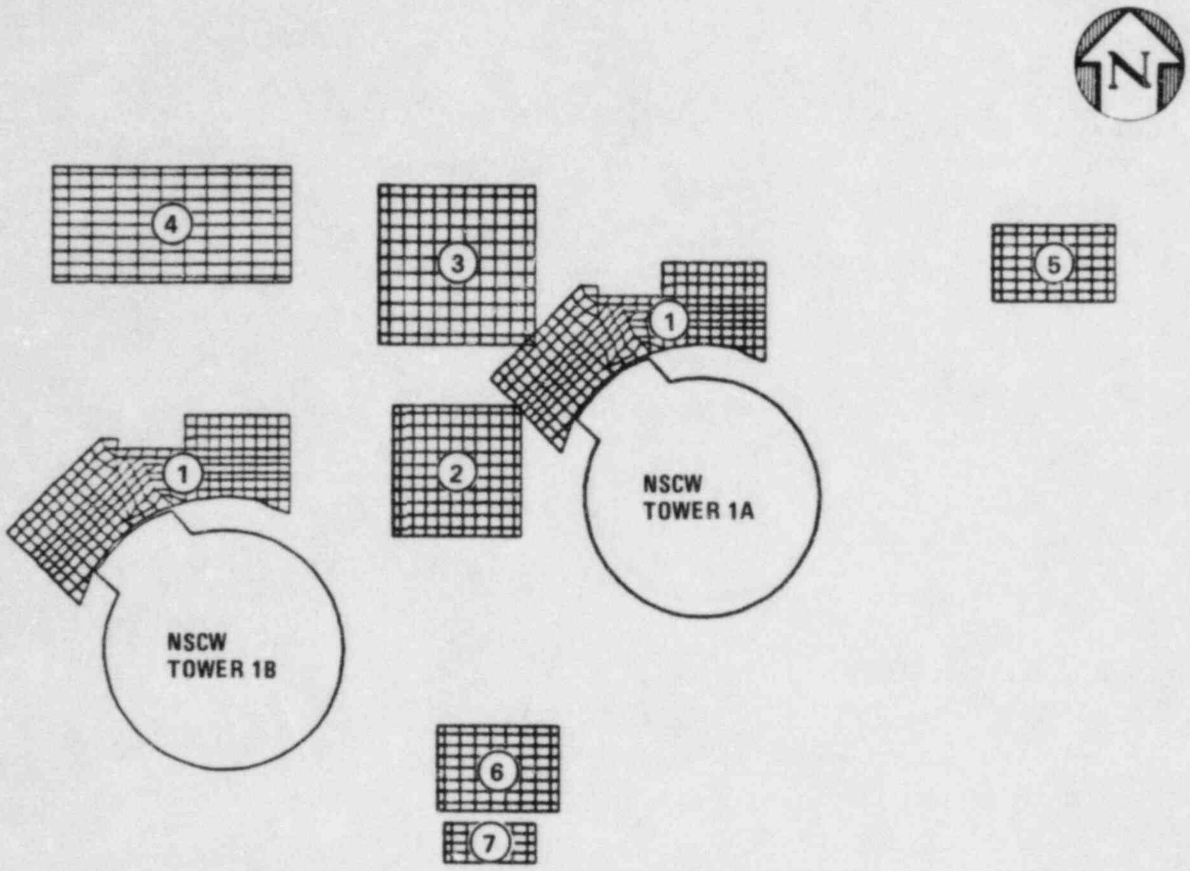


Figure 4
EQUIPMENT LOADS
AT FAN DECK LEVEL

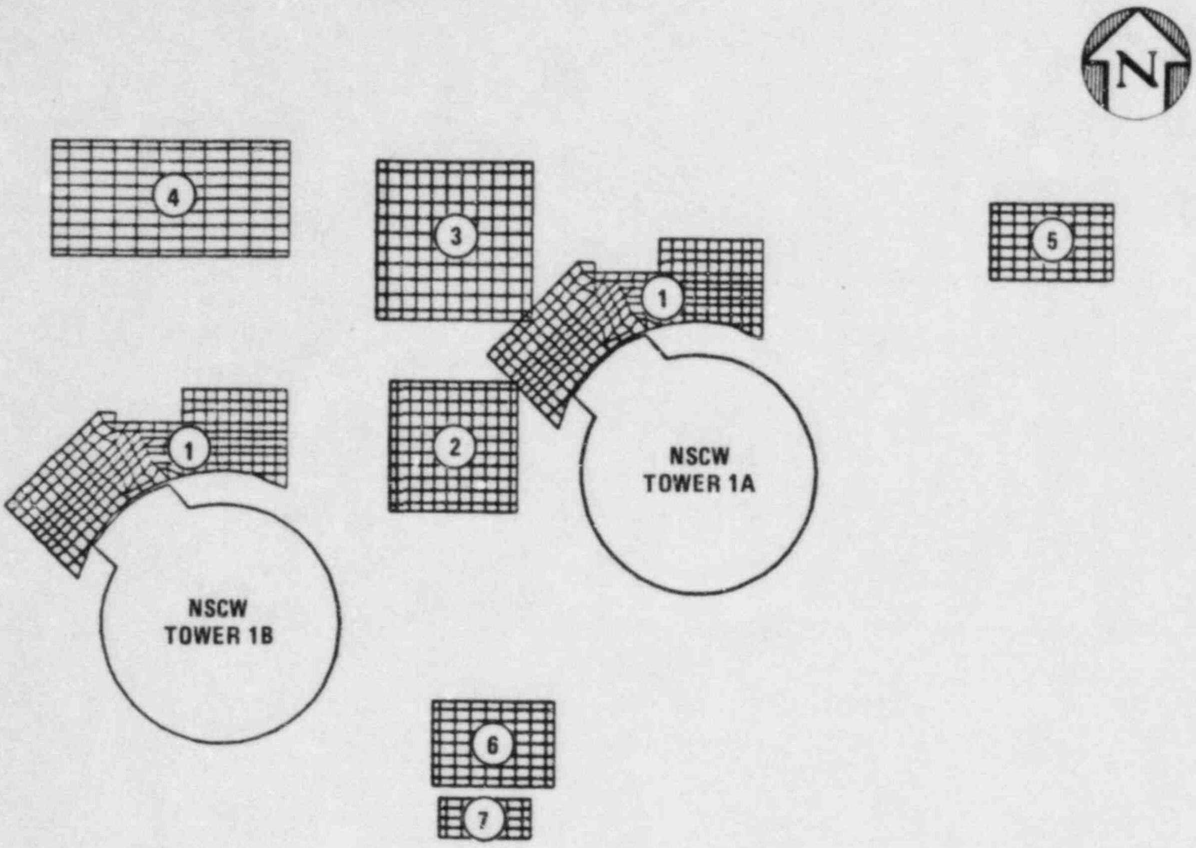


BUILDING		STATIC SURCHARGE LOAD (KSF) (1)
①	NSCW VALVE HOUSE	$P_1 = 4.1, P_2 = 3.1$ (2)
②	REACTOR MAKEUP WATER STORAGE TANK	P = 2.4 (3)
③	REFUELING WATER STORAGE TANK	P = 3.8 (3)
④	RADWASTE TRANSFER BUILDING	P = 3.6 (3)
⑤	ELECTRIC STEAM BOILER BUILDING	P = 1.1 (3)
⑥	NSCW CHEMICAL CONTROL BUILDING	P = 0.6 (3)
⑦	NSCW SULFURIC ACID STORAGE TANK	P = 0.7 (3)

NOTES:

1. STATIC CONDITION SURCHARGE LOADS ARE BASED ON FULL BUILDING DEAD LOAD PLUS LIVE LOAD
2. VALVE HOUSE PRESSURE DIAGRAM IS TRAPEZOIDAL WITH P_1 BEING THE PRESSURE AT THE CORNERS OF THE BASEMAT CLOSEST TO THE TOWERS AND P_2 BEING THE PRESSURES AT THE OTHER TWO CORNERS
3. UNIFORM PRESSURE ASSUMED OVER WHOLE MAT

Figure 5
BUILDING SURCHARGE LOADS
(Sheet 1 of 2)

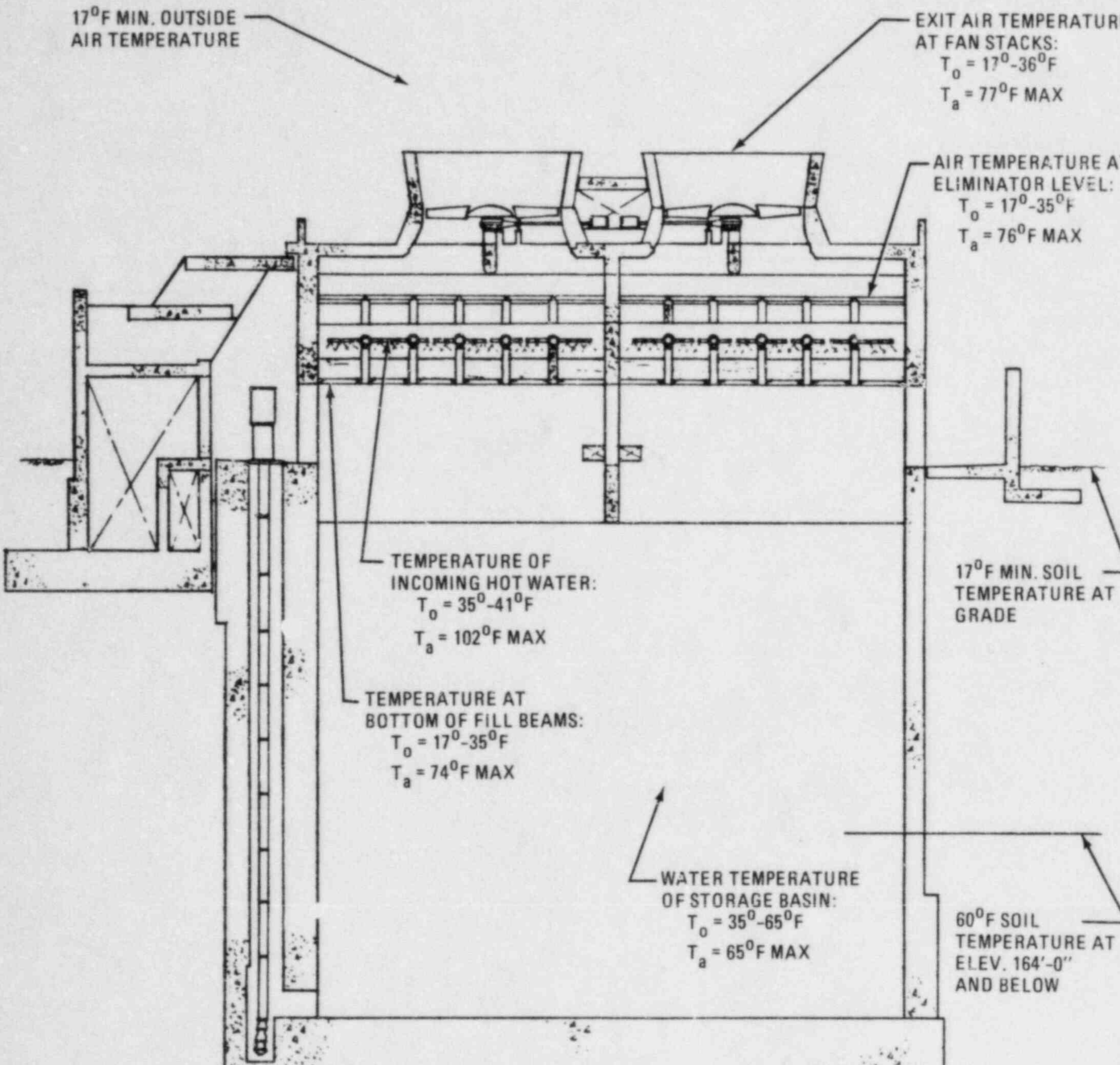


BUILDING		OBE SURCHARGE LOAD (KSF) (1)				SSE SURCHARGE LOAD (KSF) (1)			
		P_1	P_2	P_3	P_4	P_1	P_2	P_3	P_4
1	NSCW VALVE HOUSE	7.6	6.7	3.8	2.5	10.2	8.8	4.2	2.1
2	REACTOR MAKE UP WATER STORAGE TANK	3.7	2.7	2.7	1.8	4.5	3.0	3.0	1.4
3	REFUELING WATER STORAGE TANK	7.4	4.5	4.5	1.5	9.6	4.9	4.9	0.2
4	RADWASTE TRANSFER BUILDING	4.9	4.3	3.3	2.6	6.0	4.9	3.3	2.1
5	ELECTRIC STEAM BOILER BUILDING	1.1	1.0	0.9	0.8	1.2	1.1	0.9	0.8
6	NSCW CHEMICAL CONTROL BUILDING	0.6	0.5	0.5	0.4	0.6	0.6	0.5	0.4
7	NSCW SULFURIC ACID STORAGE TANK	0.9	0.8	0.7	0.6	1.1	1.0	0.7	0.5

NOTE:

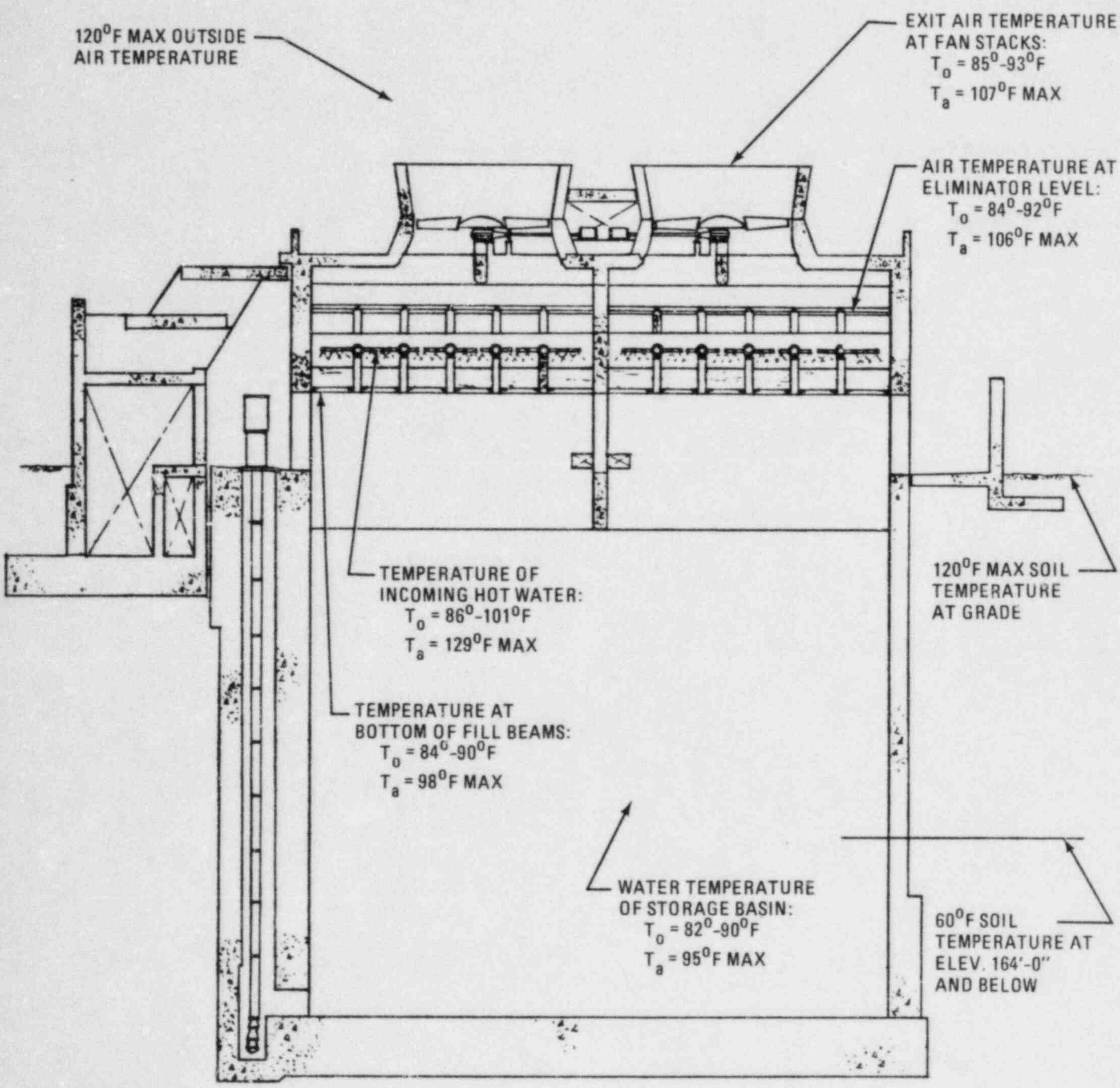
1. PRESSURE DIAGRAMS FOR SEISMIC CASES ARE TRAPEZOIDAL WITH P_1 BEING THE PEAK CORNER PRESSURE LOCATED AT THE BASEMAT CORNER CLOSEST TO THE TOWER. THE PRESSURES P_2, P_3, P_4 ARE THE PRESSURES AT THE OTHER CORNERS. THESE PRESSURE DIAGRAMS ARE BASED ON FULL DEAD LOAD AND 25% OF LIVE LOAD COMBINED WITH 100% OF THE DOWNWARD SEISMIC INERTIA LOADS AND 40% OF THE HORIZONTAL INERTIA LOADS.

Figure 5
BUILDING SURCHARGE LOADS
(Sheet 2 of 2)



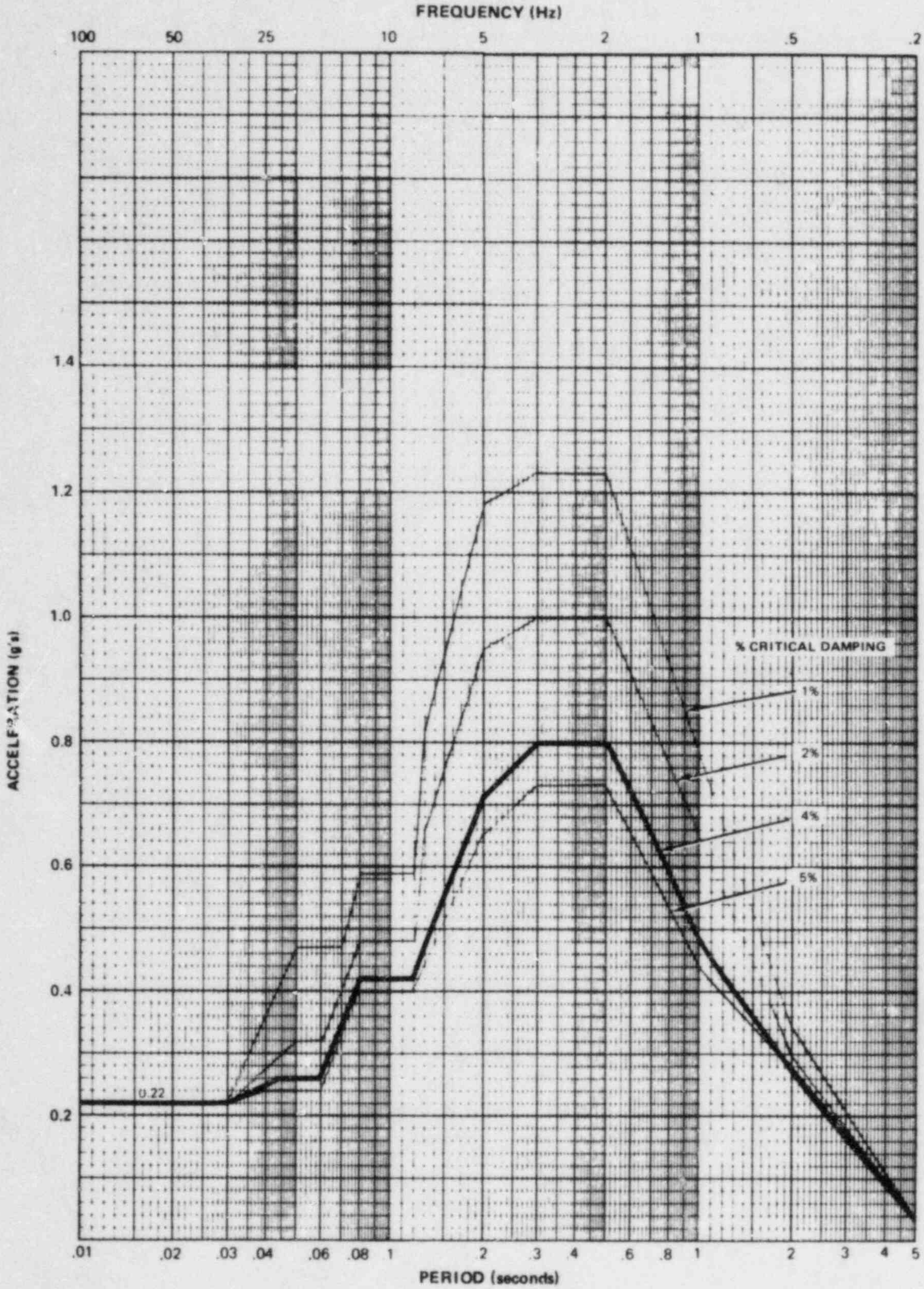
WINTER CONDITION

Figure 6
DESIGN TEMPERATURES
(Sheet 1 of 2)



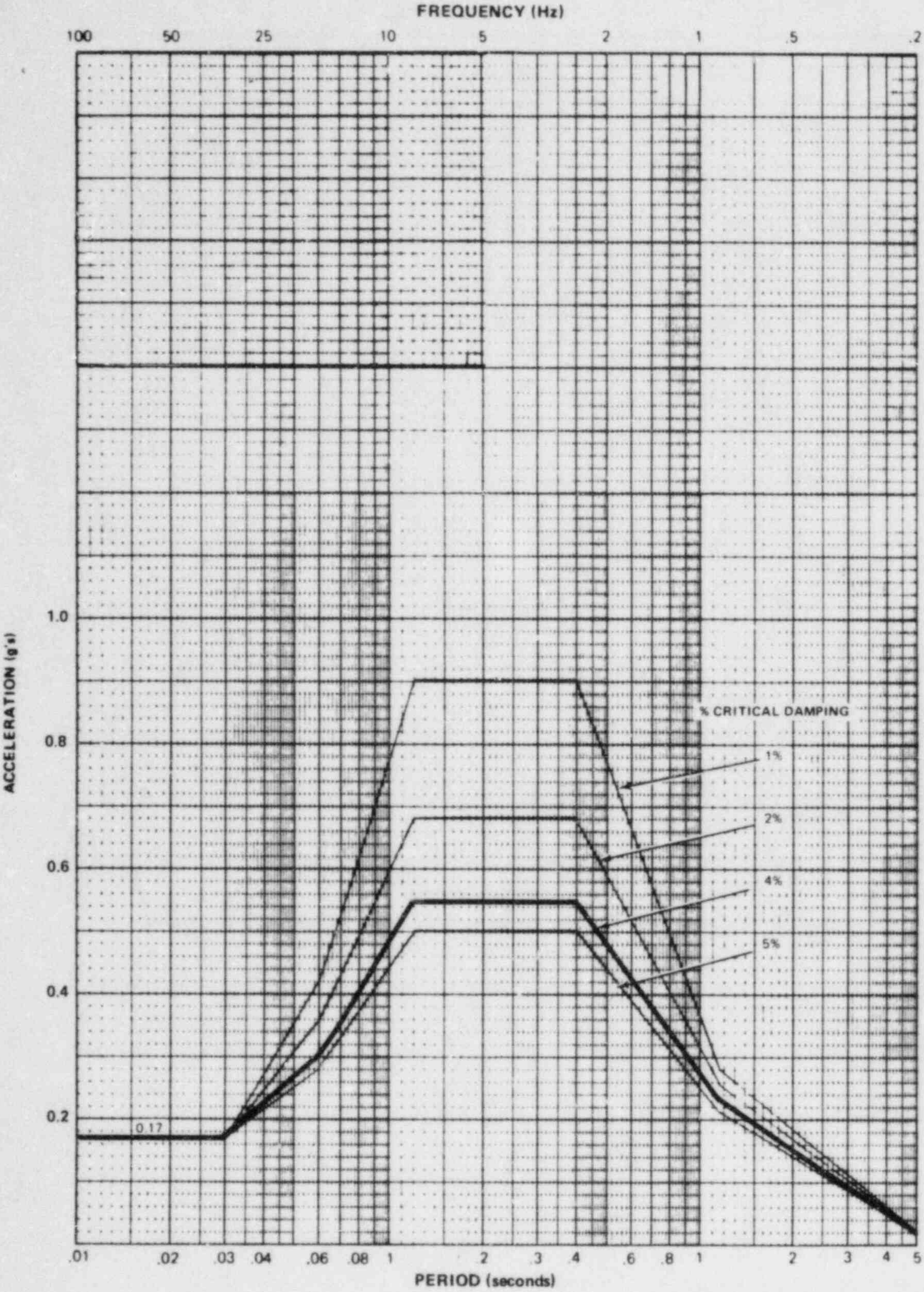
SUMMER CONDITION

Figure 6
DESIGN TEMPERATURES
(Sheet 2 of 2)



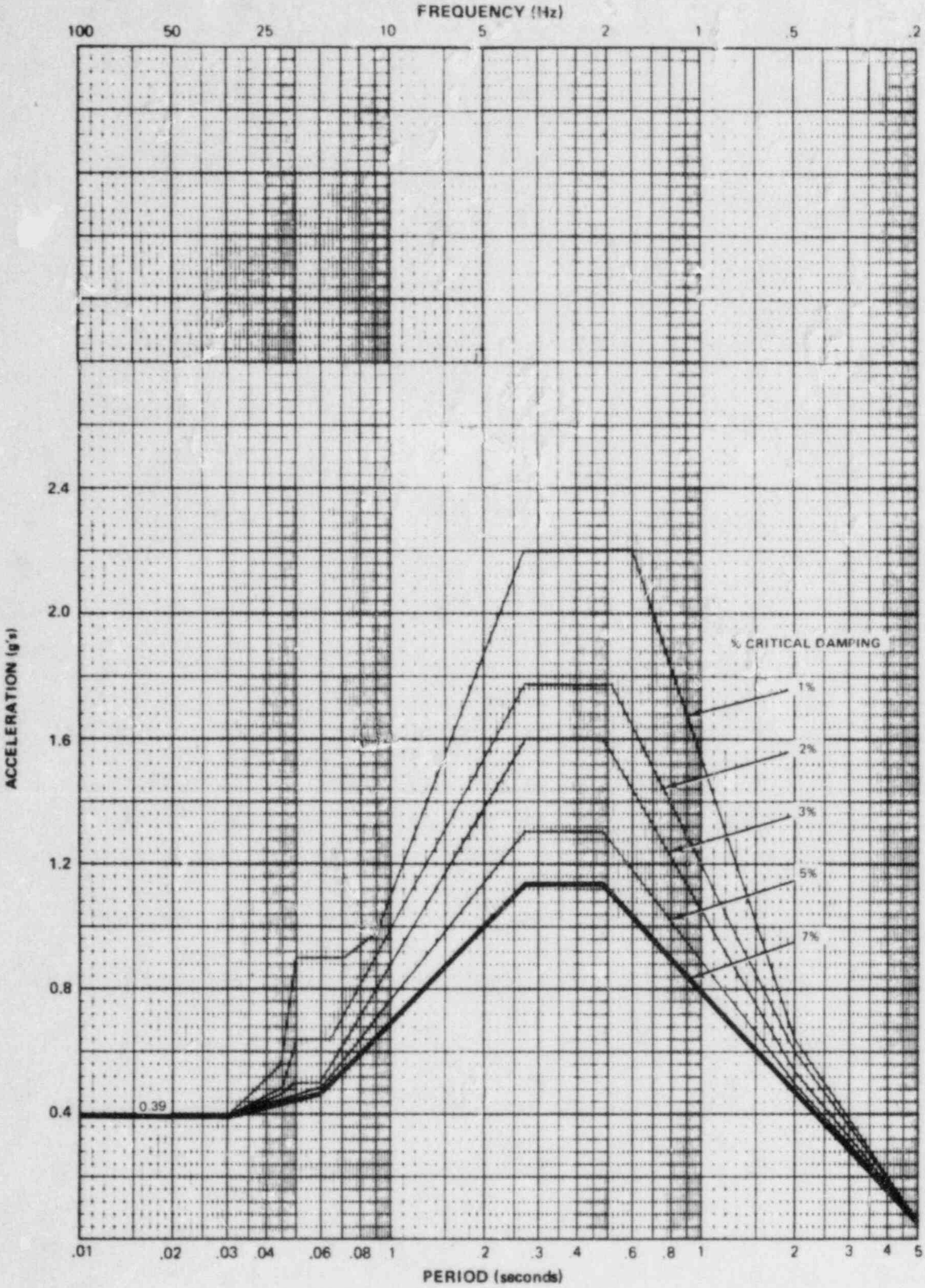
HORIZONTAL DIRECTION

Figure 7
OBE DESIGN SPECTRA FOR NSCW VALVE HOUSE
(Sheet 1 of 2)



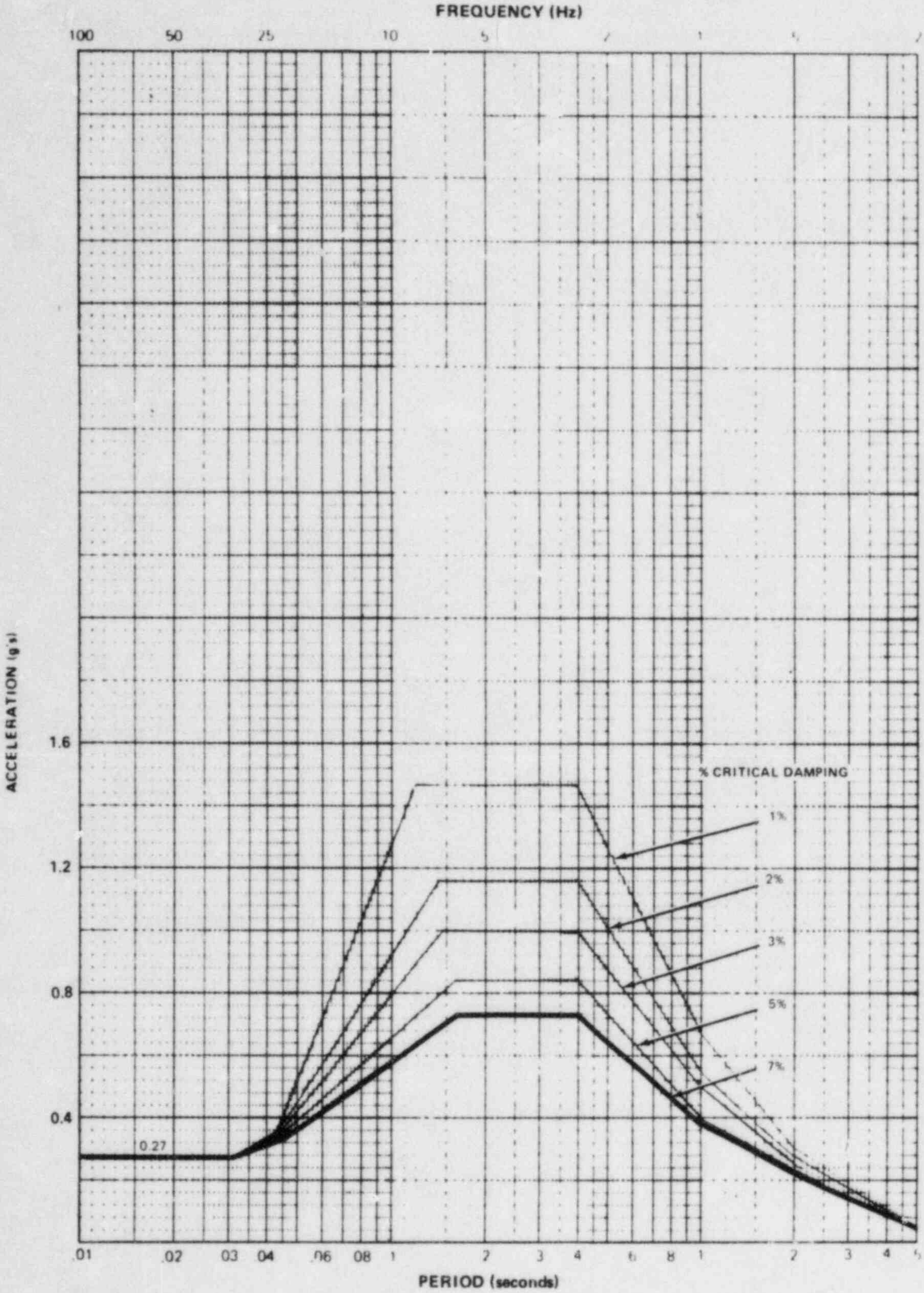
VERTICAL DIRECTION

Figure 7
OBE DESIGN SPECTRA FOR NSCW VALVE HOUSE
(Sheet 2 of 2)



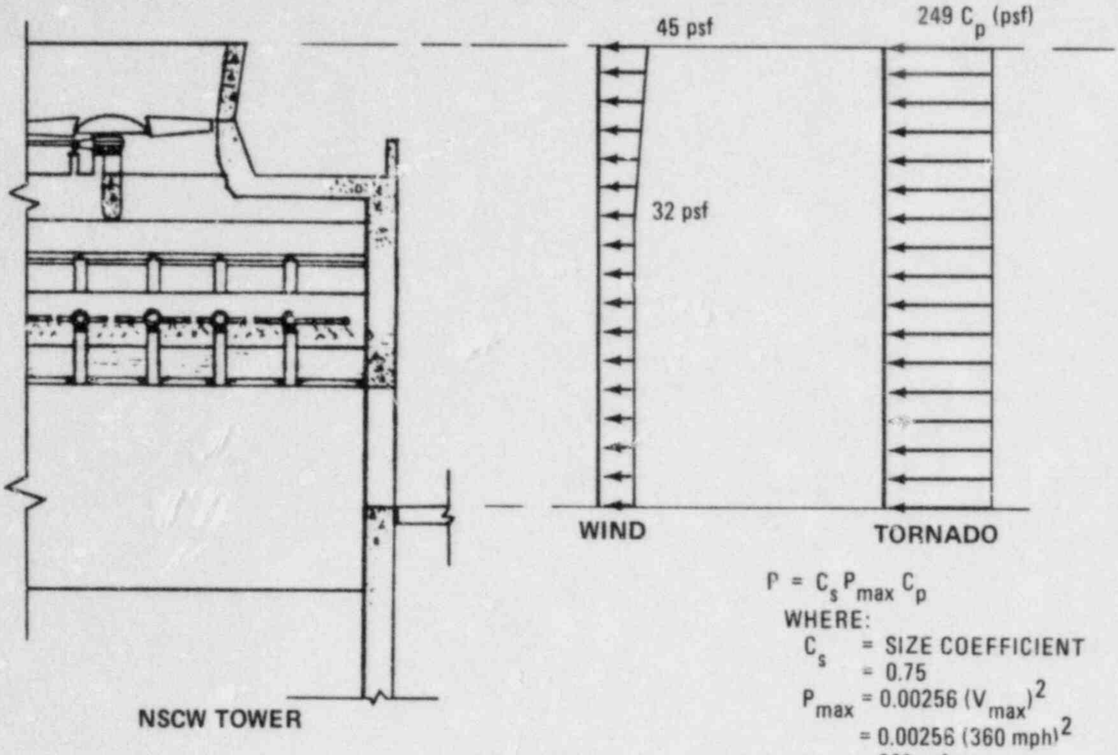
HORIZONTAL DIRECTION

Figure 8
SSE DESIGN SPECTRA FOR NSCW VALVE HOUSE
(Sheet 1 of 2)



VERTICAL DIRECTION

Figure 8
SSE DESIGN SPECTRA FOR NSCW VALVE HOUSE
(Sheet 2 of 2)



$P = C_s P_{max} C_p$

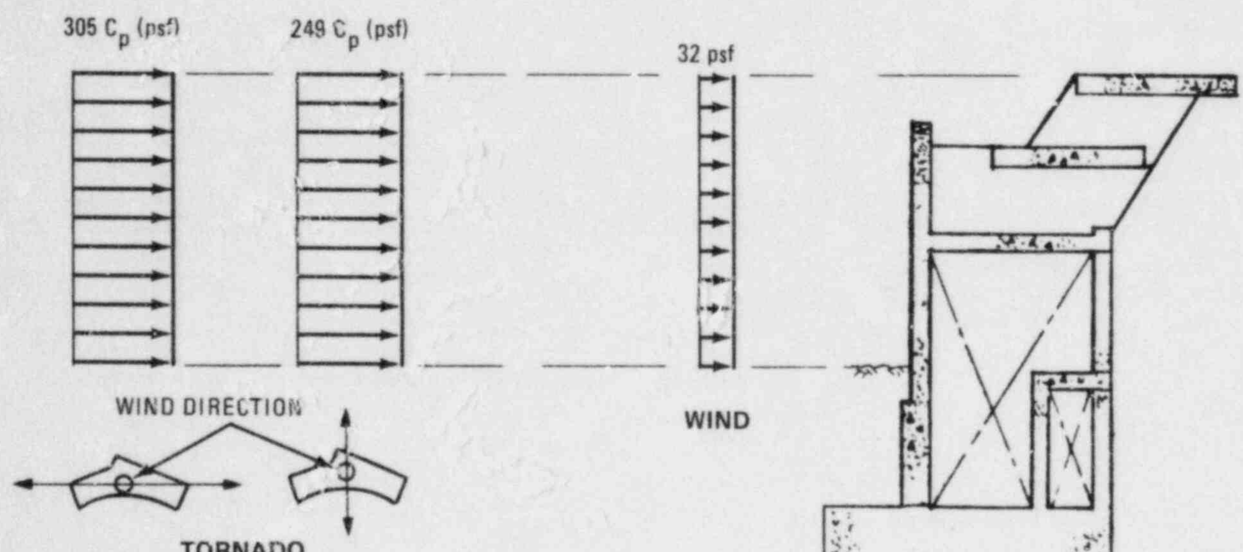
WHERE:

C_s = SIZE COEFFICIENT = 0.75

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

C_p = EFFECTIVE EXTERNAL PRESSURE COEFFICIENT

$P = 0.75 (332 \text{ psf}) C_p = 249 C_p$ (psf)



$P = C_s P_{max} C_p$

WHERE:

C_s = SIZE COEFFICIENT = 0.92 or 0.75

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

C_p = EFFECTIVE EXTERNAL PRESSURE COEFFICIENT

$P = (0.92 \text{ or } 0.75) (332 \text{ psf}) C_p = (305 \text{ or } 249) C_p$ (psf)

Figure 9
WIND AND TORNADO EFFECTIVE VELOCITY
PRESSURE PROFILES

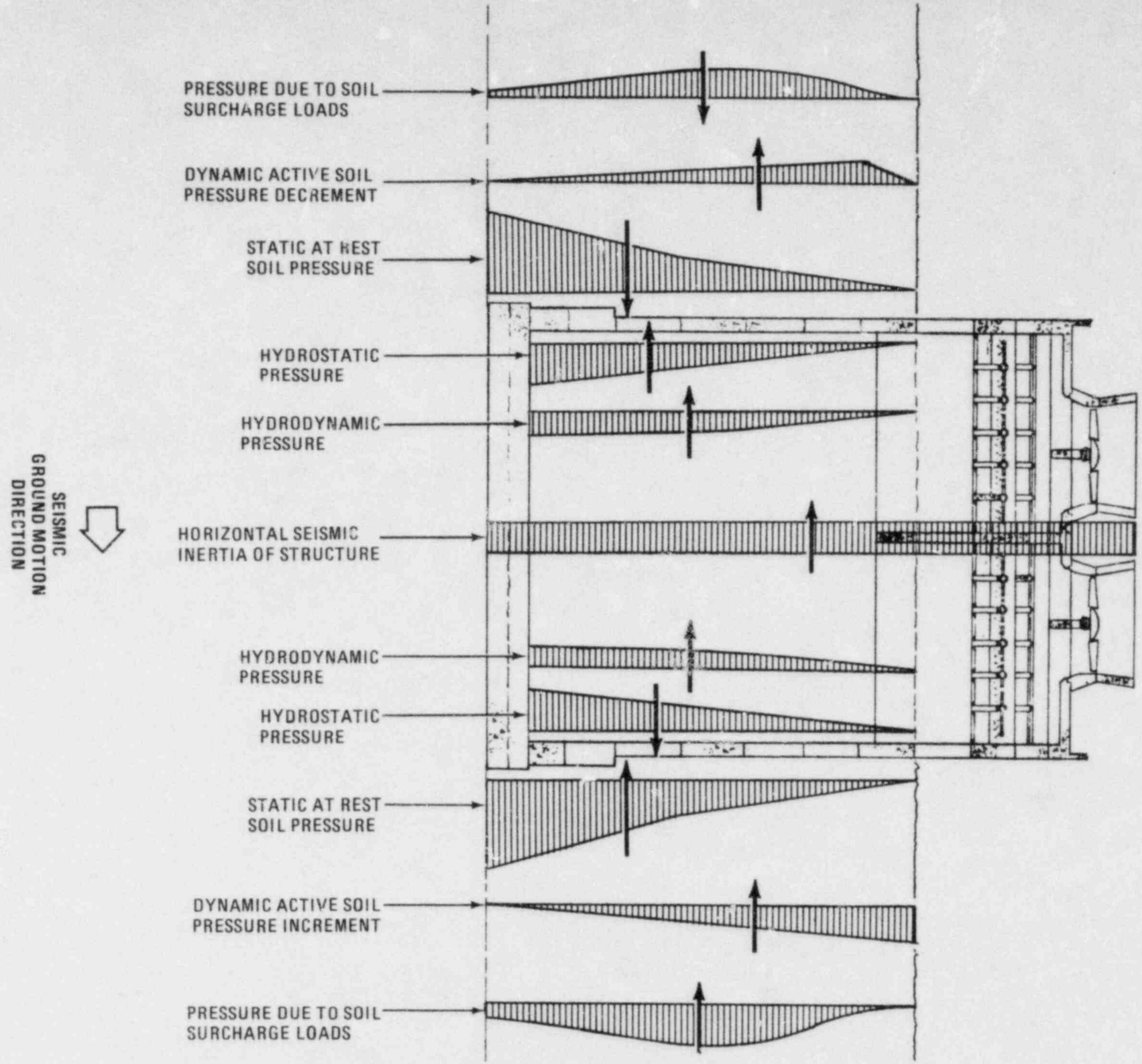


Figure 10
SUMMARY OF SEISMIC LOADS
ON NSCW TOWER

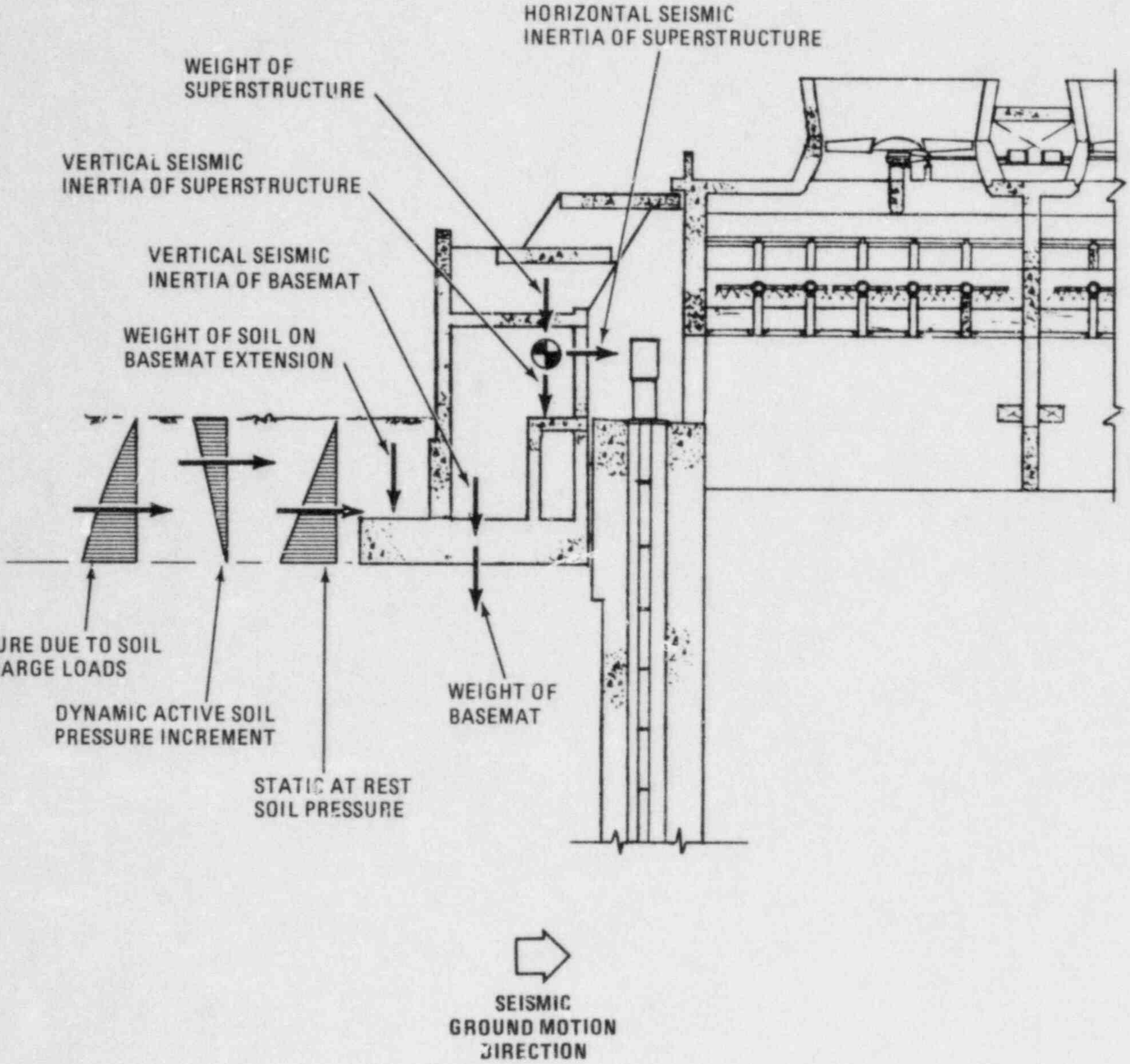


Figure 11
SUMMARY OF SEISMIC LOADS ON
NSCW VALVE HOUSE

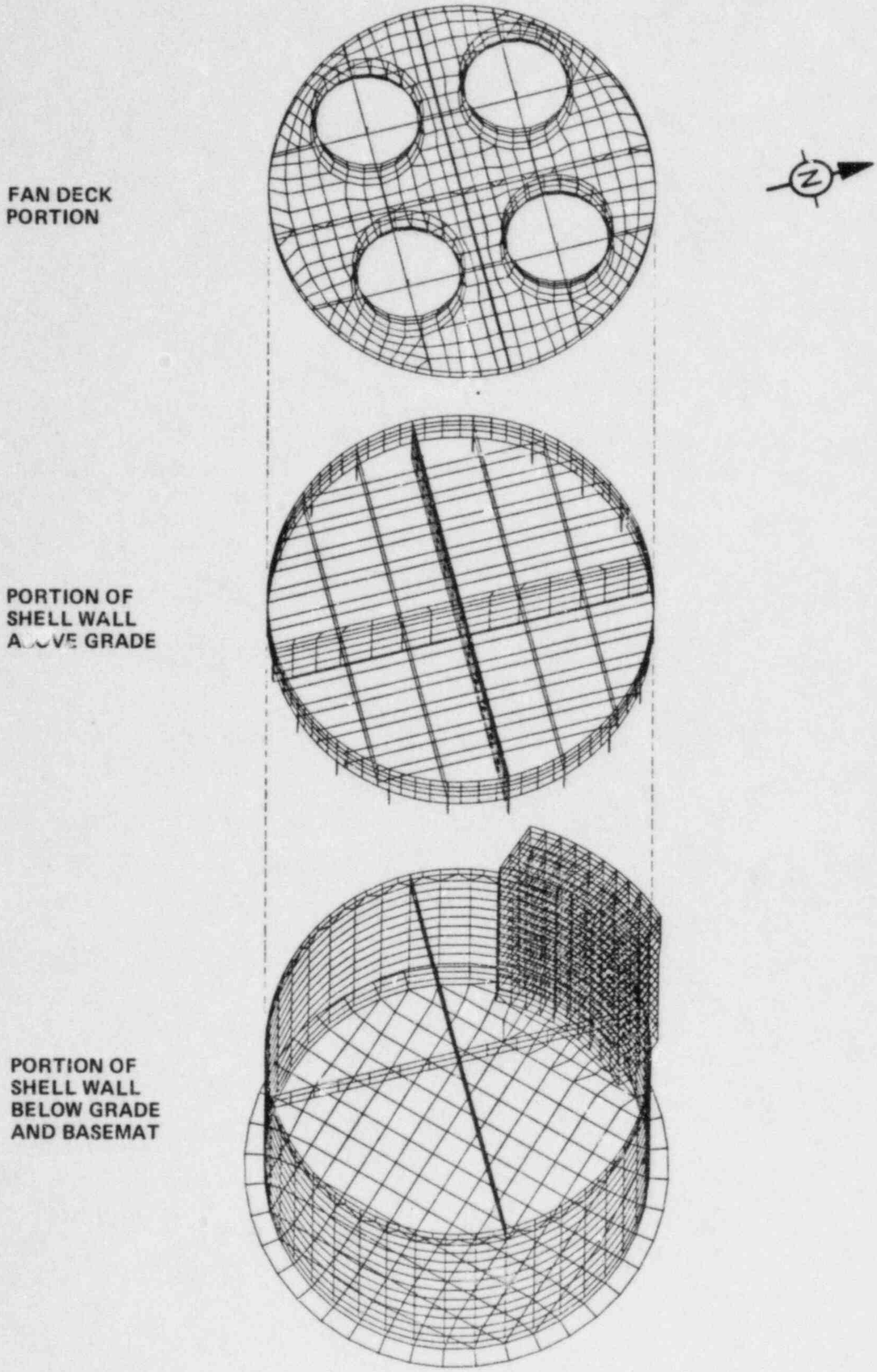
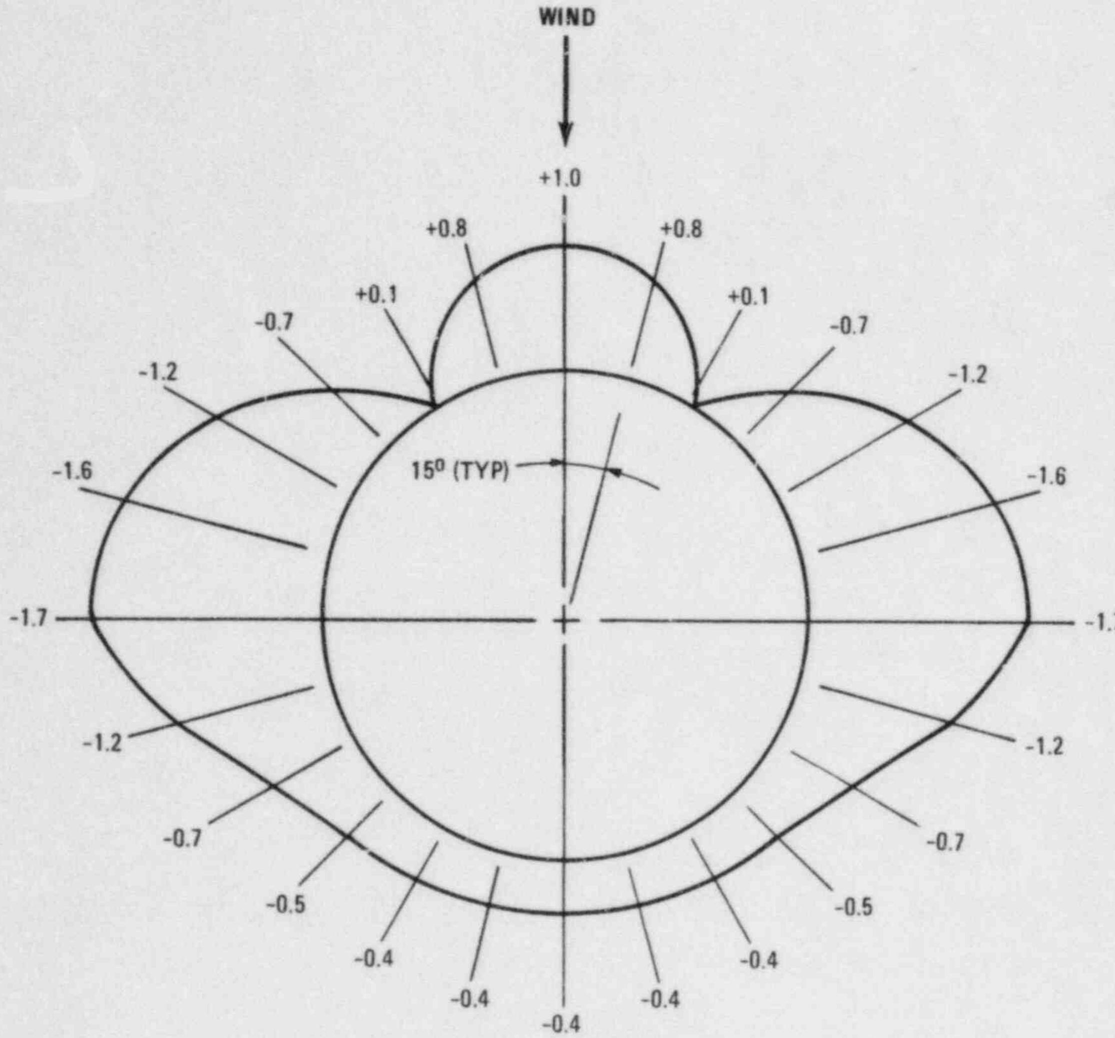


Figure 12
FINITE ELEMENT MODEL
OF NSCW TOWER



C_{pe} VALUES FOR SHELL WALL AND FAN STACKS

C_{pe} = -0.7 FOR FAN DECK SLAB

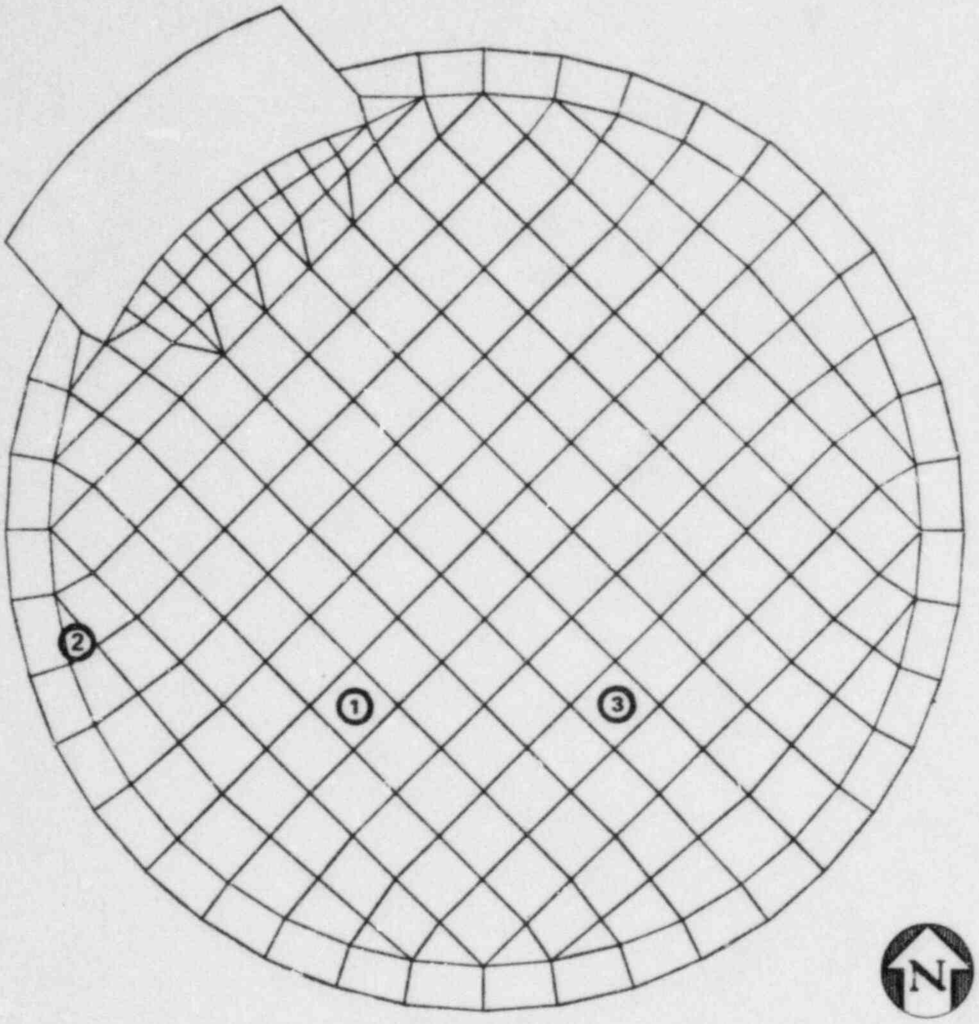
C_{pi} = +0.1 FOR CASE OF FANS OPERATING

C_{pi} = -0.8 FOR CASE OF FANS NOT OPERATING

C_{pe} = EXTERNAL PRESSURE COEFFICIENT

C_{pi} = INTERNAL PRESSURE COEFFICIENT

Figure 13
WIND PRESSURE COEFFICIENTS
FOR NSCW TOWER

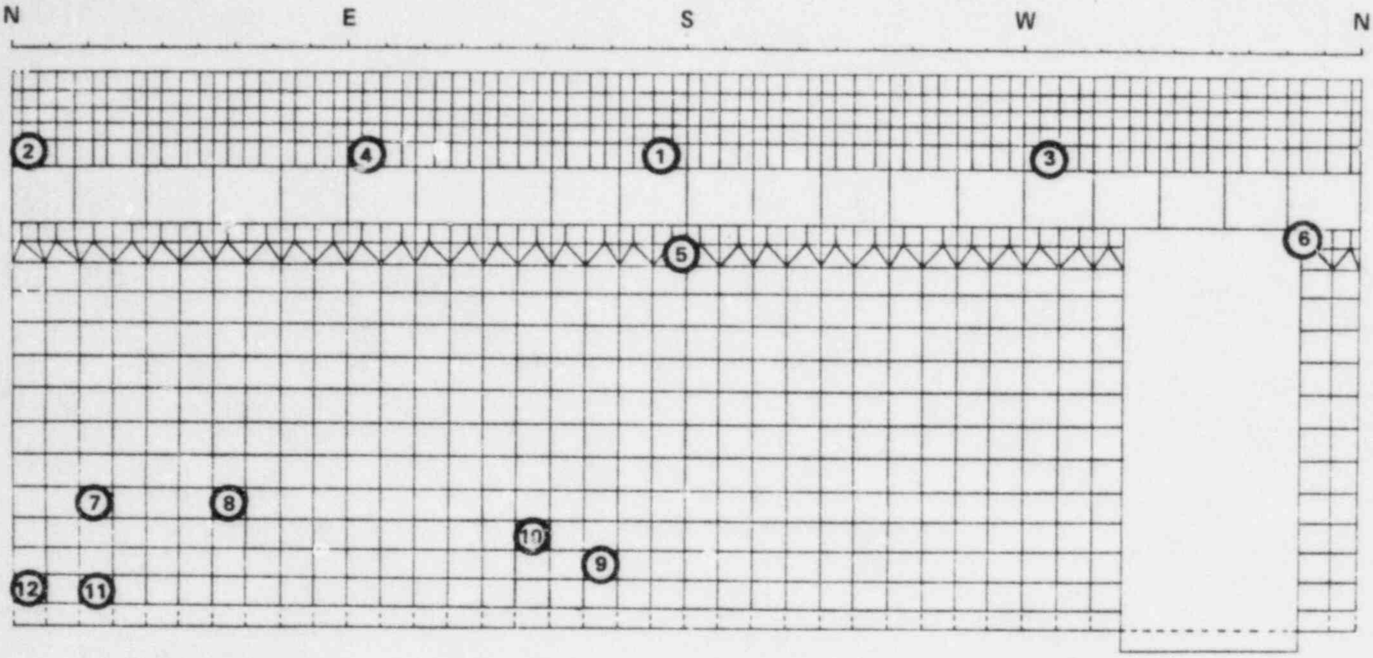


PLAN VIEW OF BASEMAT

REINFORCEMENT ORIENTATION	KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (4)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
			AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)
NE-SW	1	3	–	–1511	3.56	2.39 (3)	5.56	6.06
NE-SW	2	3	–	+1645	2.31 (3)	3.89	5.56	6.06
NW-SE	3	3	–	–1647	3.81	2.39 (3)	5.56	6.06
NW-SE	2	3	–	+1545	2.31 (3)	3.64 (3)	5.56	6.06

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE TOP FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 12", h = 108".
3. DESIGN IS GOVERNED BY MINIMUM REINFORCEMENT REQUIREMENTS.
4. SEE TABLE 4.

Figure 14
ANALYSIS AND DESIGN RESULTS
FOR NSCW TOWER BASEMAT



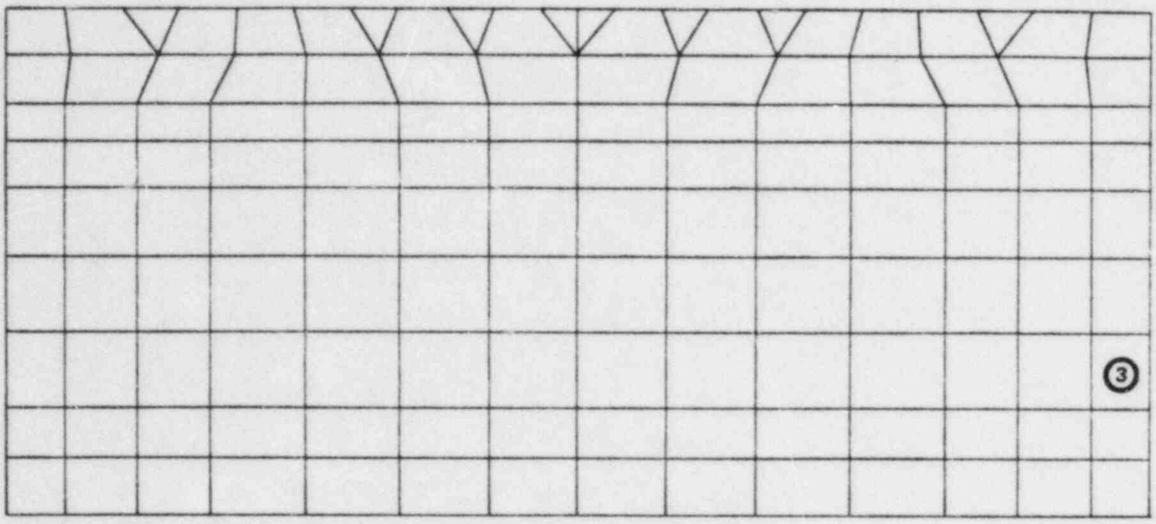
DEVELOPED ELEVATION OF SHELL WALL (LOOKING OUTWARD)

REINFORCEMENT ORIENTATION	KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (3)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
			AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	INSIDE FACE (IN^2)	OUTSIDE FACE (IN^2)	INSIDE FACE (IN^2)	OUTSIDE FACE (IN^2)
HORIZONTAL	1	3	+102	+12	1.14	1.17	3.12	4.68
HORIZONTAL	2	3	+111	−12	1.14	1.17	3.12	4.68
VERTICAL	3	3	+92	−3	1.14	0.92	4.36	4.36
VERTICAL	4	3	+81	−8	0.89	0.92	4.36	4.36
HORIZONTAL	5	3	+172	+145	1.42	2.89	4.94	7.49
HORIZONTAL	6	3	+144	−164	2.67	0.64	4.94	7.49
VERTICAL	7	3	+193	+41	1.92	2.14	4.57	4.36
VERTICAL	8	3	+181	−45	2.17	1.89	4.57	4.36
HORIZONTAL	9	3	+360	+34	3.11	4.06	4.80	4.80
HORIZONTAL	10	3	+345	−27	3.11	3.81	4.80	4.80
VERTICAL	11	3	+312	+43	2.36	4.81	2.45	6.75
VERTICAL	12	3	+193	+623	2.36	5.06	2.45	6.75

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE INSIDE FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 12", h = 36" FOR KEY ELEMENTS 1 THRU 8; AND b = 12", h = 60" FOR KEY ELEMENTS 9 THRU 12.
3. SEE TABLE 4.

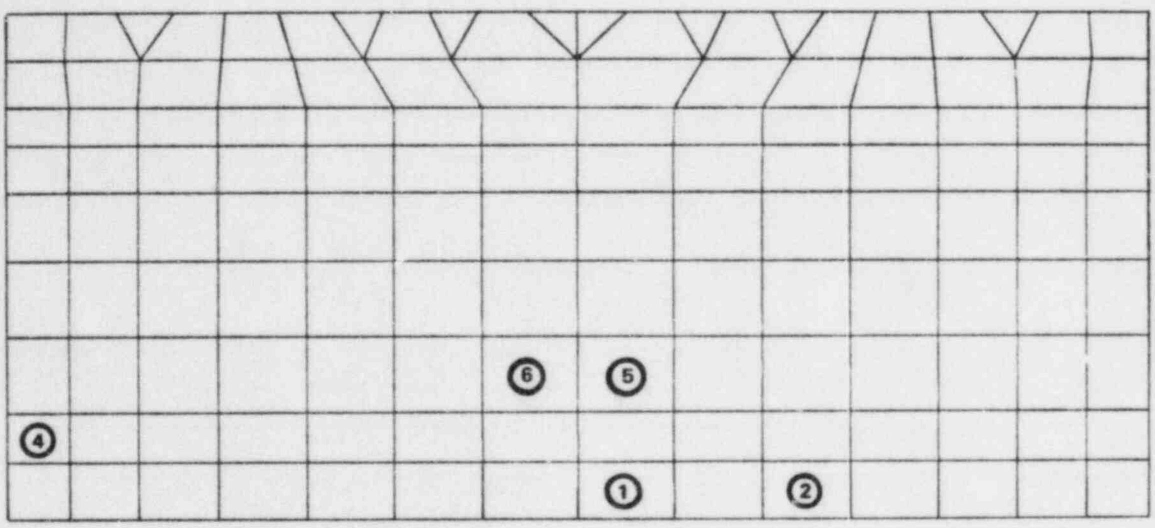
Figure 15
ANALYSIS AND DESIGN RESULTS
FOR NSCW TOWER SHELL WALL

S N



ELEVATION VIEW OF NORTH-SOUTH CROSSWALL (LOOKING WEST)

W E

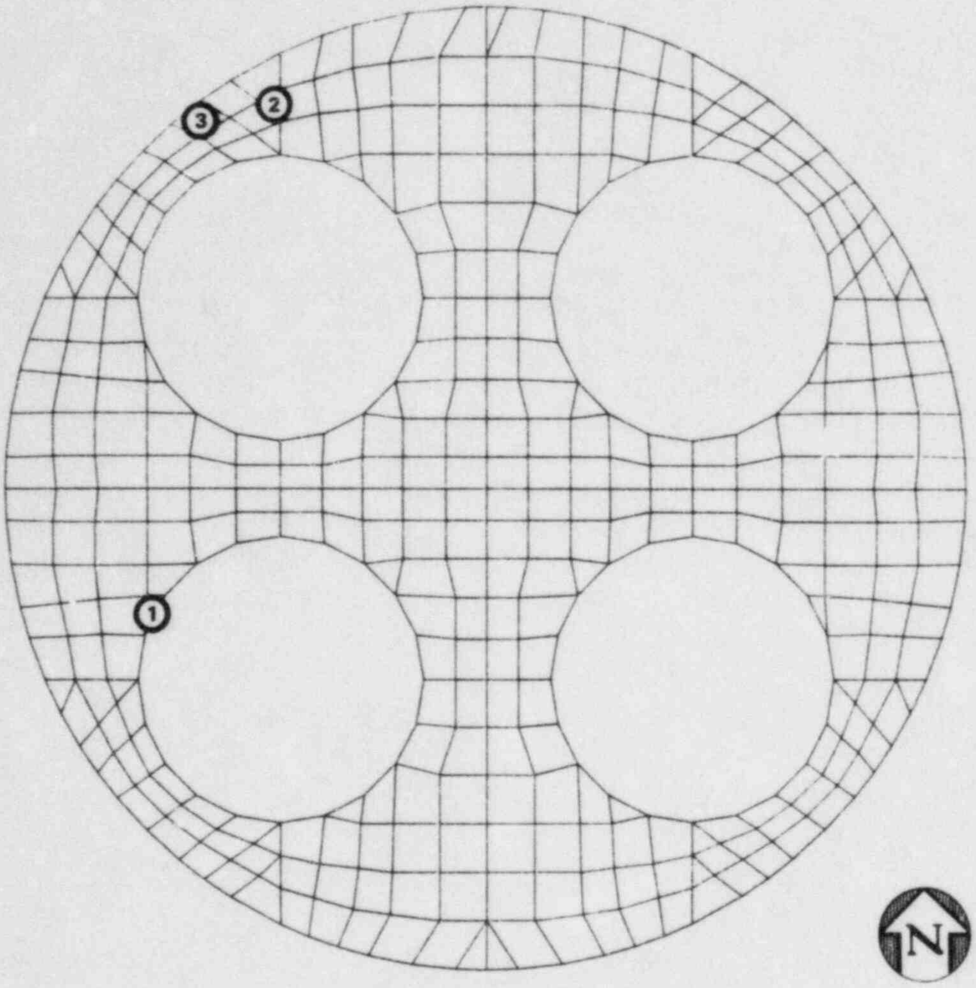


ELEVATION VIEW OF EAST-WEST CROSSWALL (LOOKING NORTH)

REINFORCEMENT ORIENTATION	KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (4)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
			AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	SOUTH OR EAST FACE (IN^2)	NORTH OR WEST FACE (IN^2)	SOUTH OR EAST FACE (IN^2)	NORTH OR WEST FACE (IN^2)
HORIZONTAL	1	3	+133	+10	1.81	1.43	3.05	3.05
HORIZONTAL	2	3	+130	+23	1.62	1.62	3.05	3.05
VERTICAL	3	3	−35	−64	0.50 (3)	0.50 (3)	1.00	1.00
VERTICAL	4	3	+15	−31	0.50 (3)	0.50 (3)	1.00	1.00
HORIZONTAL	5	3	+74	−23	1.06	0.87	1.33	1.33
HORIZONTAL	6	3	+74	−4	0.87	1.06	1.33	1.33

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE SOUTH OR EAST FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 12", h = 27".
3. DESIGN IS GOVERNED BY MINIMUM REINFORCEMENT REQUIREMENTS.
4. SEE TABLE 4.

Figure 16
ANALYSIS AND DESIGN RESULTS
FOR NSCW TOWER CROSSWALLS

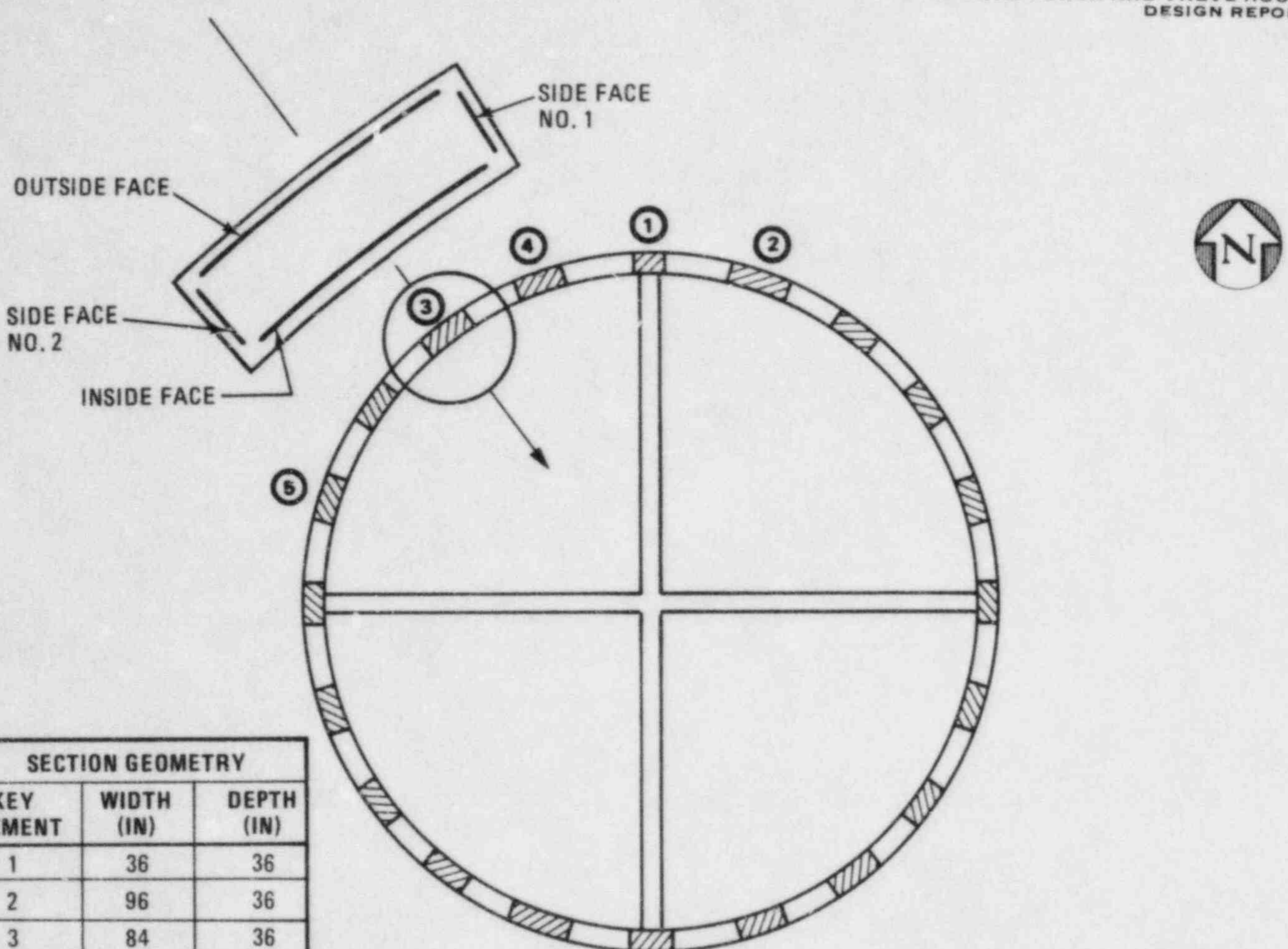


PLAN VIEW OF FAN DECK

REINFORCEMENT ORIENTATION	KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (4)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
			AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)
E-W	1	3	+34	+24	0.70	0.53 (3)	0.79	0.79
E-W	2	3	+29	+33	0.53 (3)	0.70	0.79	0.79
N-S	3	3	+30	−30	0.70	0.53 (3)	0.79	0.79

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE TOP FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 12", h = 24".
3. DESIGN IS GOVERNED BY MINIMUM REINFORCEMENT REQUIREMENTS.
4. SEE TABLE 4.

Figure 17
ANALYSIS AND DESIGN RESULTS
FOR NSCW TOWER FAN DECK



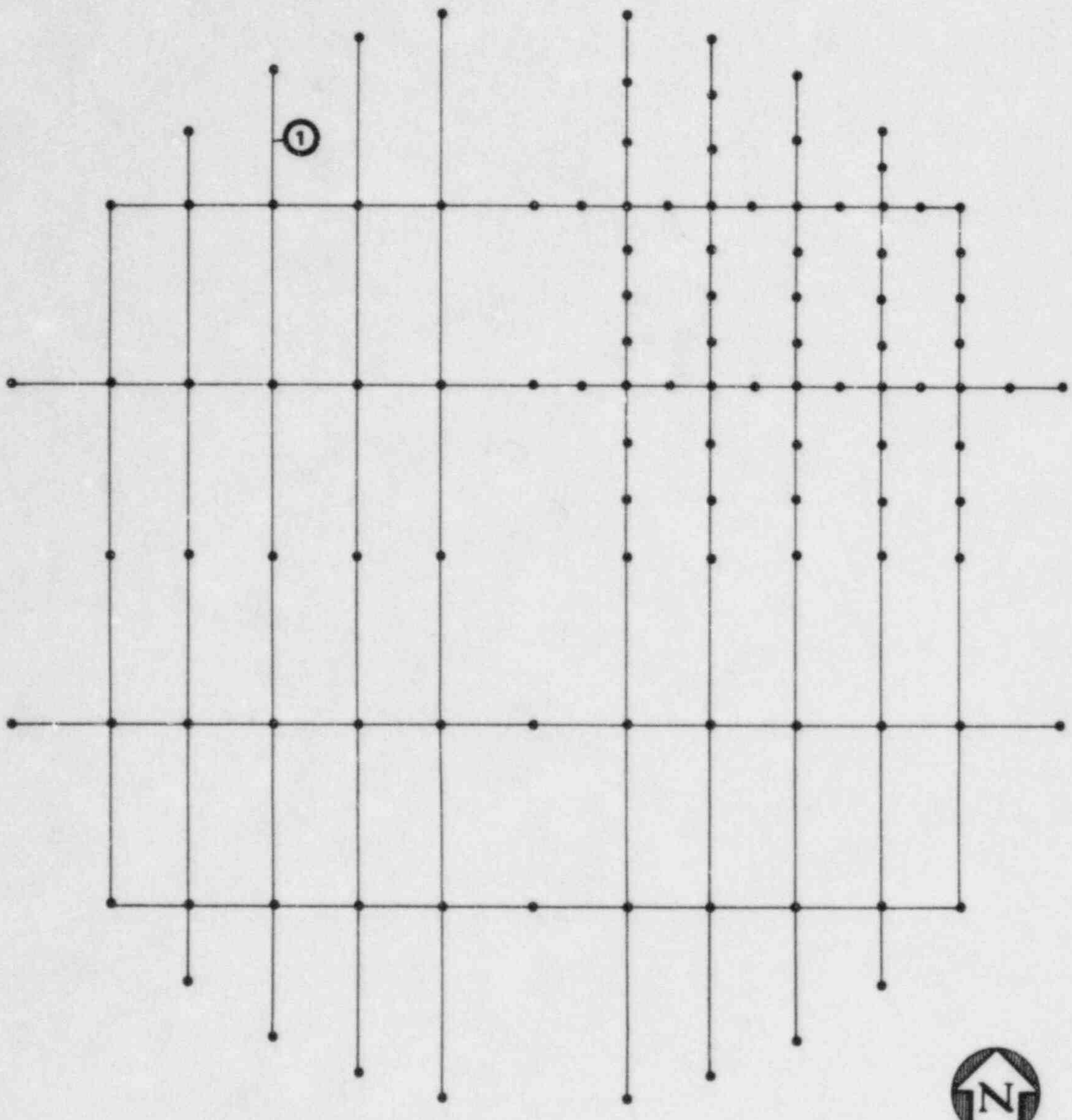
SECTION GEOMETRY		
KEY ELEMENT	WIDTH (IN)	DEPTH (IN)
1	36	36
2	96	36
3	84	36
4, 5	72	36

PLAN VIEW OF COLUMNS

KEY ELEMENT	GOVERNING LOAD COMBINATION EQUATION (3)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
		AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	INSIDE FACE (IN^2)	OUTSIDE FACE (IN^2)	INSIDE FACE (IN^2)	OUTSIDE FACE (IN^2)
1	3	-154	+166	3.71 (2)	3.63 (2)	12.00	15.00
2	3	+19	+466	9.90 (2)	9.68 (2)	36.00	45.00
3	3	-557	-1421	8.66 (2)	8.47 (2)	32.00	40.00
4	3	-519	-1859	10.43	7.26 (2)	28.00	35.00
				SIDE FACE NO. 1 (IN^2)	SIDE FACE NO. 2 (IN^2)	SIDE FACE NO. 1 (IN^2)	SIDE FACE NO. 2 (IN^2)
1	3	-44	+906	3.83 (2)	6.33 (2)	17.00	17.00
2	3	+49	+5277	10.93 (2)	13.68	17.00	17.00
3	3	-545	+3539	9.50 (2)	9.50 (2)	17.00	17.00
5	3	-245	+4614	8.11 (2)	14.36	17.00	[illegible]

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION ON EITHER THE INSIDE FACE OR SIDE FACE NO. 1.
2. DESIGN IS GOVERNED BY MINIMUM REINFORCEMENT REQUIREMENTS.
3. SEE TABLE 4.

Figure 18
ANALYSIS AND DESIGN RESULTS
FOR NSCW TOWER COLUMNS

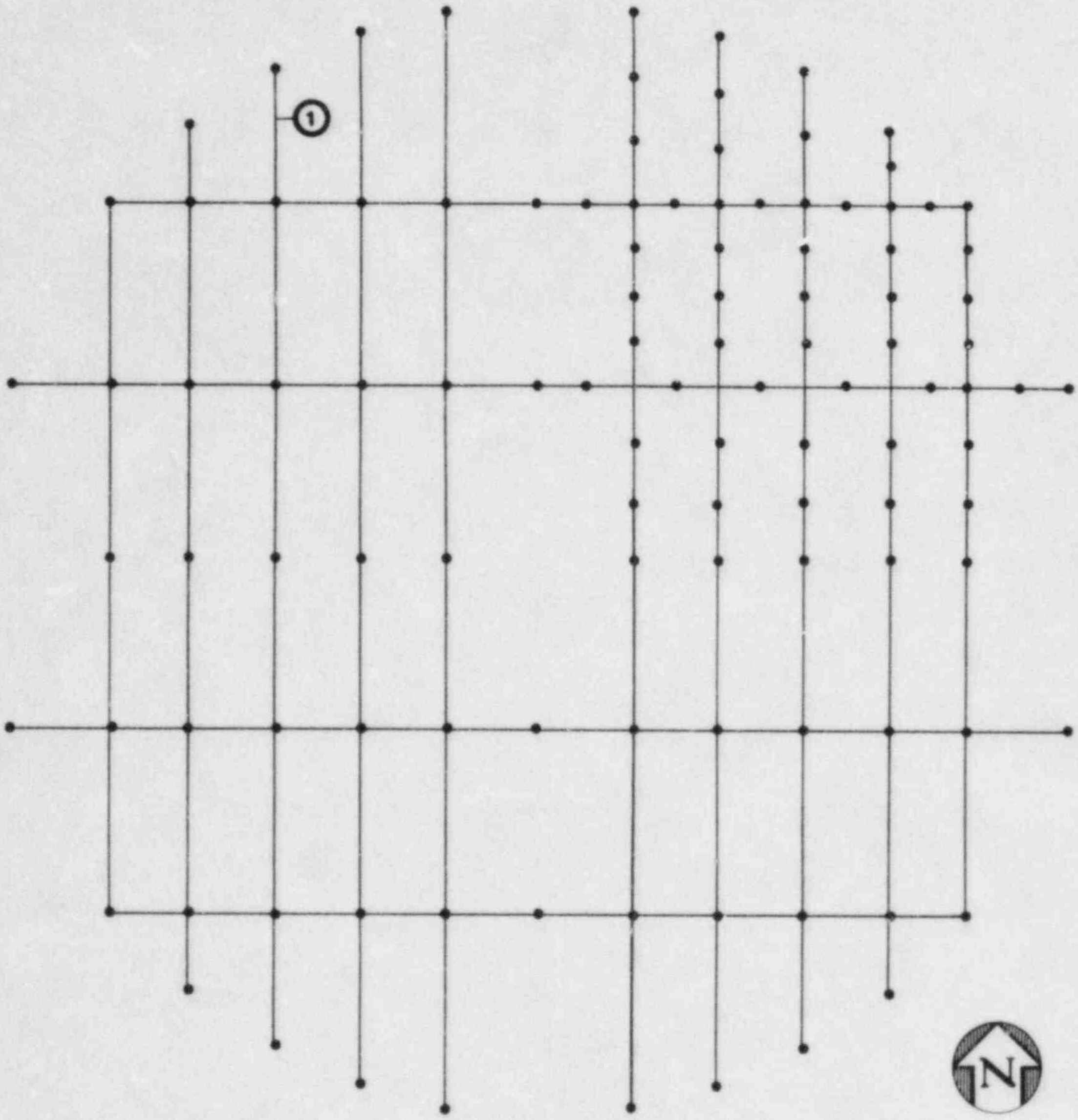


PLAN VIEW OF FILL LEVEL BEAMS

KEY ELEMENT [2]	GOVERNING LOAD COMBINATION EQUATION [4]	DESIGN FORCES [1]		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
		AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)
1	3	+49	-668	3.17	2.79 [3]	5.20	5.00

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE TOP FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 12", h = 65".
3. DESIGN IS GOVERNED BY MINIMUM REINFORCEMENT REQUIREMENTS.
4. SEE TABLE 4.

Figure 19
ANALYSIS AND DESIGN RESULTS
FOR NSCW TOWER BEAMS
(Sheet 1 of 3)

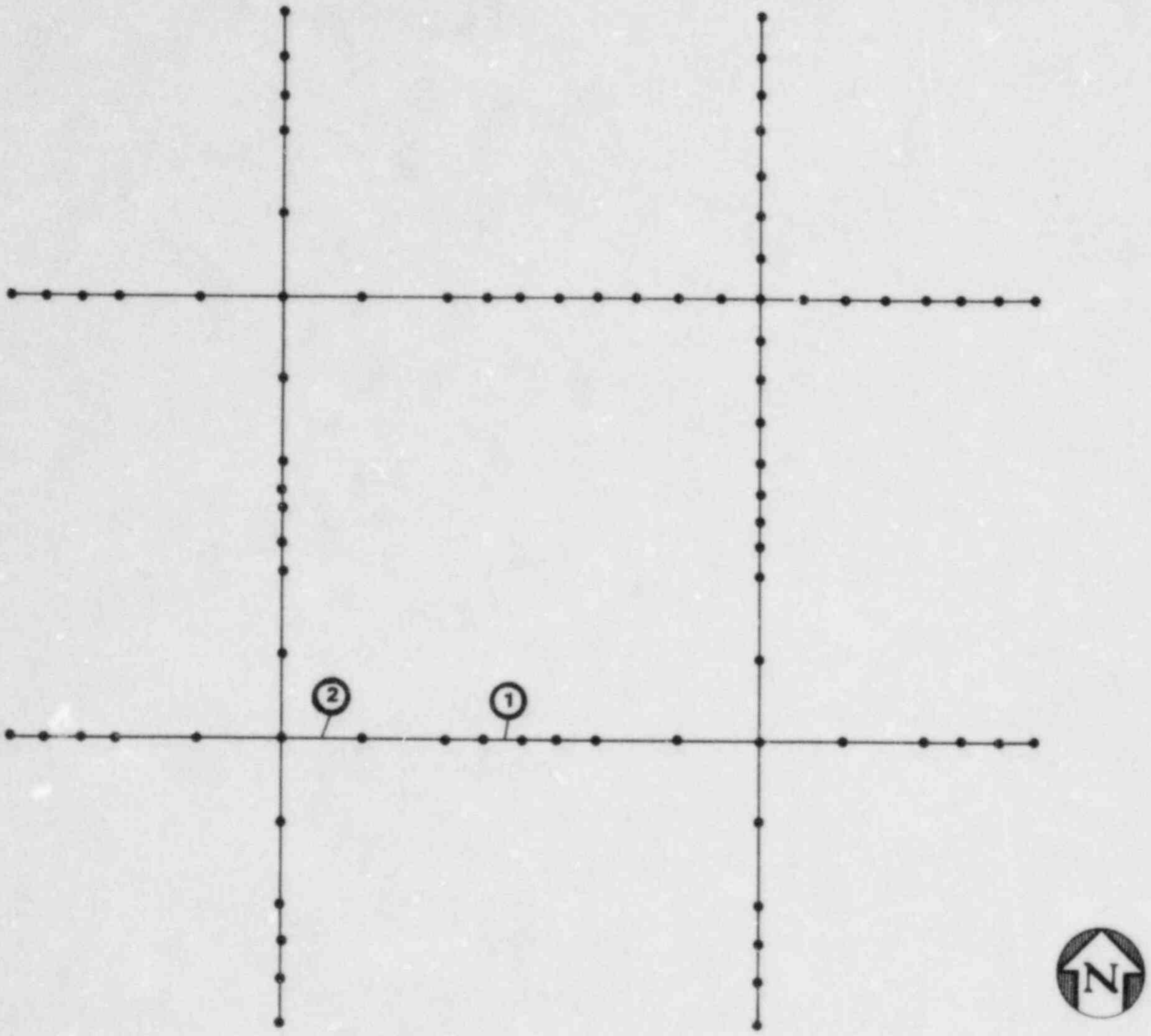


PLAN VIEW OF ELIMINATOR LEVEL BEAMS

KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (4)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
		AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)
1	4	-2	-380	3.15	1.52 (3)	4.00	3.00

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE TOP FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 14", h = 36-1/2".
3. DESIGN IS GOVERNED BY MINIMUM REINFORCEMENT REQUIREMENTS.
4. SEE TABLE 4.

Figure 19
ANALYSIS AND DESIGN RESULTS
FOR NSCW TOWER BEAMS
(Sheet 2 of 3)



PLAN VIEW OF FAN DECK LEVEL BEAMS

KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (4)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
		AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)
1	3	-99	-491	3.80 (3)	3.59 (3)	7.62	10.16
2	3	+96	+267	3.80 (3)	3.59 (3)	7.62	10.16

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE TOP FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 24", h = 54.7".
3. DESIGN IS GOVERNED BY MINIMUM REINFORCEMENT REQUIREMENTS.
4. SEE TABLE 4.

Figure 19
ANALYSIS AND DESIGN RESULTS
FOR NSCW TOWER BEAMS
(Sheet 3 of 3)

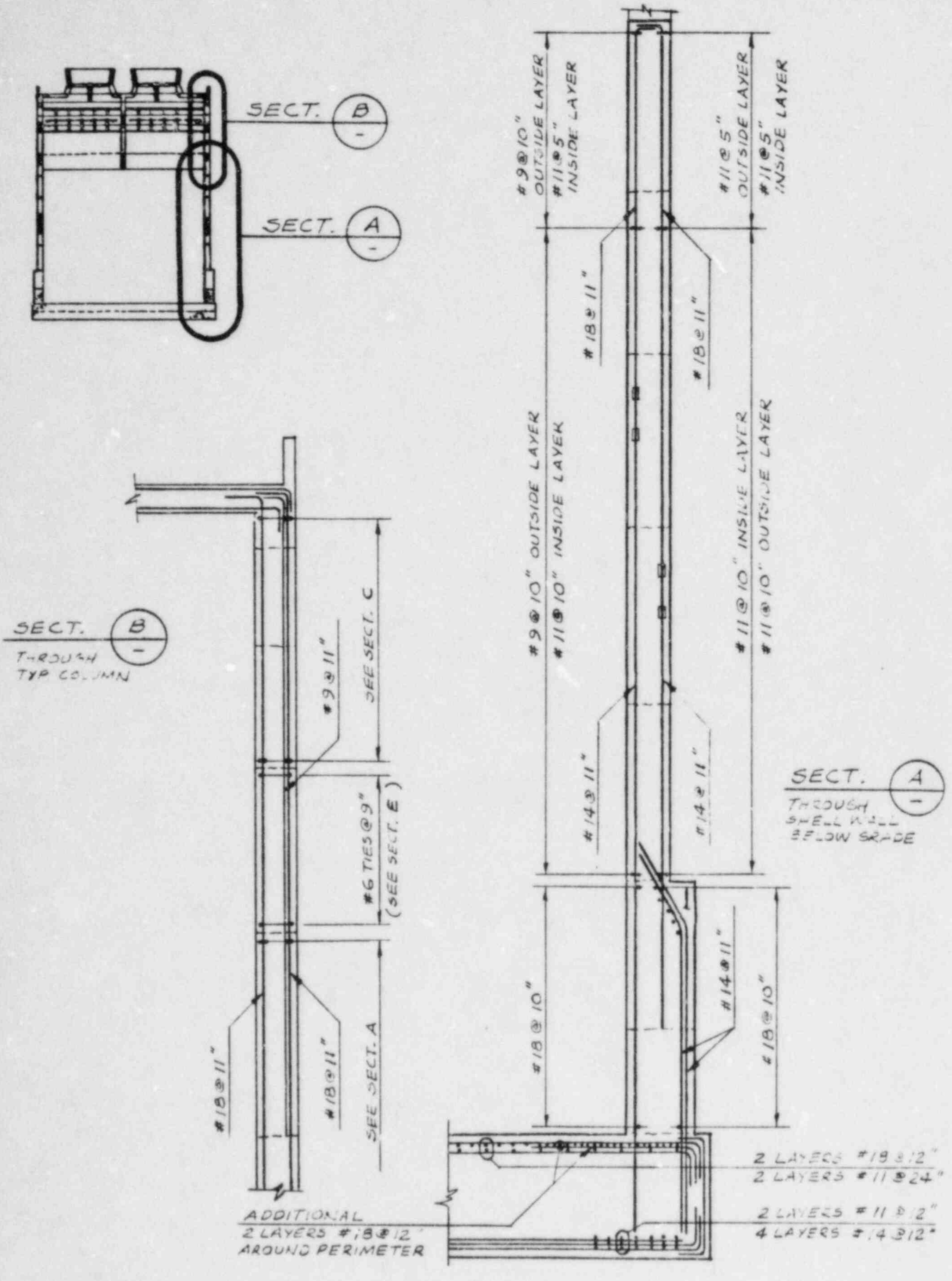


Figure 20
CONCRETE REINFORCING DETAILS
FOR NSCW TOWER
(Sheet 1 of 3)

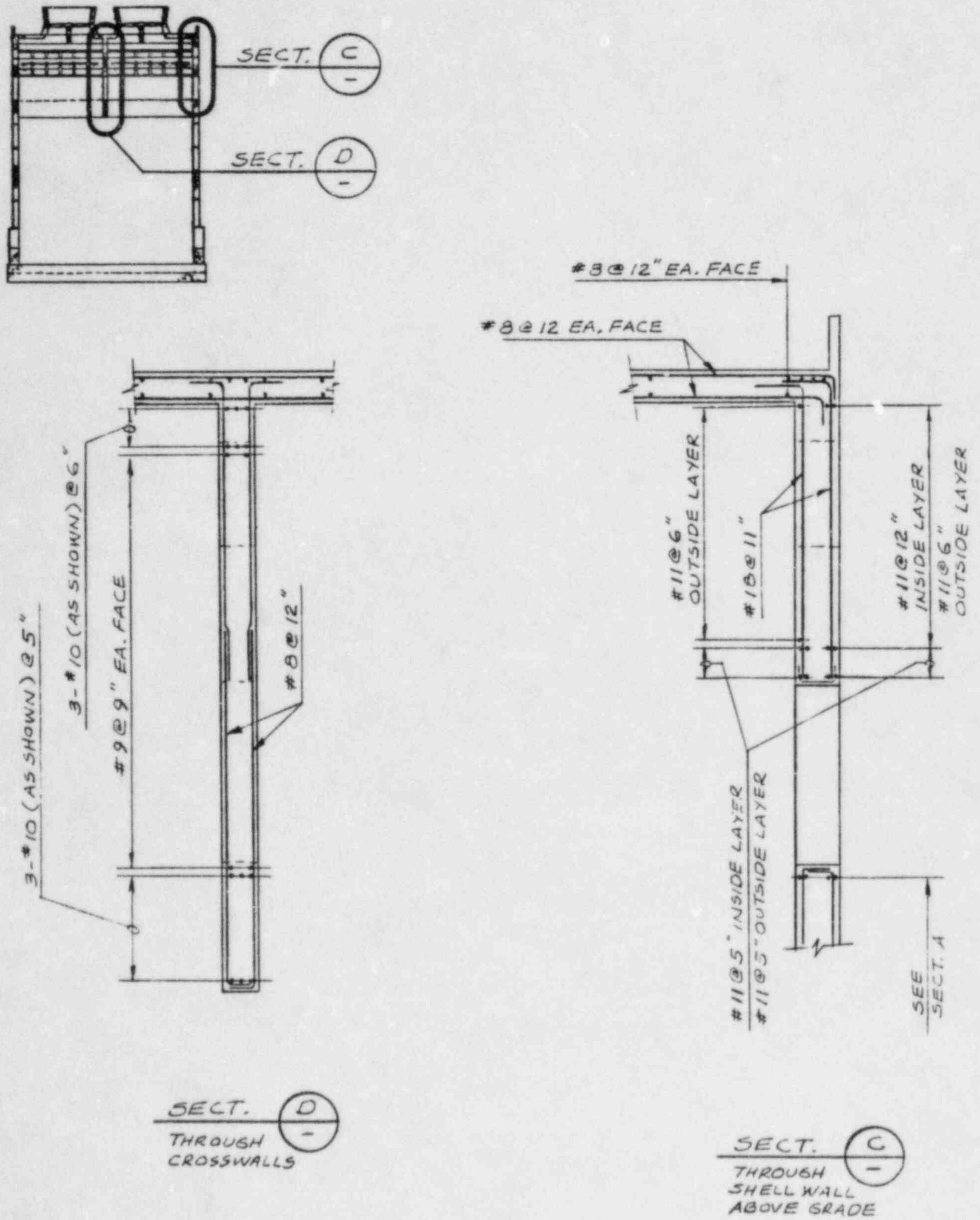


Figure 20
CONCRETE REINFORCING DETAILS
FOR NSCW TOWER
(Sheet 2 of 3)

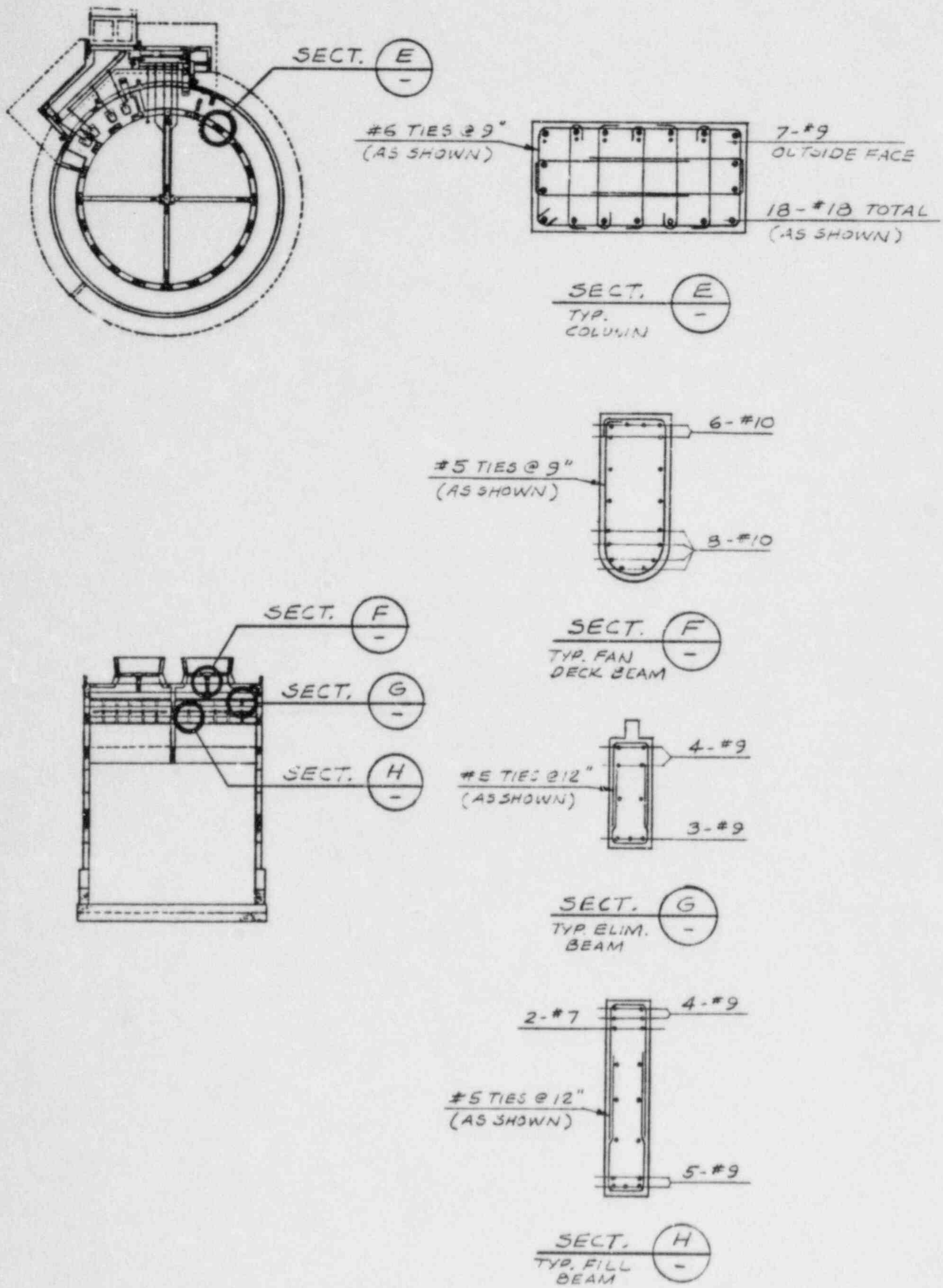
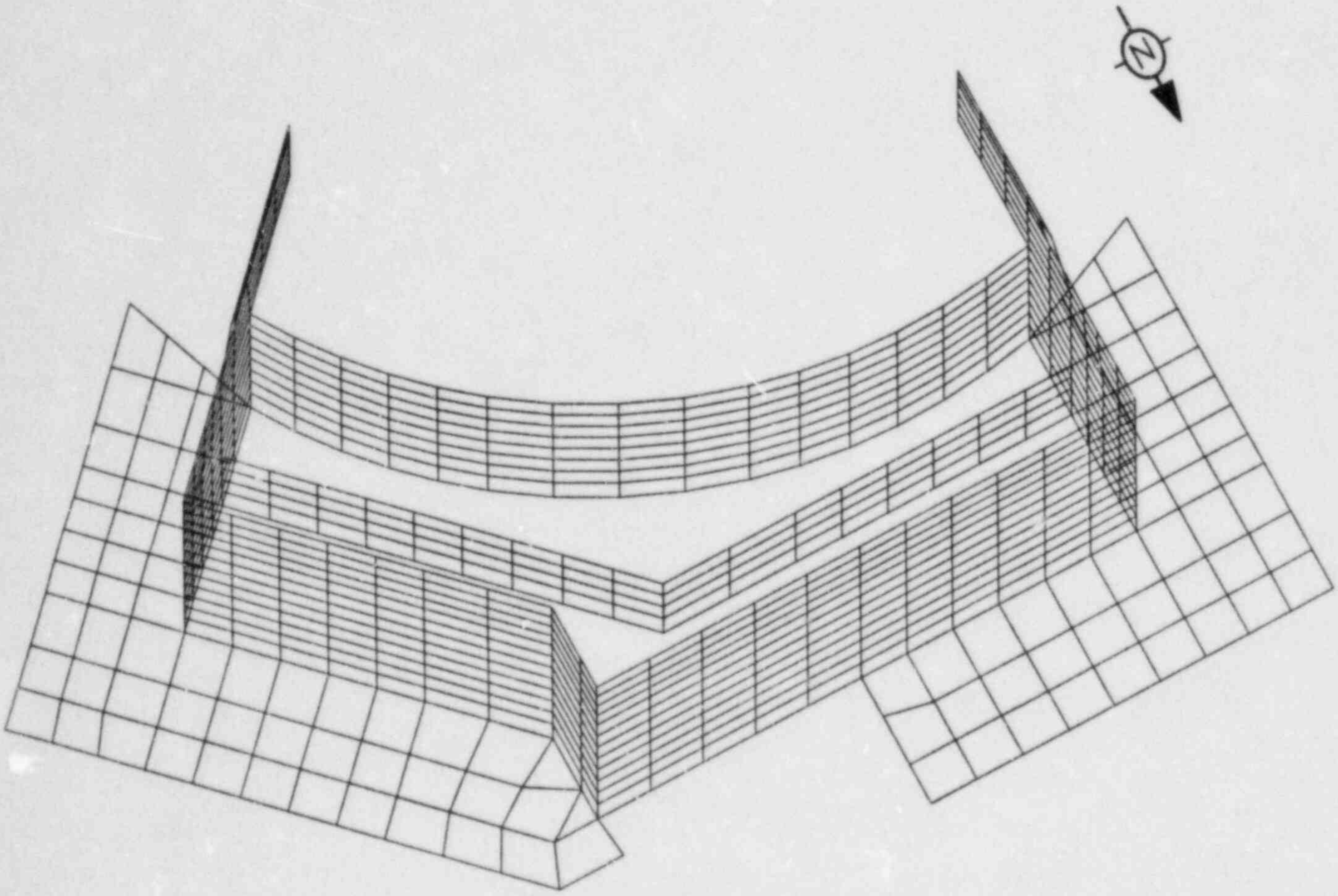
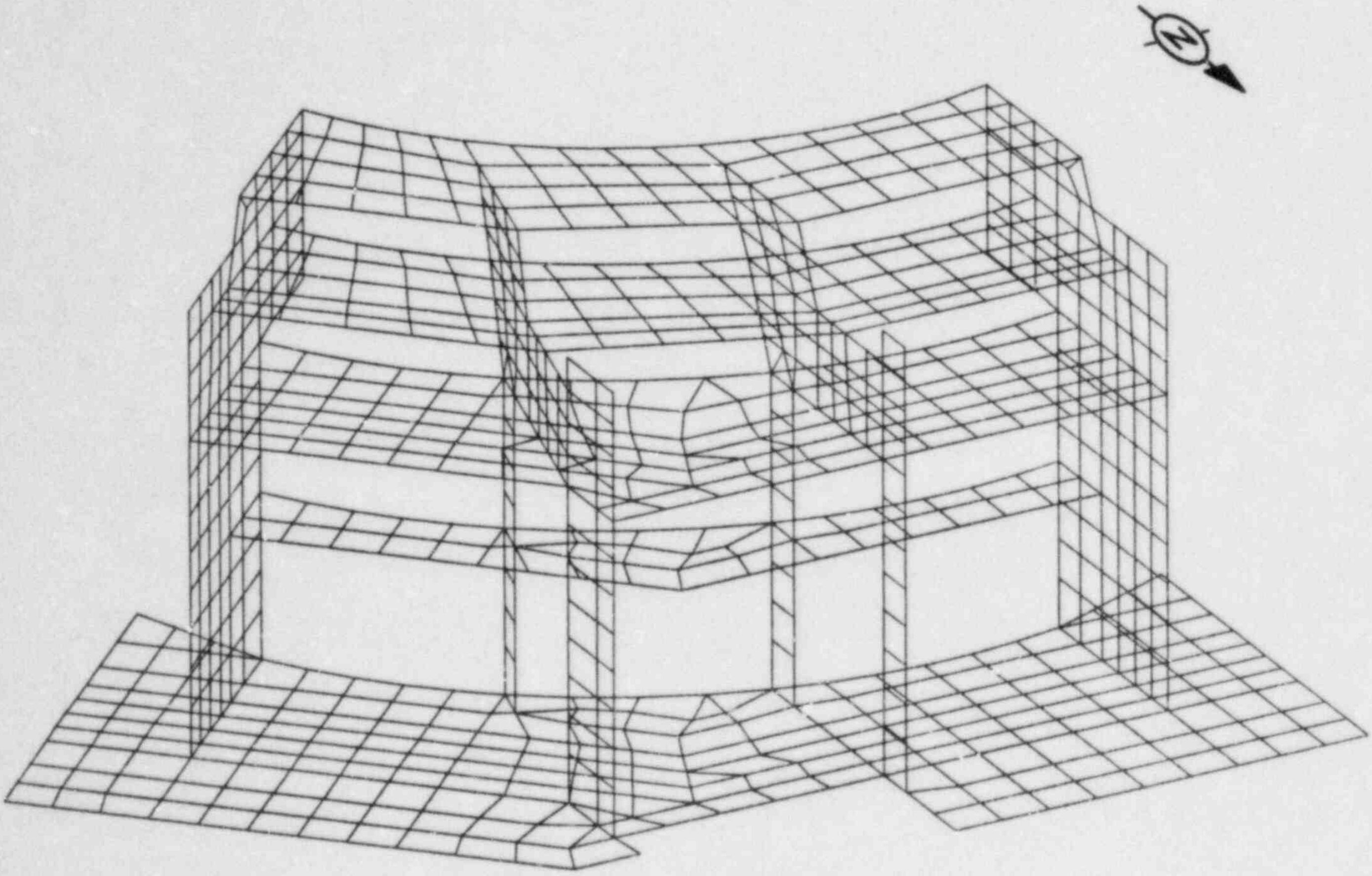


Figure 20
CONCRETE REINFORCING DETAILS
FOR NSCW TOWER
(Sheet 3 of 3)



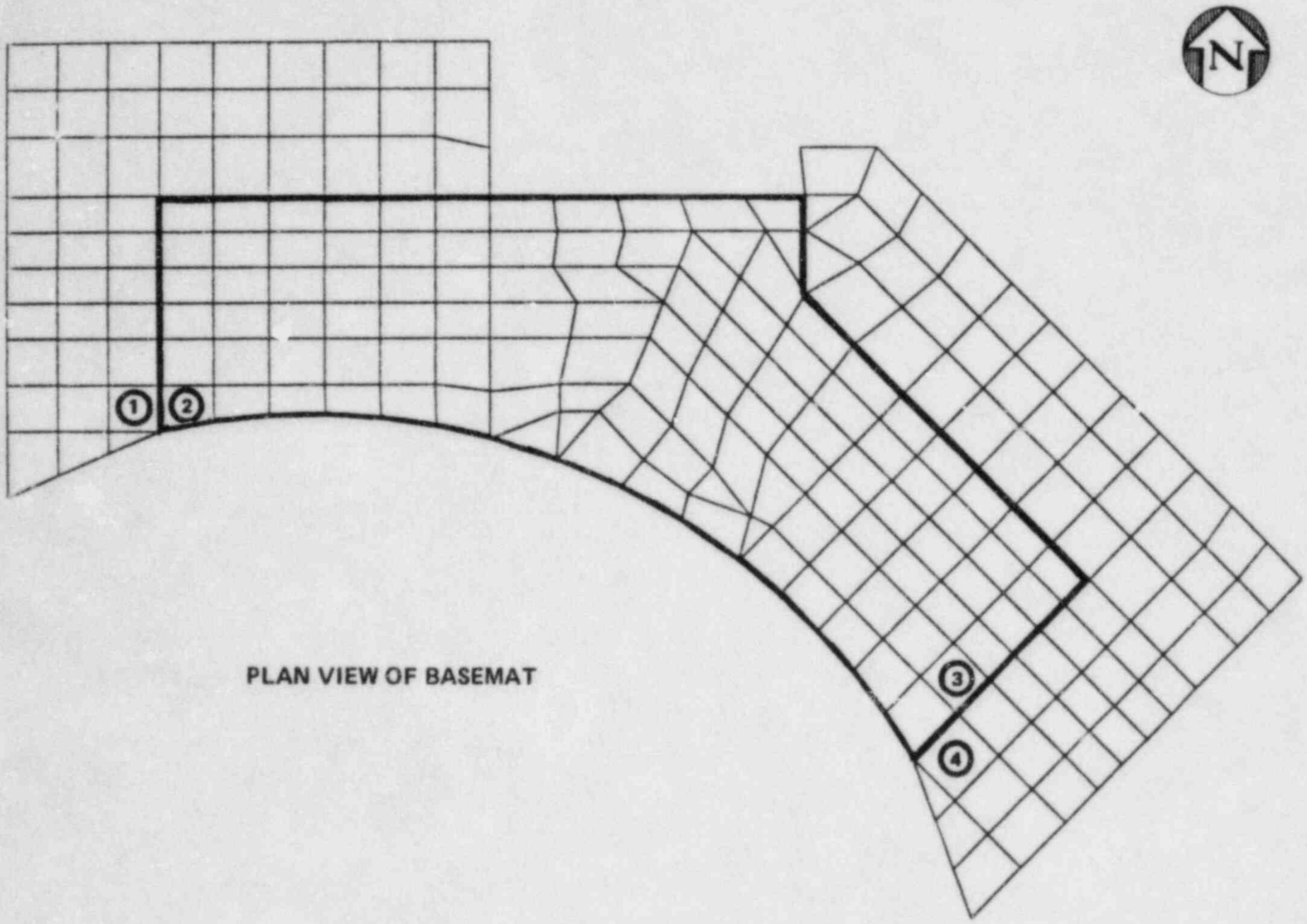
ISOMETRIC VIEW OF WALLS AND BASEMAT EXTENSIONS

Figure 21
FINITE ELEMENT MODEL
OF NSCW VALVE HOUSE
(Sheet 1 of 2)



ISOMETRIC VIEW OF SIDE WALLS AND ALL SLABS

Figure 21
FINITE ELEMENT MODEL
OF NSCW VALVE HOUSE
(Sheet 2 of 2)

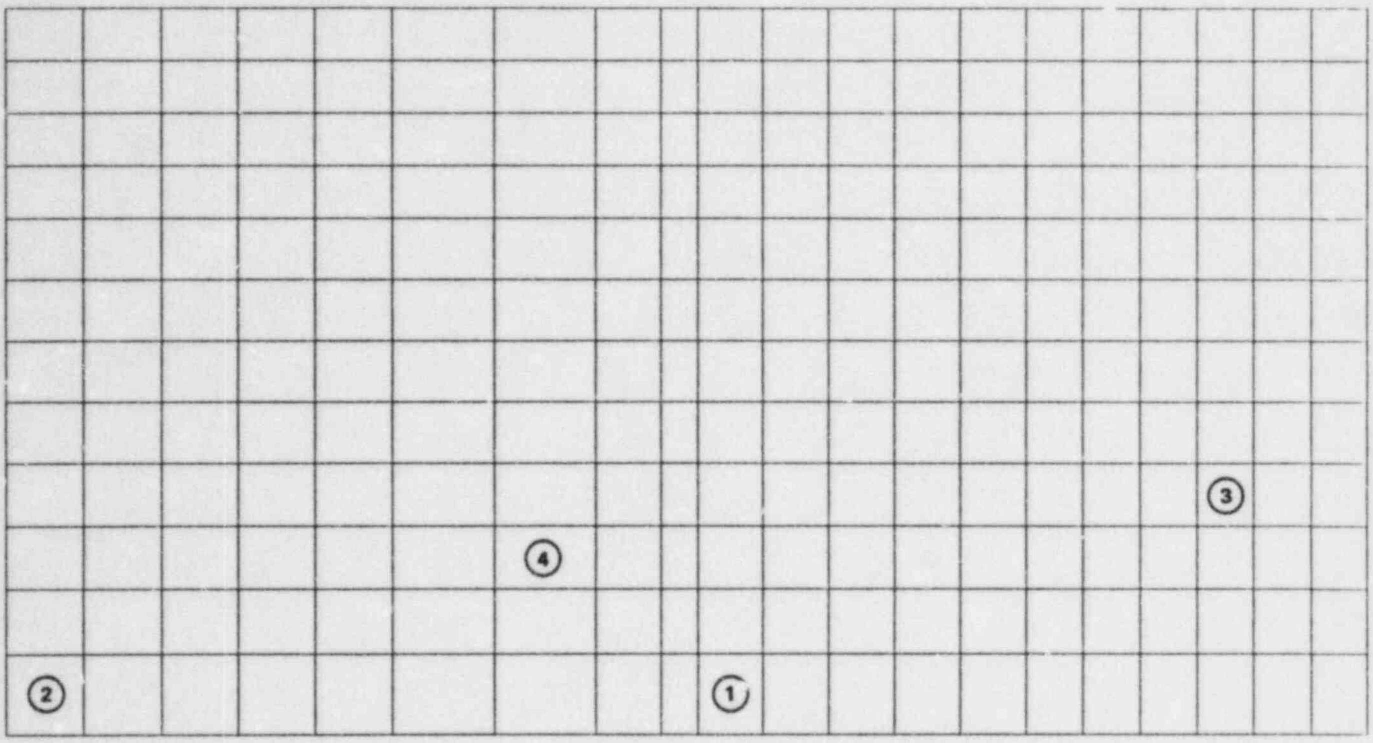


PLAN VIEW OF BASEMAT

REINFORCEMENT ORIENTATION	KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (4)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
			AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)
LONGITUDINAL	1	3	–	+ 444	1.57 (3)	1.54 (3)	2.25	2.25
LONGITUDINAL	2	3	–	+ 508	1.57 (3)	1.79	2.25	2.25
LONGITUDINAL	3	3	–	+ 584	1.57 (3)	2.04	2.25	2.25
LONGITUDINAL	4	3	–	+ 532	1.57 (3)	2.04	2.25	2.25

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE TOP FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 12", h = 72".
3. DESIGN IS GOVERNED BY MINIMUM REINFORCEMENT REQUIREMENTS.
4. SEE TABLE 5.

Figure 22
ANALYSIS AND DESIGN RESULTS
FOR NSCW VALVE HOUSE BASEMAT

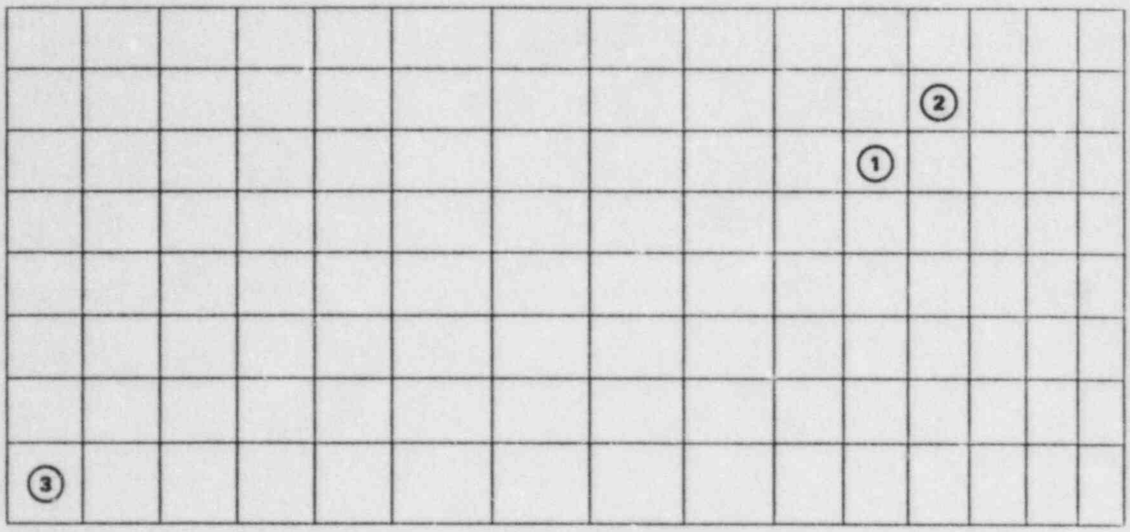


ELEVATION VIEW OF WALL C (LOOKING SOUTHWEST)

REINFORCEMENT ORIENTATION	KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (4)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
			AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	INSIDE FACE (IN^2)	OUTSIDE FACE (IN^2)	INSIDE FACE (IN^2)	OUTSIDE FACE (IN^2)
VERTICAL	1	3	+ 96	+17	1.14	1.14	2.34	2.34
VERTICAL	2	3	+ 138	- 38	1.76	2.26	2.34	2.34
HORIZONTAL	3	3	+ 46	+ 2	0.81	0.47 (3)	1.27	1.27
HORIZONTAL	4	3	+ 59	+ 19	0.76	0.76	1.27	1.27

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE INSIDE FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 12", h = 24" FOR KEY ELEMENTS 1 AND 3; AND b = 12", h = 36" FOR KEY ELEMENTS 2 AND 4.
3. DESIGN IS GOVERNED BY MINIMUM REINFORCEMENT REQUIREMENTS.
4. SEE TABLE 5.

Figure 23
ANALYSIS AND DESIGN RESULTS
FOR NSCW VALVE HOUSE WALLS
(Sheet 1 of 3)

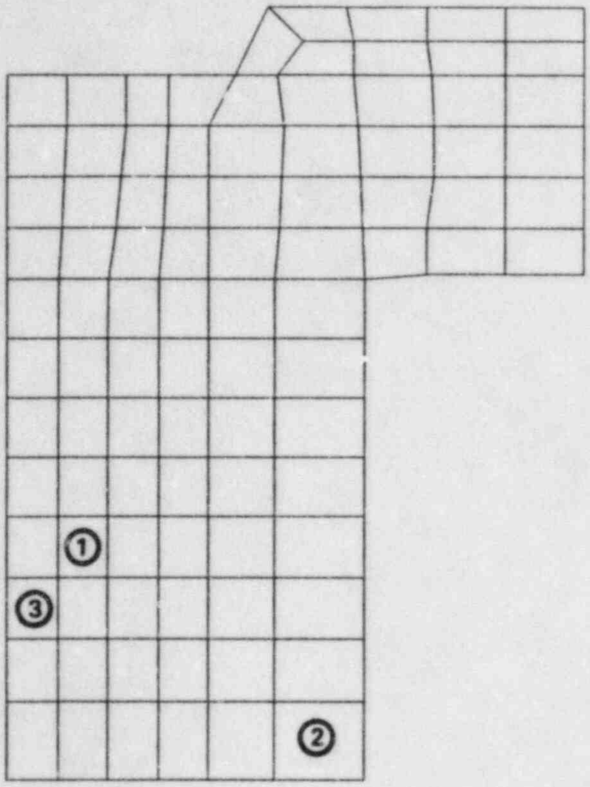


ELEVATION VIEW OF WALL A (LOOKING SOUTHWEST)

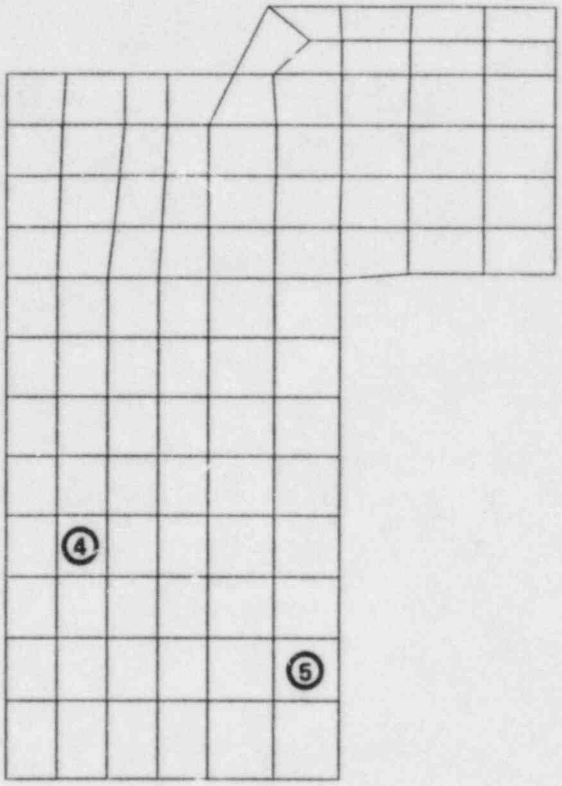
REINFORCEMENT ORIENTATION	KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (3)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
			AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	INSIDE FACE (IN^2)	OUTSIDE FACE (IN^2)	INSIDE FACE (IN^2)	OUTSIDE FACE (IN^2)
HORIZONTAL	1	3	+ 82	-2	0.81	0.81	1.27	1.27
HORIZONTAL	2	3	+ 91	+4	0.97	0.97	1.27	1.27
VERTICAL	3	3	+ 100	-21	1.31	1.31	1.56	1.56

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE INSIDE FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 12", h = 24".
3. SEE TABLE 5.

Figure 23
ANALYSIS AND DESIGN RESULTS
FOR NSCW VALVE HOUSE WALLS
(Sheet 2 of 3)



ELEVATION VIEW OF WALL D
(LOOKING SOUTHEAST)

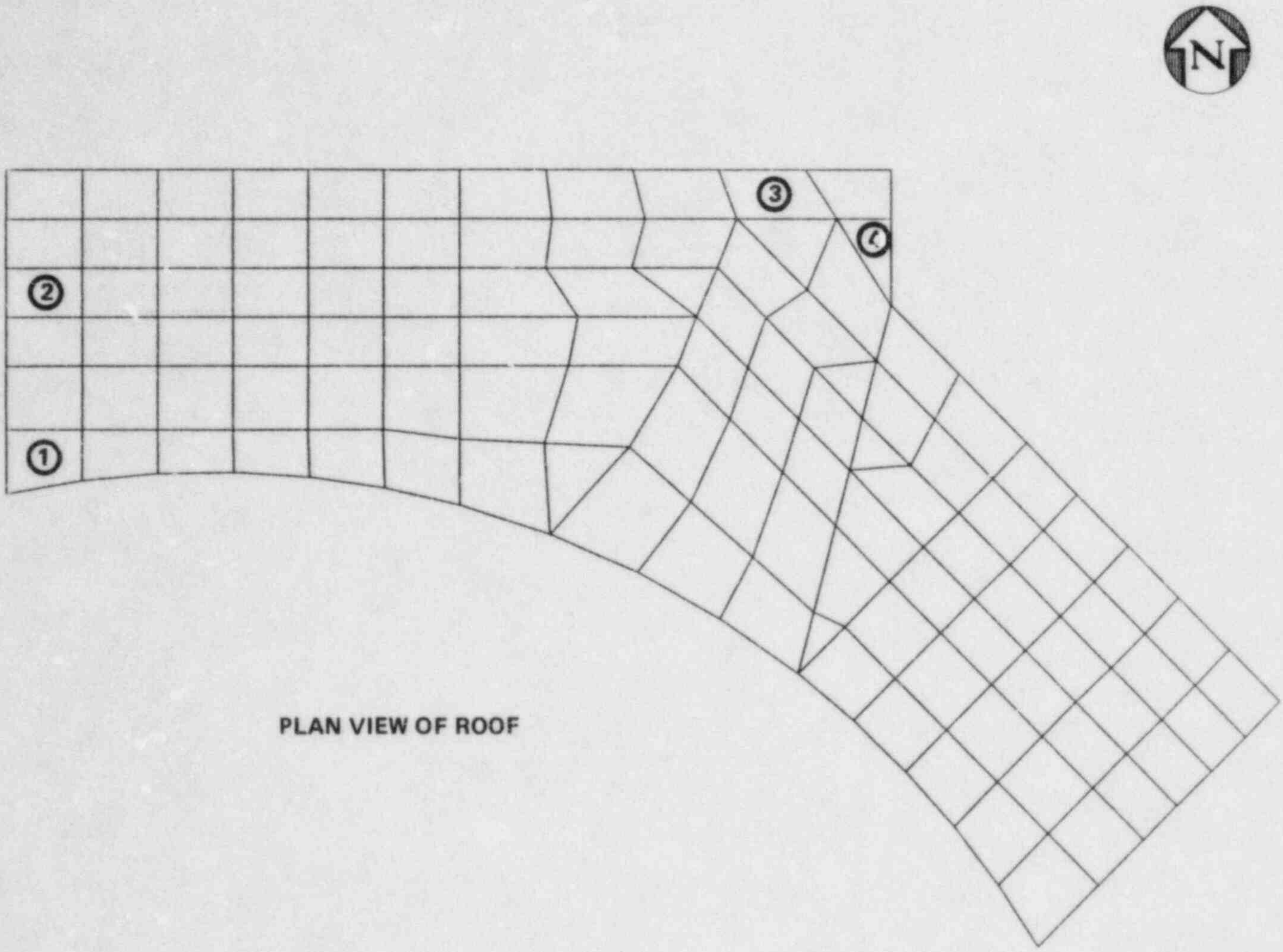


ELEVATION VIEW OF WALL E
(LOOKING EAST)

REINFORCEMENT ORIENTATION	KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (3)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
			AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	INSIDE FACE (IN^2)	OUTSIDE FACE (IN^2)	INSIDE FACE (IN^2)	OUTSIDE FACE (IN^2)
HORIZONTAL	1	3	+ 57	+ 10	0.81	0.81	1.27	1.27
VERTICAL	2	3	+ 107	+ 10	1.14	1.14	1.56	1.56
VERTICAL	3	3	+ 104	+ 1	0.97	1.14	1.56	1.56
HORIZONTAL	4	3	+ 95	+ 13	1.14	1.14	1.27	1.27
VERTICAL	5	3	+125	- 2	1.31	1.31	1.56	1.56

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE INSIDE FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 12", h = 24".
3. SEE TABLE 5.

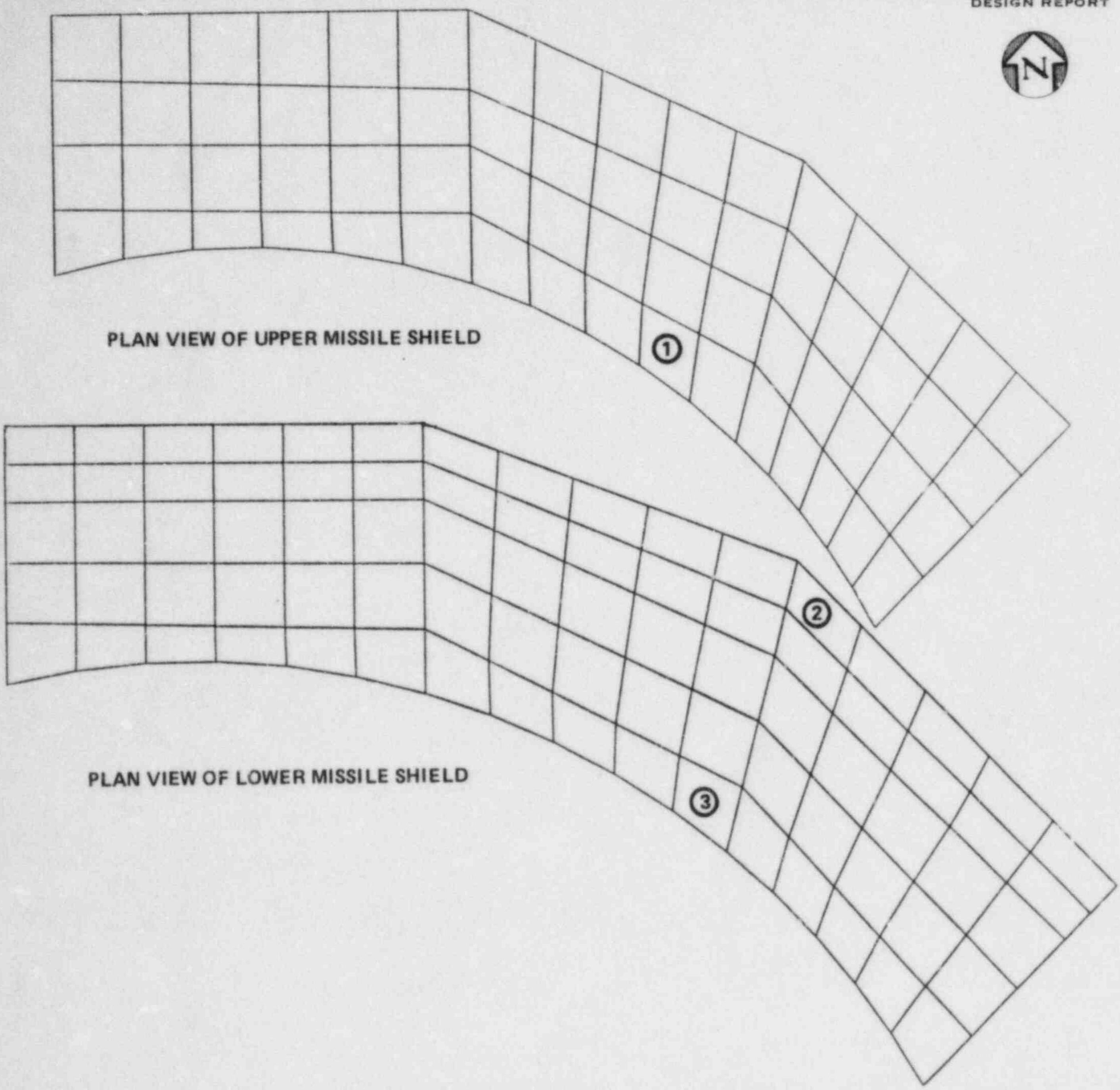
Figure 23
ANALYSIS AND DESIGN RESULTS
FOR NSCW VALVE HOUSE WALLS
(Sheet 3 of 3)



REINFORCEMENT ORIENTATION	KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (3)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
			AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)
LONGITUDINAL	1	3	+ 65	- 20	0.95	0.95	1.27	1.27
LONGITUDINAL	2	3	+ 52	- 19	0.66	0.95	1.27	1.27
TRANSVERSAL	3	3	+ 44	+ 4	0.81	0.52	1.27	1.27
TRANSVERSAL	4	3	+ 47	- 3	0.52	0.66	1.27	1.27

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE TOP FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 12", h = 21".
3. SEE TABLE 5.

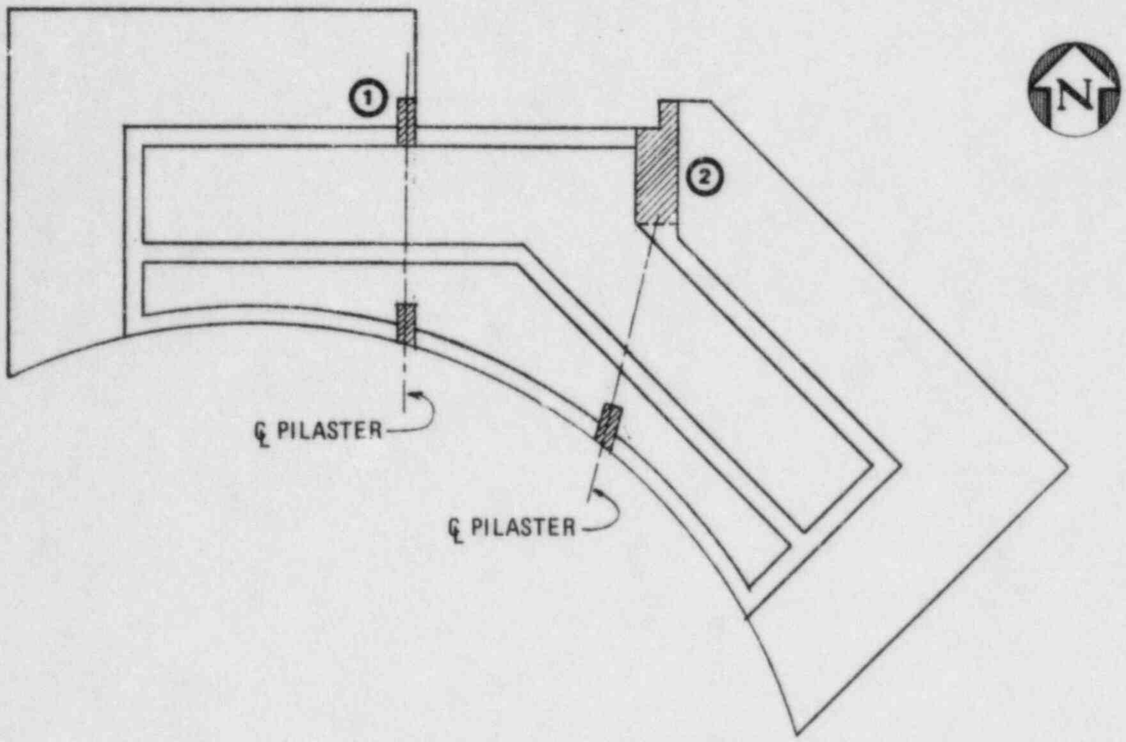
Figure 24
ANALYSIS AND DESIGN RESULTS
FOR NSCW VALVE HOUSE ROOF



REINFORCEMENT ORIENTATION	KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (3)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED		REINFORCEMENT PROVIDED	
			AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)	TOP FACE (IN^2)	BOTTOM FACE (IN^2)
LONGITUDINAL	1	3	+ 51	+15	0.79	0.79	0.95	0.95
LONGITUDINAL	2	3	+ 43	- 28	0.79	0.63	0.95	0.95
LONGITUDINAL	3	3	+ 27	- 47	0.79	0.63	0.95	0.95

NOTES:
1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE TOP FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 12", h = 24".
3. SEE TABLE 5.

Figure 25
ANALYSIS AND DESIGN RESULTS
FOR NSCW VALVE HOUSE MISSILE SHIELDS



PLAN VIEW OF PILASTERS

REINFORCEMENT ORIENTATION	KEY ELEMENT (2)	GOVERNING LOAD COMBINATION EQUATION (3)	DESIGN FORCES (1)		REINFORCEMENT REQUIRED (IN^2)	REINFORCEMENT PROVIDED (IN^2)
			AXIAL FORCE (KIP)	BENDING MOMENT (FT-KIP)		
VERTICAL	1	3	+130	+1789	9.35	12.48
VERTICAL	2	3	+135	+7293	17.40	21.84

NOTES:

1. POSITIVE AXIAL FORCE INDICATES TENSION; POSITIVE BENDING MOMENT INDICATES COMPRESSION AT THE SOUTH FACE.
2. CROSS SECTION GEOMETRY IS AS FOLLOWS: b = 24'', h = 60'' FOR KEY ELEMENT NO. 1 AND b = 48'', h = 108'' FOR KEY ELEMENT NO. 2.
3. SEE TABLE 5.

Figure 26
ANALYSIS AND DESIGN RESULTS
FOR NSCW VALVE HOUSE PILASTERS

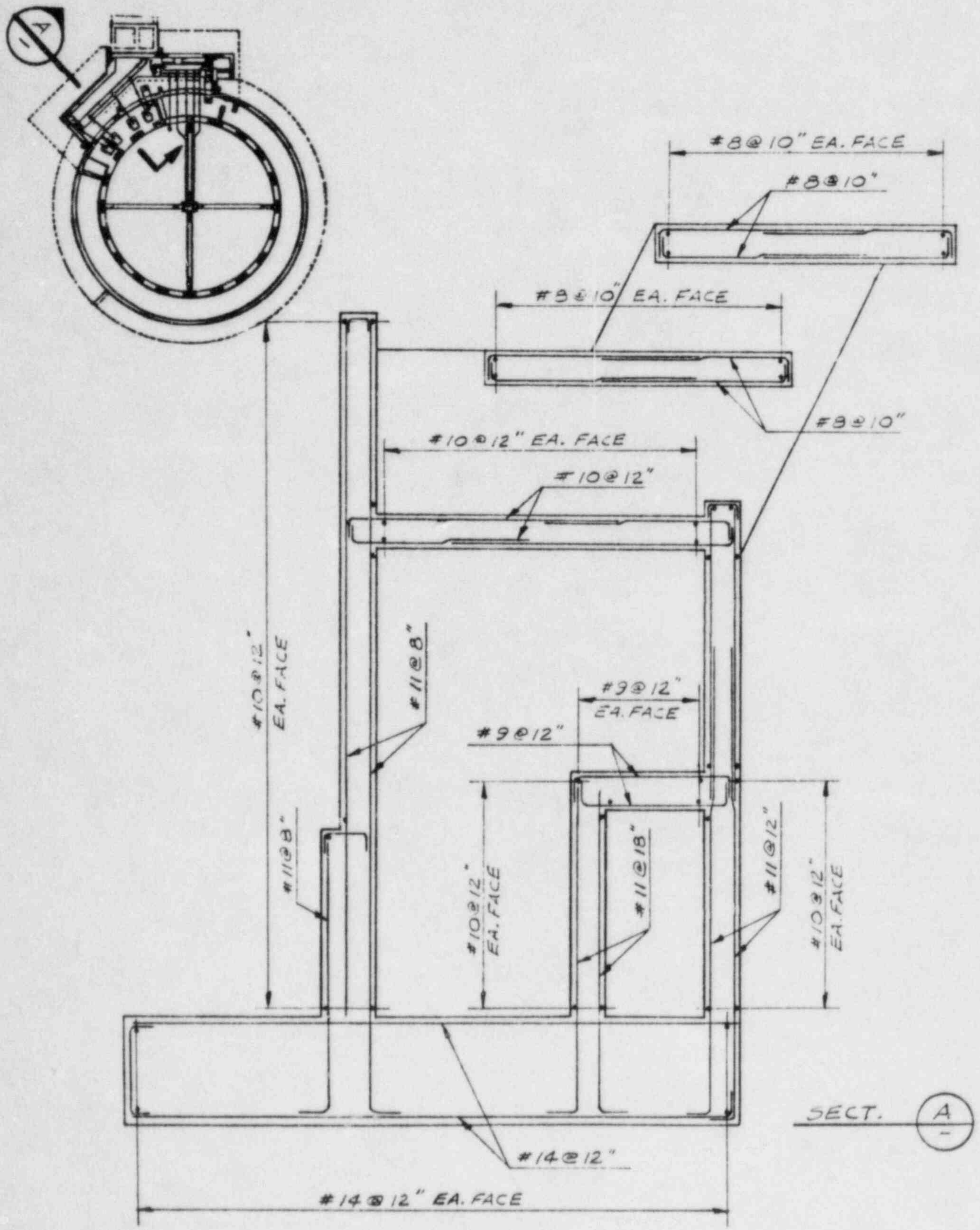


Figure 27
CONCRETE REINFORCING DETAILS
FOR NSCW VALVE HOUSE
(Sheet 1 of 2)

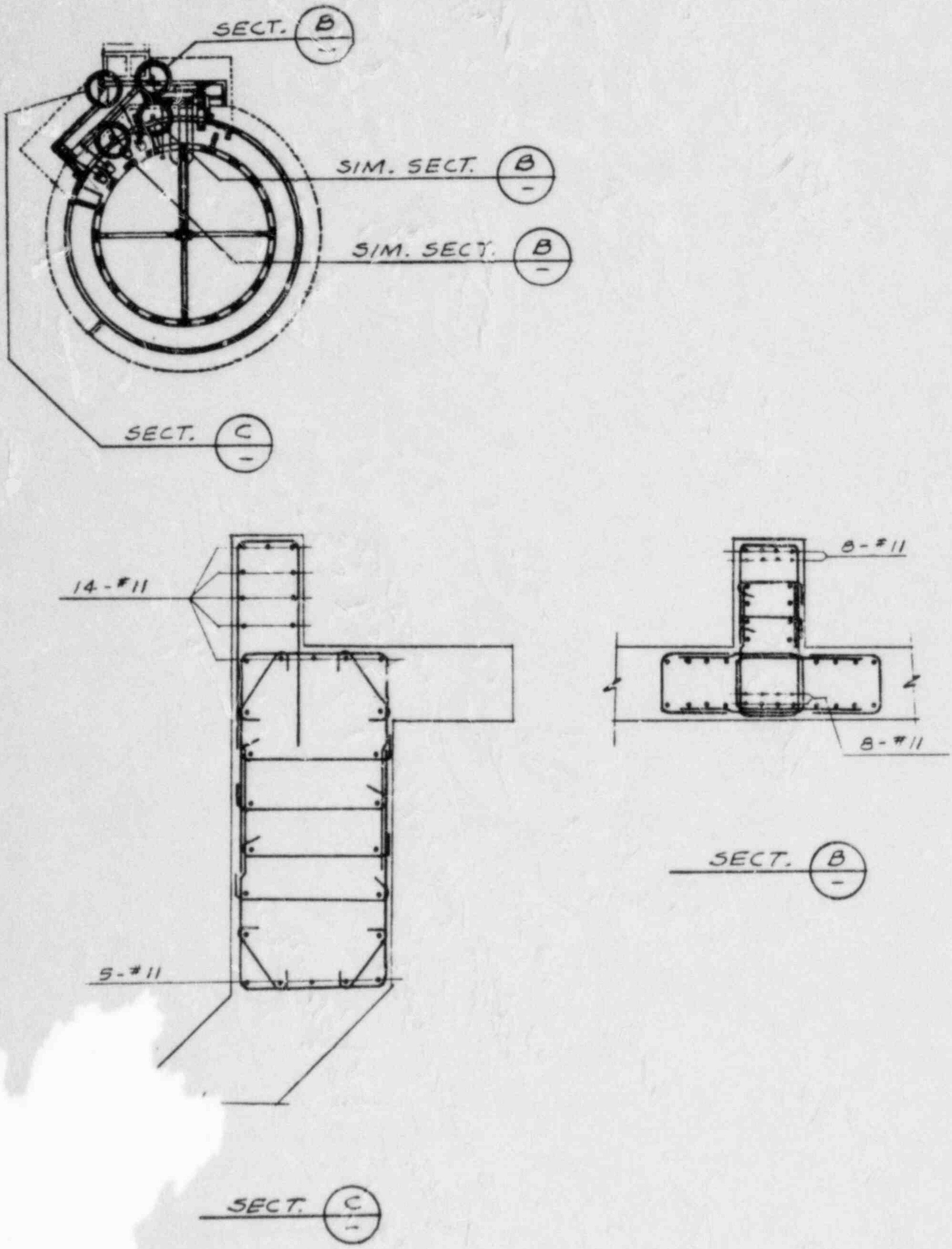


Figure 27
CONCRETE REINFORCING DETAILS
FOR NSCW VALVE HOUSE
(Sheet 2 of 2)

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.

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DESIGN REPORT

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.

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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).

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

where:

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

E_k = missile kinetic energy (ft-lb).

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

V_s = missile striking velocity (ft/s).

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

---

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 (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.

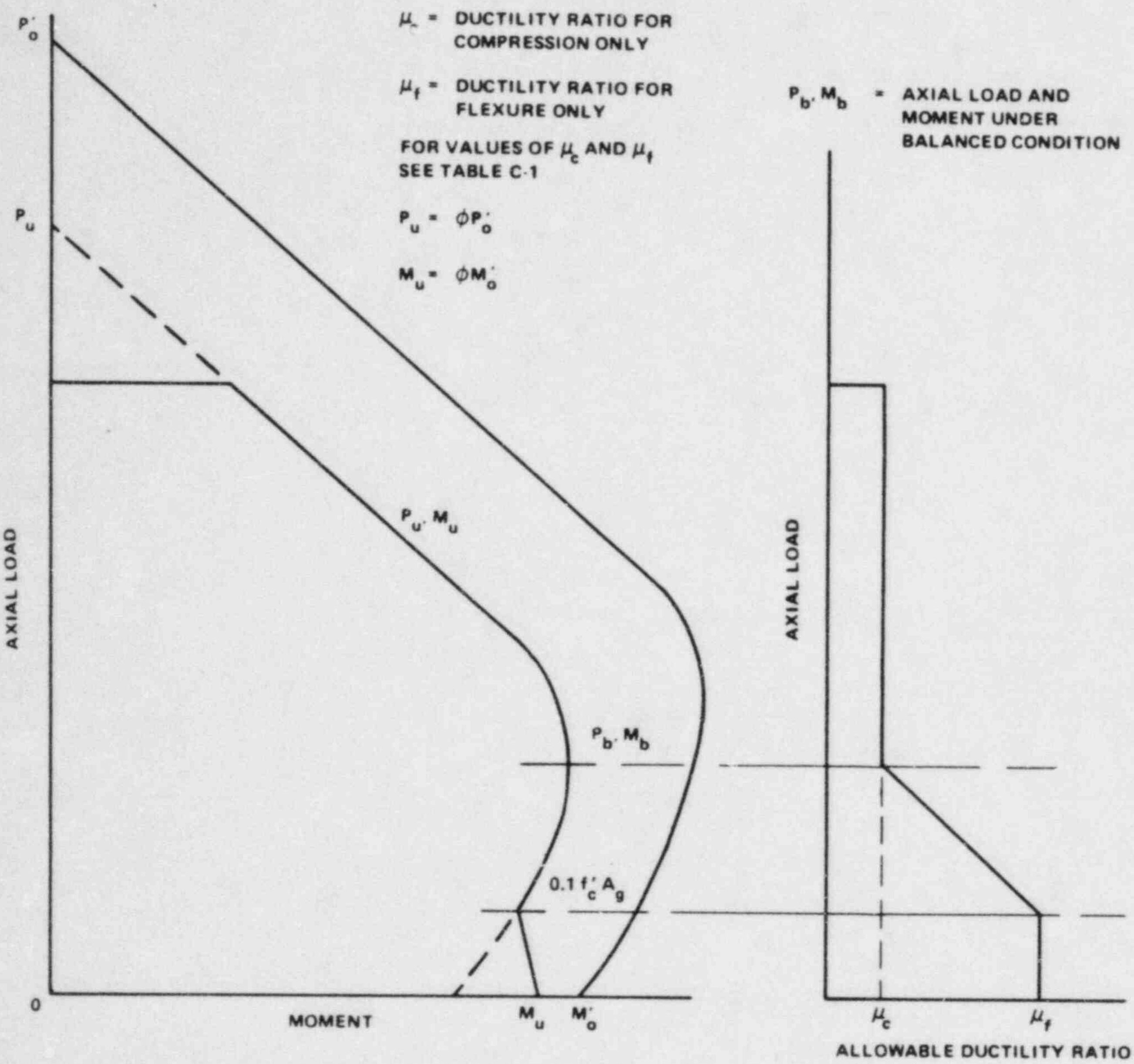
(3) Ductility ratio up to 10 can be used without an angular rotation check.

(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