VOGTLE ELECTRIC GENERATING PLANT

GEORGIA POWER COMPANY

CATEGORY 1 TUNNELS

DESIGN REPORT

Prepared

by

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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 Category 1 tunnels, in order to assist in planning and conducting a structural audit. Quantitative information is provided regarding the scope of the actual design computations and the final design results.

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

2.0 DESCRIPTION OF STRUCTURES

2.1 GENERAL DESCRIPTION

There are 17 tunnels in the two unit VEGP design. Eight are associated with Unit 1, seven with Unit 2, and two are common to both units. Thirteen of the tunnels house safety-related systems and components and are designed to all Category 1 requirements. Due to their proximity to and interface requirements with other safety-related structures, the remaining four tunnels, which do not house any safety-related systems or components, are designed to maintain their structural integrity under earthquake and tornado conditions. This report is limited to the 13 seismic Category 1 tunnels housing safety-related systems and components. A summary of these tunnels is given below. All of the tunnels are constructed of reinforced concrete and are placed within and founded on densely compacted sand and silty sand Category 1 backfill.

Tunnel	Number of Tunnels	Unit
1. Nuclear Service Cooling Water (NSCW)	4	1, 2
2. Diesel Generator Piping	4	1, 2
3. Diesel Generator Electric	2	1, 2
4. Auxiliary Feedwater	2	1, 2
5. Turbine Electric	1	Common
Total	13	

The primary function of the tunnels is to house electric circuits and/or piping systems routed between the various plant structures. Each has a unique set of requirements which contribute to its layout, configuration, and design. These include in-service inspection of safety-related piping, plant security, personnel access, ventilation, and high-energy line effects. A description of the primary functions of each tunnel follows.

2.1.1 NSCW Tunnels

The four NSCW tunnels (two per unit, trains A and B) serve to protect, house, and support safety-related piping and electric circuits between the NSCW valve houses, refueling water storage tanks, and reactor makeup water storage tanks and the auxiliary building and diesel generator piping tunnels. These are enclosed rectangular tunnel structures located below grade except for the tank interface structures which rise above grade next to the tanks. A center wall divides the tunnel into two chambers in order to separate the piping and electric circuits.

2.1.2 Diesel Generator Piping Tunnels

The four diesel generator piping tunnels (two per unit, trains A and B) primarily serve to protect, house, and support the safety-related cooling water supply and return piping between the diesel generator building and the auxiliary building and NSCW tunnels. These are enclosed rectangular tunnel structures located below grade except for the associated ventilation shafts.

2.1.3 Diesel Generator Electric Tunnels

The two diesel generator electric tunnels (one per unit) serve to protect, house, and support the train A and B safety-related electric circuits between the control and diesel generator buildings. These tunnels are divided into two chambers by a continuous center wall in order to maintain separation between train A and B components. These are enclosed rectangular tunnel structures located below grade, except for the associated ventilation shafts at the control buildings and access shafts at the diesel generator buildings.

2.1.4 Auxiliary Feedwater Tunnels

The two auxiliary feedwater tunnels (one per unit) serve to protect, house, and support the safety-related piping between the auxiliary feedwater pumphouses and the auxiliary and control buildings. These are partially double chambered where required to isolate independent trains. These are enclosed rectangular tunnel structures, located with their roofs above grade. Special ventilation shafts are provided for pressure release due to high energy lines. The main steam tunnel has a section which allows the auxiliary feedwater tunnel to cross over it.

2.1.5 Turbine Electric Tunnel

The turbine electric tunnel (one tunnel common to both units) serves as the transition structure to protect, house, and support electric circuits between the control and turbine buildings. A divider wall at the plant centerline separates the units. It is an enclosed rectangular tunnel structure located below grade.

2.2 LOCATION AND FOUNDATION SUPPORT

All Category 1 structures are founded within the area of the power block excavation. The excavation removed in-situ soils to elevation 130'± where the marl bearing stratum was encountered. All Category 1 structures are located either directly on the marl bearing stratum or on Category 1 backfill placed above the marl bearing stratum. The backfill consists of densely compacted select sand and silty sand. The nominal finished grade elevation is 220'-0". The high groundwater table is at elevation 165'-0".

All Category 1 tunnels are placed within and founded on Category 1 backfill. They are buried at various depths as required by their layout in relation to the other structures. Location drawings for Units 1 and 2 are provided in figures 1 and 2.

2.3 GEOMETRY AND DIMENSIONS

Developed elevations, as cut in figures 1 and 2, along with typical cross-sections for each tunnel, are shown in figures 3 through 8.

2.4 KEY STRUCTURAL ELEMENTS

Each tunnel is a box section with either one or two chambers. The walls, roofs, and floors are continuous except for the roof of the auxiliary feedwater tunnel. Seismic separation gaps of 3 inches minimum are provided at all interfaces with other structures. The primary structural elements are, therefore, the floor, walls, and roofs. The cross-section and length of each tunnel containing safety-related components is investigated for seismic wave propagation effects.

2.5 MAJOR EQUIPMENT

There is no major equipment located in any of the tunnels. There are safety-related heating, ventilating and air conditioning (HVAC) fans and motors located in the access shafts of the NSCW and diesel generator electric tunnels. There are various sumps and sump pumps as required.

2.6 SPECIAL FEATURES

2.6.1 Pressure Relief Shafts

Pressure relief shafts are provided on the auxiliary feedwater tunnels due to the high-energy lines. These shafts allow direct venting to the atmosphere while maintaining the security and tornado missile integrity of the tunnel.

2.6.2 Removable Covers

Removable covers are provided on the auxiliary feedwater tunnel to allow access for in-service inspection of critical piping. The covers are designed to Category 1 criteria and are bolted down to prevent them from being lifted by internal pressurization.

3.0 DESIGN BASES

3.1 CRITERIA

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

3.1.1 Codes and Standards

- American Concrete Institute (ACI), Building Code Requirements for Reinforced Concrete, ACI 318-71, including 1974 Supplement.
- American Institute of Steel Construction (AISC), Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, adopted February 12, 1969, and Supplements No. 1, 2, and 3.

3.1.2 Regulations

- 10 CFR 50, Domestic Licensing of Production and Utilization Facilities.

3.1.3 General Design Criteria (GDC)

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

3.1.4 Industry Standards

Nationally recognized industry standards such as American Society for Testing and Materials (ASTM), American Concrete Institute, and American Iron and Steel Institute (AISI) are used to specify material properties, testing procedures, fabrication, and construction methods.

3.2 LOADS

The basic loads applicable for consideration in design of the tunnels are individually discussed below. A summary of load term definitions is provided in Appendix A. Additional construction loads from sources such as cranes and transporters have been considered but are not discussed as they are not coincident with those postulated during plant operation.

3.2.1 Normal Loads

3.2.1.1 Dead Loads (D)

• Reinforced concrete	150 pcf
• Subsystems (piping, cable tray, and conduit applied to walls and roofs where applicable	50 psf
• Soil weight directly supported by tunnels	126 pcf

3.2.1.2 Live Loads (L)

• Concentrated load applied to slabs (applied to maximize moment and shear) to provide design margin for additional support and construction loads	5 kips
• Distributed load on roofs (applied directly to roof or as a soil surcharge)	250 psf
• Distributed load on floors	100 psf
• Lateral at-rest soil pressure	$.7\gamma_m H$ (refer to section 3.4.6)

- Surcharge effects due to an AASHTO HS20-44 truck

3.2.1.3 Operating Thermal Loads (T_o)

- No long duration temperature differentials are anticipated on the structural elements
- Embedment of the tunnels will moderate the extreme ambient temperatures.

3.2.1.4 Pipe Reactions (R_o)

The local effect of pipe reactions have been considered.

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 for the Category 1 tunnels are discussed in the Seismic Analysis Report. The horizontal and vertical structure OBE accelerations are shown in table 1. As all key structural elements are comprised of reinforced concrete, the OBE damping value, as a percentage of critical damping, applicable to the Category 1 tunnels is 4 percent.

The basic OBE seismic wave particle acceleration and corresponding particle velocities applicable to the analysis and design for wave propagation effects are as follows:

- Particle acceleration, A_m = 46 in./sec^2
- Compression wave particle velocity, v_{mp} = 3.6 in./sec
- Shear wave particle velocity, V_{ms} = 7.2 in./sec
- Surface wave particle velocity, V_{mr} = 7.2 in./sec

Dynamic lateral earth pressures are developed by the Mononabe-Okabe analysis of dynamic pressures in dry cohesionless materials. The dynamic incremental soil pressure profile is shown in figure 9.

3.2.2.2 Design Wind (W)

The applicable wind load is the 100-year mean recurrence interval 110 mph wind per ANSI A58.1-1972 (reference 1). Coefficients are per Exposure C, applicable to flat open country. The effective wind velocity pressure profile is shown in table 2.

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 for the Category 1 tunnels are discussed in the Seismic Analysis Report. The horizontal and vertical structure SSE accelerations are shown in table 1. As all key structural elements are comprised of reinforced concrete, the applicable SSE damping value, as a percentage of critical damping, applicable to the Category 1 tunnels is 7 percent.

The basic SSE seismic wave particle acceleration and corresponding particle velocities applicable to the analysis and design for wave propagation effects are as follows:

- Particle acceleration, A_m = 77 in./sec^2
- Compression wave particle velocity, V_{mp} = 6.0 in./sec
- Shear wave particle velocity, V_{ms} = 12.0 in./sec
- Surface wave particle velocity, V_{mr} = 12.0 in./sec

Dynamic lateral earth pressures are developed by applying the Mononabe-Okabe analysis of dynamic pressures in dry cohesionless materials. The dynamic incremental soil pressure profile is shown in figure 9.

3.2.3.2 Tornado Loads (W_t)

Loads due to the design tornado include wind pressures, atmospheric pressure differentials, and tornado missile strikes. Tornado wind, pressure drop, and missile effects are applied to all parts of the tunnels extending above grade. Tunnel sections below grade are evaluated for pressure drop and tornado missile effects. The design tornado parameters, which are in conformance with the Region I parameters defined in Regulatory Guide 1.76, are as follows:

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

The tornado effective velocity pressure profile is provided in table 2, and is in accordance with reference 2.

The Category 1 tunnels are partially vented structures. Conservatively, all walls and roofs are designed for a tornado pressure effect of ±3 psi. The tornado missile parameters applied to the Category 1 tunnels are listed in table 3. Missile trajectories up to and including 45 degrees off of horizontal use the listed horizontal velocities. Those trajectories greater than 45 degrees use the listed vertical velocities.

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

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

3.2.3.3 Probable Maximum Precipitation, PMP (N)

PMP loads are only applicable to the roofs of the access shafts of the diesel generator electric and NSCW tunnels. Special roof scuppers are provided with sufficient capacity to ensure that the depth of ponding water due to the PMP rainfall does not exceed 18 inches. This results in an applied PMP load of 94 psf.

3.2.3.4 Blast Load (B)

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

3.2.4 Abnormal Loads

There are high-energy lines in the auxiliary feedwater and diesel generator piping tunnels. Pressure and temperature time-histories, pipe reactions, jet impingement, and pipe impact loads have been developed for postulated breaks and are applied to the tunnels as applicable.

3.3 LOAD COMBINATIONS AND STRESS/STRENGTH LIMITS

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

3.4 MATERIALS

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

3.4.1 Concrete

- Compressive strength $f'_c = 4.0$ ksi
- Modulus of elasticity $E_c = 3834$ ksi
- Shear modulus $G = 1440$ ksi
- Poisson's ratio $\upsilon = 0.17\text{-}0.25$

3.4.2 Reinforcement

3.4.2.1 ASTM A615 Grade 60

- Minimum yield stress — F_y = 60.0 ksi
- Minimum tensile strength — F_{ult} = 90.0 ksi
- Minimum elongation — = 7-9% in 8 inches

3.4.2.2 Welded Wire Fabric - ASTM A185

- Minimum yield stress — F_y = 56.0 ksi
- Minimum tensile strength — F_{to} = 70.0 ksi

3.4.3 Structural Steel - ASTM A36

- Minimum yield stress — F_y = 36.0 ksi
- Minimum tensile strength — F_{ult} = 58.0 ksi
- Modulus of elasticity — E_s = 29,000 ksi

3.4.4 Structural Bolts - ASTM A325 (1/2 inch to 1 inch inclusive)

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

3.4.5 Anchor Bolts and Headed Anchor Studs

3.4.5.1 ASTM A36:

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

3.4.5.2 ASTM A108:

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

3.4.5.3 ASTM A307:

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

3.4.6 Foundation Media

The Category 1 tunnels are founded in densely compacted sand and silty sand Category 1 backfill. The design parameters of the Category 1 backfill are as follows.

3.4.6.1 General Description

See section 2.2

3.4.6.2 Category 1 Backfill Properties

- Moist unit weight: $\gamma_m = 126$ pcf
- Saturated unit weight: $\gamma_t = 132$ pcf
- Shear modulus:

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

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

3.4.6.3 Seismic Wave Propagation Parameters

- Poisson's ratio, $\mu = 0.4$
- Friction coefficient, $\mu_f = 0.5$
- Compression wave propagation velocity, $C_p = 7400$ ft/sec
- Shear wave propagation velocity, $C_s = 2000$ ft/sec
- Surface wave propagation velocity, $C_r = 2000$ ft/sec

The wave propagation velocities are average values for depths between 400 and 500 feet.

4.0 STRUCTURAL ANALYSIS AND DESIGN

This section provides the methodologies employed to analyze and design the key structural elements of the Category 1 tunnels using the applicable loads and load combinations specified in section 3.0.

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

Two independent analyses are performed on the Category 1 tunnels, with the results combined as appropriate. The first analysis consists of analyzing typical transverse cross-sections of each tunnel as closed frames, primarily for out-of-plane bending and shear effects. The second analysis consists of analyzing the gross tunnel longitudinally to account for seismic wave propagation effects.

The structural analyses are primarily performed by manual calculations using standard structural analysis techniques. The analysis techniques, boundary conditions, and applications of loads for each of the key structural elements are provided to illustrate the method of analysis. Representative analysis and design results are provided for each key structural element to illustrate the final analysis and design results.

4.1 SELECTION OF GOVERNING LOAD COMBINATION

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

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

4.2 COMBINED EFFECTS OF THREE COMPONENT EARTHQUAKE LOADS

The combination of co-directional responses due to three component earthquake effects for the transverse analysis is performed using the Square Root of the Sum of the Squares (SRSS) method, i.e.,

$R = \left(R_i^2 + R_j^2 + R_k^2\right)^{1/2}$ or the Component Factor method, i.

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

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

4.3 TRANSVERSE ANALYSIS AND DESIGN METHODOLOGY

The transverse section of each tunnel is analyzed for the effects of out-of-plane loadings. The primary loads considered are those that exert inward pressures on the tunnel cross-section. Each tunnel cross-section is evaluated on the basis of the controlling combinations of design load intensity, span and tunnel configuration along the tunnel length. Each cross-section is modeled as a two-dimensional closed frame. Each frame is analyzed employing classical beam formulas and moment distribution techniques to determine the maximum moments and shears.

The primary pressure loads considered are the static at-rest soil pressure, horizontal and vertical seismic inertial loads due to the structure mass, the lateral dynamic incremental seismic soil pressure, lateral and vertical surcharges from adjacent structures, and surface live load surcharge effects. These loads are applied as equivalent static uniform or linearly varying pressure loads.

The transverse reinforcing steel is proportioned and detailed to ACI 318 Code requirements. In general, the maximum reinforcement determined for a governing face of a key structural element is provided on both faces.

Appropriate design consideration is given to large openings in any of the key structural elements. Typical analysis and design results for the key structural elements in selected tunnels is provided in table 4.

4.4 LONGITUDINAL ANALYSIS AND DESIGN METHODOLOGY

The longitudinal analysis and design of underground tunnels to account for seismic wave propagation effects considers the following:

- Axial tension and compression due to traveling seismic wave
- Shear and bending due to traveling seismic wave
- Strain caused by dynamic differential movement at tunnel connections and bends

Analytical procedures for evaluating these effects are described in reference 3. For very long structures, the procedures are based on the assumption that there is no relative motion between the flexible structure and the ground. Seismic stresses in the tunnel are estimated from the calculated strains and curvature in the surrounding soil due to the passage of seismic waves. For short structures, slippage may occur between the tunnel and the soil and the calculated axial stresses are proportionately less than those assuming strain in the tunnel equal to the maximum soil strain. The effects of bends are evaluated using procedures based on equations for beams on elastic foundations. The calculated seismic stresses from the longitudinal analysis are combined with stresses from the transverse loading conditions to confirm that design integrity is maintained.

The amount of stress or strain induced in a tunnel section due to seismic wave propagation is proportional to the uninterrupted length and the frictional force developed per unit length. Four tunnel configurations are evaluated in detail which envelope these parameters. The tunnels evaluated are the Unit 1 NSCW tunnel (train B), Unit 2 diesel generator piping tunnel (train A), Unit 1 auxiliary feedwater tunnel (train A), and the common turbine electric tunnel. All other tunnel configurations have less severe combinations of uninterrupted length and frictional force per unit length and therefore have lower values of induced seismic stress or strain than the cases analyzed.

4.4.1 Stresses in Straight Sections

The portions of a long tunnel that are sufficiently removed from the influences of end boundary conditions, and that are free of any external support other than the surrounding soil, are assumed to be flexible and to follow essentially the displacements and deformations of the soil during seismic ground motion. Soil displacements due to the passage of shear, compression, and surface waves are calculated based on wave propagation velocities and the maximum ground particle acceleration and velocity due to the design earthquake. The apparent stresses in the uncracked tunnel are calculated using the resulting strain, curvature, and modulus of elasticity of the structural material.

Wave propagation velocities are calculated in accordance with reference 3 where effective propagation velocities are taken as the propagation velocity of the underlying competent soil or rock. For VEGP, the estimated propagation velocities at a depth of 450 feet are used. The maximum ground particle acceleration for the OBE and SSE earthquake motion is taken as the peak ground acceleration. Maximum shear wave and surface wave particle velocities are based on scaling the results of an integration of the VEGP acceleration time history which gives a maximum particle velocity of 5 feet per second for a 1.0 g maximum ground acceleration. The maximum particle velocity for a compression wave is conservatively taken as one-half the maximum shear wave particle velocity.

In the case of a straight tunnel, the transfer of soil strain as axial strain into the tunnel depends on the end bearing of the element against the soil and the frictional resistance between the element surface and the soil. Neglecting end bearing, the minimum length of structure required to develop full friction is twice the maximum slippage length which is calculated in accordance with reference 3. For tunnels where the total length is less than twice the maximum slippage length, the tunnel will displace relative to the surrounding soil due to strain incompatibility between the soil and the tunnel element, and the calculated axial stresses will be proportionately less than those calculated assuming no relative slippage between the tunnel and the soil.

The frictional force per unit length of tunnel of rectangular cross section is the sum of the frictional forces acting on each face and is given by

$$f = \Sigma P (\mu_f)$$

where P = total soil force acting on a tunnel face per unit length

μ_f = coefficient of friction

4.4.2 Stresses at Bends

The analysis of tunnels with bends or restrained ends is based on the equations for beams on elastic foundations derived by

Hetenyi (reference 4). In the case of a bend, the transverse leg is assumed to deform as a beam on an elastic foundation due to the axial force in the longitudinal leg. The displacement at the bend is defined by the overall spring constant at the bend which depends on the stiffness of the longitudinal and transverse legs as well as the degree of fixity at the bend and at the far ends of both legs. The stiffness of the leg is classified (according to reference 4) as rigid, intermediate, or flexible.

The effective slippage length at the bend is calculated based on the unit frictional force, spring constant at the bend, maximum ground strain, and the cross-sectional properties of the tunnel. Having the effective slippage length, the displacement at the bend is calculated. With the displacement, the shear and moment in the transverse leg are calculated for the appropriate configuration.

4.4.3 Evaluation of Seismic Wave Incidence Angle

The maximum ground velocity and acceleration for an earthquake motion contain contributions from compressional, shear, and surface waves. Since it is not possible to determine the relative contribution of each of the various wave types to the total ground motion, and since the maximum values are not likely to occur simultaneously, the axial and bending stresses are calculated separately, according to wave type and angle of incidence, and the maximum values for each wave type are combined by the SRSS method. The maximum combination of axial and bending stress for an angle of incidence between 0° and 45° is used for design. The governing angle of incidence is determined to be zero degrees. The equations used are provided in table 5.

4.4.4 Application of Methodology

For each case analyzed, the seismically induced stresses and displacements are calculated at bends, free ends, and other critical locations. The results of these calculations, which are based on the assumption of an uncracked transverse section, are provided in table 6.

The forces and strains computed are secondary (displacement controlled) forces and strains. Load factors and stress limits based on stress generated by mechanical loads are not applicable in this case. Seismically induced stresses based on the uncracked concrete section reduce as strain increases and the concrete cracks. The maximum amount of strain that can occur in the tunnel is limited to the maximum seismically induced ground strain.

Evaluation of seismic effects, therefore, depends on whether or not the concrete tunnel is expected to crack as a result of the earthquake ground motion.

If the maximum calculated seismically induced combined axial and bending stress at any point in the tunnel does not exceed the tensile strength (modulus of rupture) of the concrete, the tunnel is assumed to remain uncracked. Since the longitudinal stiffness of the tunnel is based on the full, uncracked concrete section, seismically induced strains in the tunnel are minimized, and the relative displacement between the tunnel and soil are maximized. The calculated displacements are, therefore, compared with gaps and clearances that are provided to assure that any relative movement is within tolerable limits.

If the maximum calculated seismically induced combined axial and bending stress at any point exceeds the modulus of rupture, the concrete is assumed to crack. The longitudinal reinforcing steel in each of the tunnels is selected based on the minimum ACI 318 Code requirements for temperature and shrinkage steel. These reinforcing bars are distributed over the zone of concrete tension, and carry the full tensile force, thus ensuring that only fine closely spaced cracks develop. In this case, strain in the reinforcing steel is distributed along the length of the tunnel, and the average strain is checked to assure that it is well within the allowable ductile limit of the steel.

When cracking occurs at a bend, and is primarily the result of bending moment calculated on the basis of an uncracked section, the bending moment is recalculated using a cracked moment of inertia for the tunnel section. Since the bending moment is strain-induced, the resulting increase in strain at the bend significantly reduces the induced bending moment. The resulting strain at the bend is then checked to assure that it is within the allowable ductility limit of the steel.

4.4.5 Analysis and Design Results

A list of the major results of the longitudinal tunnel evaluation are itemized as follows:

A. Calculated seismically induced strain in the long leg portion of all tunnels under both OBE and SSE conditions is less than the cracking strain of concrete. However, even if the concrete were to crack, all tunnel cross-sections are adequately reinforced with well-distributed rebar such that strain would be distributed along the length of the tunnel. The maximum ground strain (SSE) is calculated to be

$$\varepsilon_m = 5.24 \times 10^{-4}$$

B. It is found that this maximum ground strain cannot be transferred to the tunnels since all tunnels are shorter than twice the calculated maximum slippage length. However, even if this maximum ground strain could be transferred to a cracked tunnel section, the average rebar strain would be far less than the yield strain of the steel (2×10^{-3}).

C. At bends, the calculated strain due to axial force and bending moment is found to exceed the cracking strain of concrete in several cases. However, using the reduced bending moment due to the cracked section, the rebar does not yield.

D. The calculated maximum displacement of the tunnels relative to the soil at free ends and at bends under both OBE and SSE conditions is considerably less than the minimum gaps provided.

4.5 COMBINATION OF TRANSVERSE AND LONGITUDINAL ANALYSES AND DESIGN DETAILS

In general, the axial and bending stresses due to seismic wave propagation affect only the longitudinal tunnel steel reinforcement and concrete stresses, while the stresses due to the transverse analysis affect only the transverse steel reinforcement and concrete stresses. However, at bends, the axial movement of the long leg induces lateral earth pressures on the inside wall of the short leg. These pressures are included in the cross-sectional analysis when significant.

Typical reinforcing details for each of the Category 1 tunnels is provided in figures 10 through 14.

5.0 MISCELLANEOUS ANALYSIS AND DESIGN

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

5.1 STABILITY ANALYSIS

Overall safety factors for stability of the Category 1 tunnels are not calculated, as significant sliding or overturning cannot occur under design load conditions. Also since the foundation levels (the lowest foundation elevation is elevation 174'-7") are above the high water table (elevation 165'-0"), the Category 1 tunnels are not subjected to flotation effects.

5.2 TORNADO LOAD EFFECTS

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

Controlling roof and exterior wall panels are evaluated for tornado load effects, and the localized response is combined with the analysis results of the overall structural response, as applicable, to confirm that design integrity is maintained. Additional reinforcing steel is provided, if necessary, to satisfy design requirements in accordance with the ACI 318 Code.

In addition, barriers are provided for the openings in the exterior walls or roofs unless the systems or components located in the exterior rooms are nonsafety-related. In this case, the interior walls and slabs are treated as barriers for the safety-related systems or components located in the interior rooms. Any openings in the exterior walls or slabs and the interior walls or slabs that may be susceptible to missile entry are evaluated to ensure that no safety-related systems or components are located in a potential path of the missile.

The methodology used to analyze and design the structural elements to withstand the tornado load effects is described in reference 2. Specific procedures used for analysis of missile impact effects are described in Appendix C.

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

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

6.0 CONCLUSION

The analysis and design of the Category 1 tunnels includes all credible loading conditions and complies with all applicable design requirements.

7.0 REFERENCES

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

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

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

4. Hetenyi, M., "Beams on Elastic Foundation," University of Michigan, 1946.

TABLE 1

CATEGORY 1 TUNNELS SEISMIC ACCELERATION VALUES

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

TABLE 2

WIND AND TORNADO EFFECTIVE VELOCITY
PRESSURE PROFILES

Wind Pressures

- All parts of tunnels within 30 feet above grade - 32 psf

- All parts of tunnels above 30 feet above grade - 40 psf

Tornado Pressures

- All parts of tunnels above grade - 292 psf

(Includes a typical size factor, C_s = 0.88)

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' Plank	200	216	200	160
3" Ø std x 10' Pipe	78.5	212	200	160
1" Ø x 3' Steel Rod	8	Unlimited	317	254
6" Ø std x 15' Pipe	285	101	160	128
12" Ø std x 15' Pipe	744	46	150	120
13-1/2" Ø x 35' Utility Pole	1490	30(1)	211	169
Automobile (20-ft^2 Projected Area)	4000	0	75	60

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

TABLE 4

SUMMARY OF TRANSVERSE ANALYSIS AND DESIGN RESULTS (Sheet 1 of 2)

Tunnel	Structural Element	Governing Load Combination Equation	Design Moment Mu (ft-k)	A_s (in.^{2}) Required	A_s (in.^{2}) Provided	Reinforcing
NSCW Tunnel (Train A)	Roof	3	87.2	1.00	1.27	#10 @ 12"
	Wall	3	12.8	0.44(1)	0.44	#6 @ 12"
	Mat	3	90.0	1.04	1.27	#10 @ 12"
NSCW Tunnel (Train B)	Roof	3	43.9	0.49	0.60	#7 @ 12"
	Wall	3	19.8	0.44(1)	0.60	#7 @ 12"
	Mat	3	59.4	0.69	0.79	#8 @ 12"
Diesel Generator Piping Tunnels (Train A)	Roof	3	11.3	0.44(1)	0.44	#6 @ 12"
	Wall	3	25.0	0.44(1)	0.44	#6 @ 12"
	Mat	3	16.2	0.44(1)	0.44	#6 @ 12"
Diesel Generator Piping Tunnels (Train B)	Roof	3	13.3	0.44(1)	0.44	#6 @ 12"
	Wall	3	8.1	0.44(1)	0.44	#6 @ 12"
	Mat	3	17.8	0.44(1)	0.44	#6 @ 12"

(1) Governed by minimum code reinforcing requirements

TABLE 4

SUMMARY OF TRANSVERSE ANALYSIS AND DESIGN RESULTS (Sheet 2 of 2)

Tunnel	Structural Element	Governing Load Combination Equation	Design Moment Mu (ft-k)	A_s (in.^{2}) Required	A_s (in.^{2}) Provided	Reinforcing
Turbine Electric Tunnel	Roof	3	52.3	0.71	0.79	#8 @ 12"
	Wall	3	20.32	0.44(1)	0.44	#6 @ 12"
	Mat	3	86.6	1.00	1.00	#9 @ 12"
Auxiliary Feed-water Tunnel	Removable Covers	9	51.0	0.58	0.62	#5 @ 12"
	Wall	3	0.5	0.44(1)	0.44	#6 @ 12"
	Mat	3	21.3	0.44(1)	0.44	#6 @ 12"
Diesel Generator Electric Tunnel	Roof	3	143.3	1.00	1.00	#9 @ 12"
	Wall	3	105.8	0.74	1.00	#9 @ 12"
	Mat	3	221.3	1.55	1.56	#11 @ 12"

(1) Governed by minimum code reinforcing requirements

TABLE 5

SEISMIC WAVE STRESS EQUATIONS (Sheet 1 of 2)

COMPRESSION WAVE

$$\sigma_{ap} = \pm \frac{EV_{mp}}{C_p} \cos^2\theta$$

$$\sigma_{bp} = \pm \frac{ERA_{mp}}{C_p} \sin\theta \cos^2\theta$$

SHEAR WAVE

$$\sigma_{as} = \pm \frac{EV_{ms}}{C_s} \sin\theta \cos\theta$$

$$\sigma_{bs} = \pm \frac{ERA_{ms}}{C_s} \cos^3\theta$$

SURFACE WAVE

$$\sigma_{ar} = \pm \frac{EV_{mr}}{C_r} \cos^2\theta$$

$$\sigma_{br} = \pm \frac{ERA_{mr}}{C_r} \sin\theta \cos^2\theta$$

COMBINED STRESS DUE TO INDIVIDUAL WAVE TYPES

$$\sigma_a = \pm [\sigma_{ap}{}^2 + \sigma_{as}{}^2 + \sigma_{ar}{}^2]^{1/2}$$

$$\sigma_b = \pm [\sigma_{bp}{}^2 + \sigma_{bs}{}^2 + \sigma_{br}{}^2]^{1/2}$$

Where:

σ_a	=	Axial Stress
σ_b	=	Bending Stress
E	=	Modulus of Elasticity for the Structure
V_{mp}, V_{ms}, V_{mr}	=	Partical Velocity due to Compression Wave, Shear Wave, and Surface Wave Respectively

TABLE 5

SEISMIC WAVE STRESS EQUATIONS (Sheet 2 of 2)

A_{mp}, A_{ms}, A_{mr}	=	Particle Acceleration due to Compression Wave, Shear Wave, and Surface Wave, Respectively
R	=	Distance from the Cross-Sectional Neutral Axis of the Structure to the Extreme Fiber
θ	=	Angle of Incidence of Propagating Wave from the Structure Axis
C_p, C_s, C_r	=	Velocity of Compression Wave, Shear Wave, and Surface Wave, Respectively.

TABLE 6

SUMMARY OF LONGITUDINAL ANALYSIS RESULTS

Tunnel	$\sigma = \sigma_a + \sigma_\Delta$ (psi)				Displacement, Δ (in.)				Exceeds Tensile Capacity of Concrete (1)		Maximum Strain in Reinforcing Steel (in./in.) (2)
	At Bend		Maximum in Straight Section		At Bend		At Free End				
	OBE	SSE	OBE	SSE	OBE	SSE	OBE	SSE	OBE	SSE	SSE
Unit 1 Auxiliary Feedwater Tunnel	467	636	134	154	0.24	0.32	0.52	0.97	No	Yes	0.00190
Unit 2 Diesel Generator Piping Tunnel - Train A	346	560	295	323	0.17	0.28	0.22	0.41	No	Yes	0.00175
Unit 1 NSCW Tunnel Train B	113	170	108	144	0.11	0.16	0.16	0.28	No	No	N/A
Common Turbine Electric Tunnel	N/A	N/A	175	196	N/A	N/A	0.69	1.17	No	No	N/A

(1) Tensile capacity of concrete = $7.5 (f'_c)^{1/2}$ = 474 psi
(2) Yield strain in reinforcing steel = .002 (in./in.)

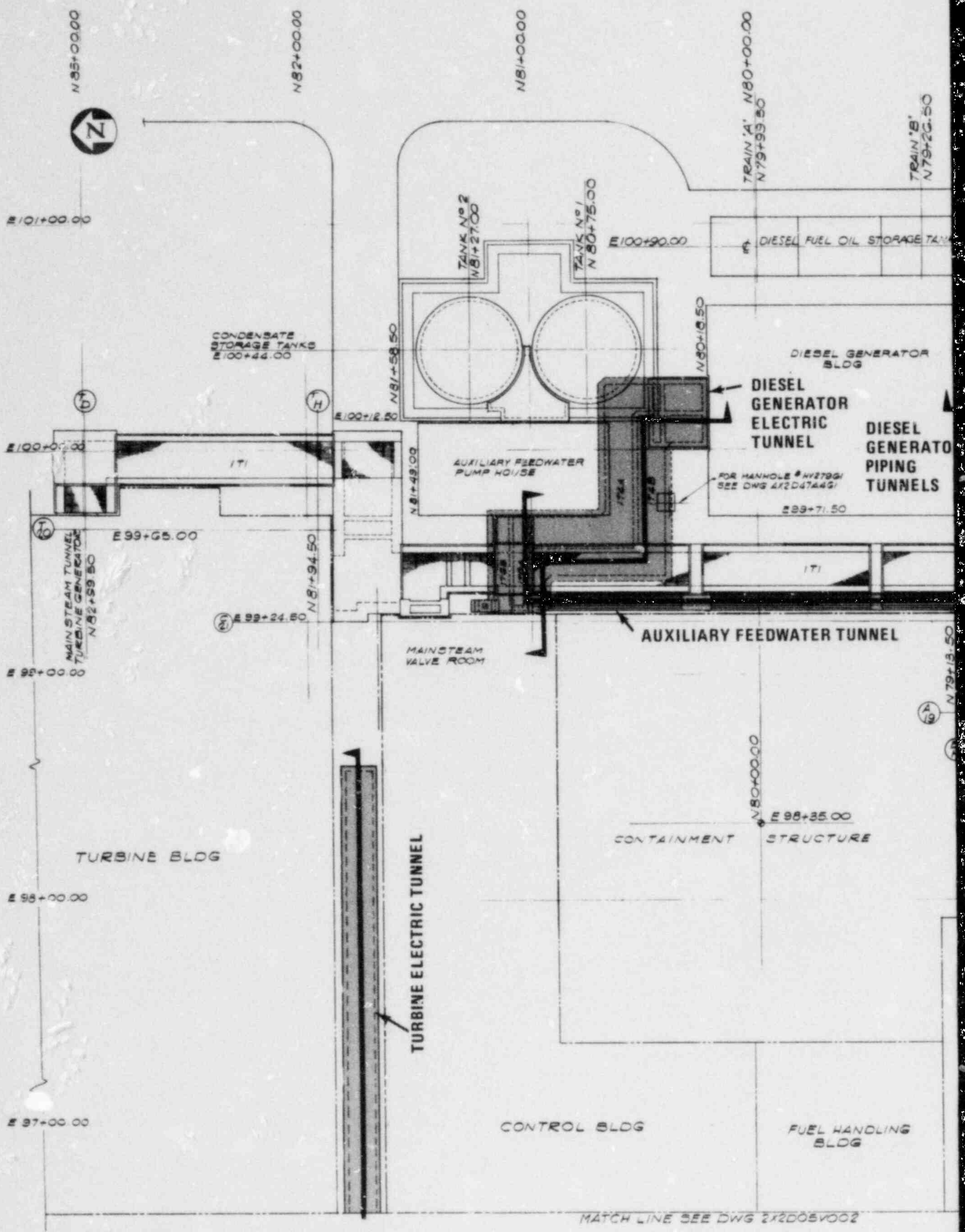
TABLE 7

TORNADO MISSILE ANALYSIS RESULTS[1]

Panel Description and Location	Panel Size			Computed Ductility	Allowable Ductility
	Length (ft)	Width[2] (ft)	Thickness (ft)		
Auxiliary Feedwater Tunnels Removable Covers	7.5	2.8	2.0	5.5	10
Turbine Electric Tunnel Roof	14.0	6.0	2.5	8	10
NSCW Tunnel Roof	12.0	5.0	2.0	3.5	10

(1) Governing load combination of tornado effects is:
$W_t = W_{tg} + .5\ W_{tp} + W_{tm}$

(2) Effective width used for one-way slab analysis.



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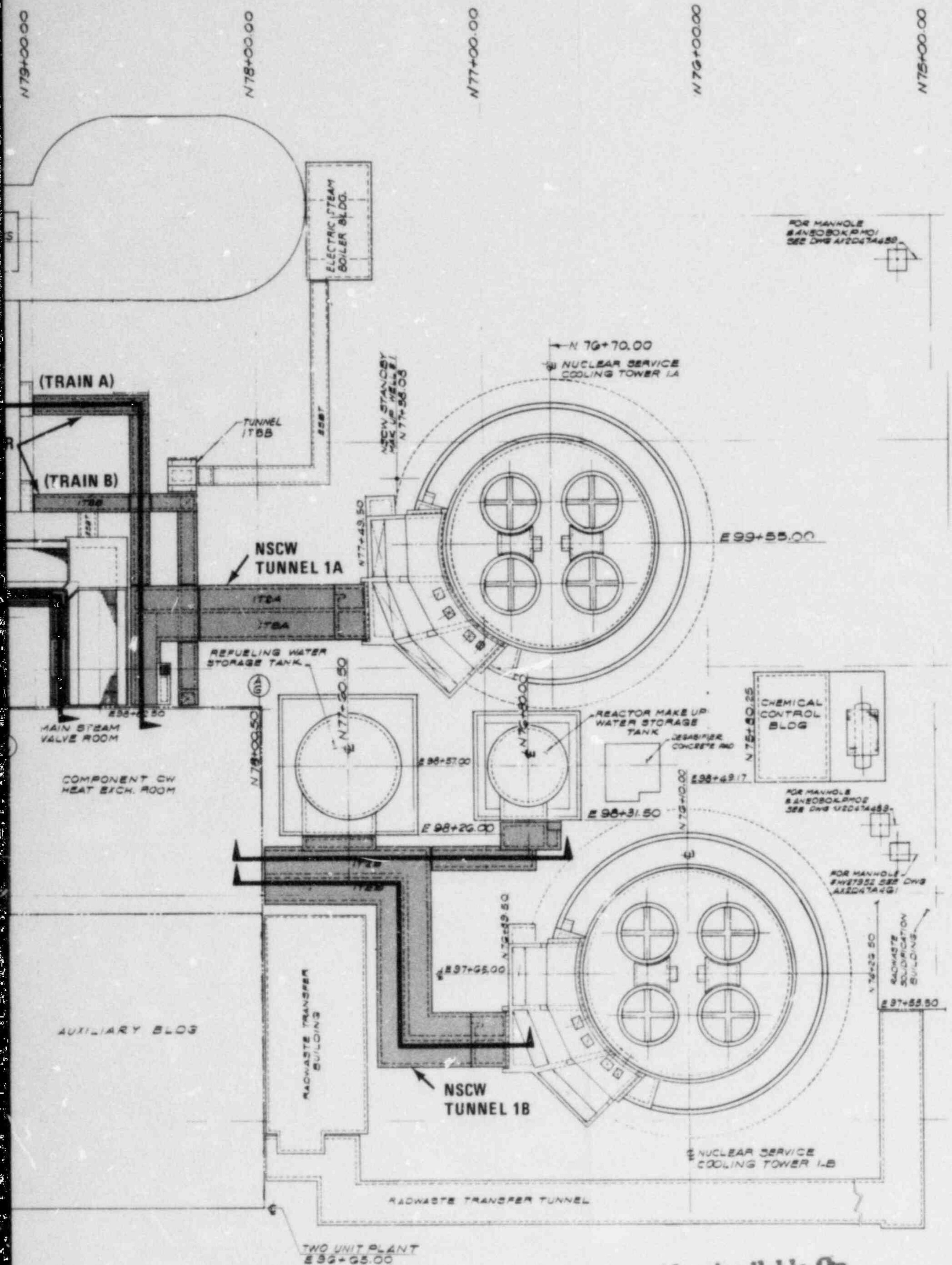
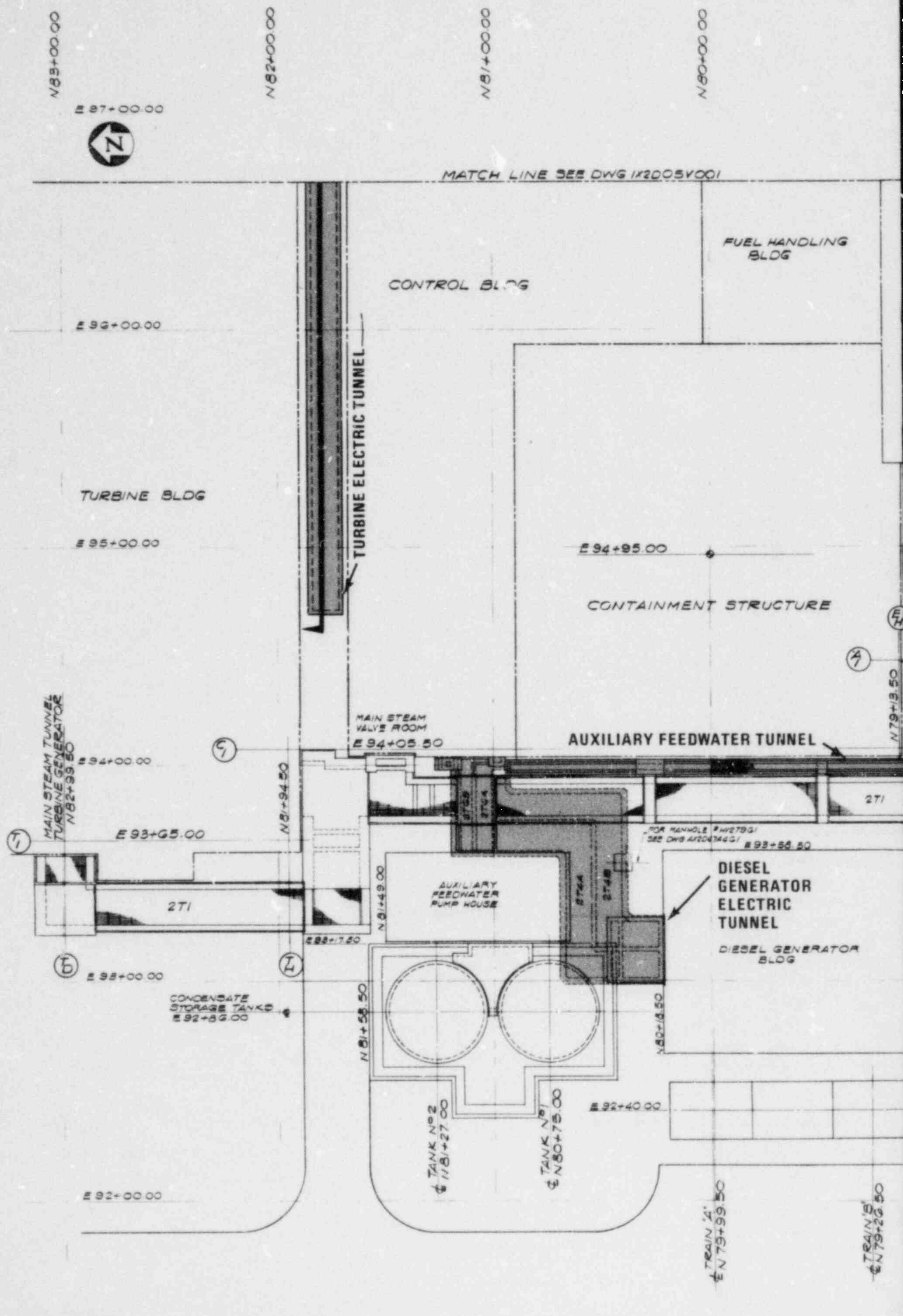
APERTURE
CARD

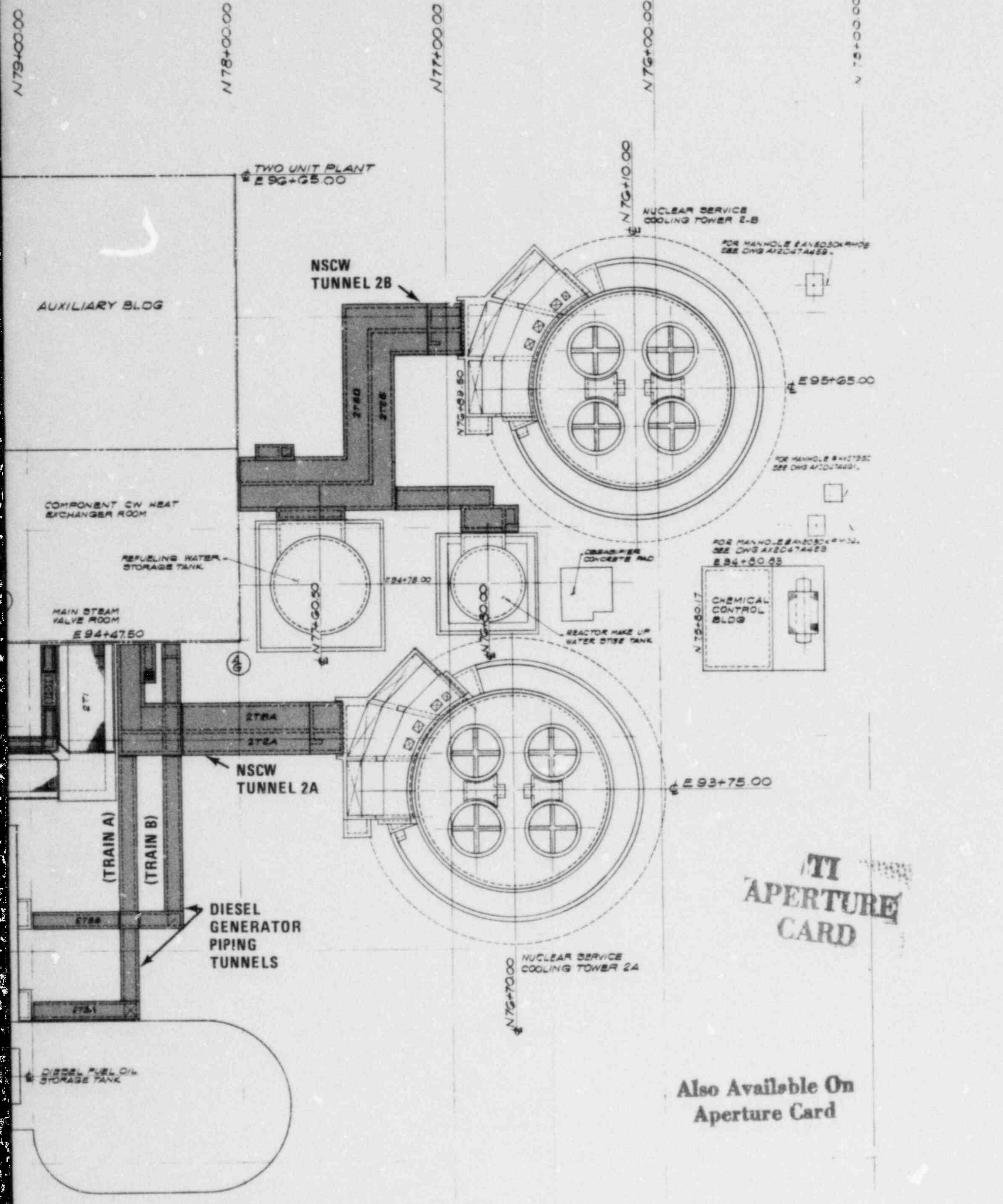


Also Available On
Aperture Card

Figure 1
LOCATION OF CATEGORY 1 TUNNELS – UNIT 1

8411050221-01


N83+00.00
N82+00.00
N81+00.00
N80+00.00
E 97+00.00
N
MATCH LINE SEE DWG 1X2D05V001
FUEL HANDLING BLDG
CONTROL BLDG
E 96+00.00
TURBINE ELECTRIC TUNNEL
TURBINE BLDG
E 95+00.00
E 94+95.00
CONTAINMENT STRUCTURE
N79+13.50
MAIN STEAM VALVE ROOM
E 94+05.50
AUXILIARY FEEDWATER TUNNEL
MAIN STEAM TUNNEL TURBINE GENERATOR N82+99.50
E 94+00.00
N81+94.50
2T1
E 93+65.00
E 93+86.50
AUXILIARY FEEDWATER PUMP HOUSE
N81+49.00
DIESEL GENERATOR ELECTRIC TUNNEL
DIESEL GENERATOR BLDG
E 93+00.00
CONDENSATE STORAGE TANKS
E 92+86.00
N81+58.50
N80+13.50
E 92+40.00
TANK No. 2 N81+27.00
TANK No. 1 N80+75.00
E 92+00.00
TRAIN 'A' N79+99.50
TRAIN 'B' N79+26.50



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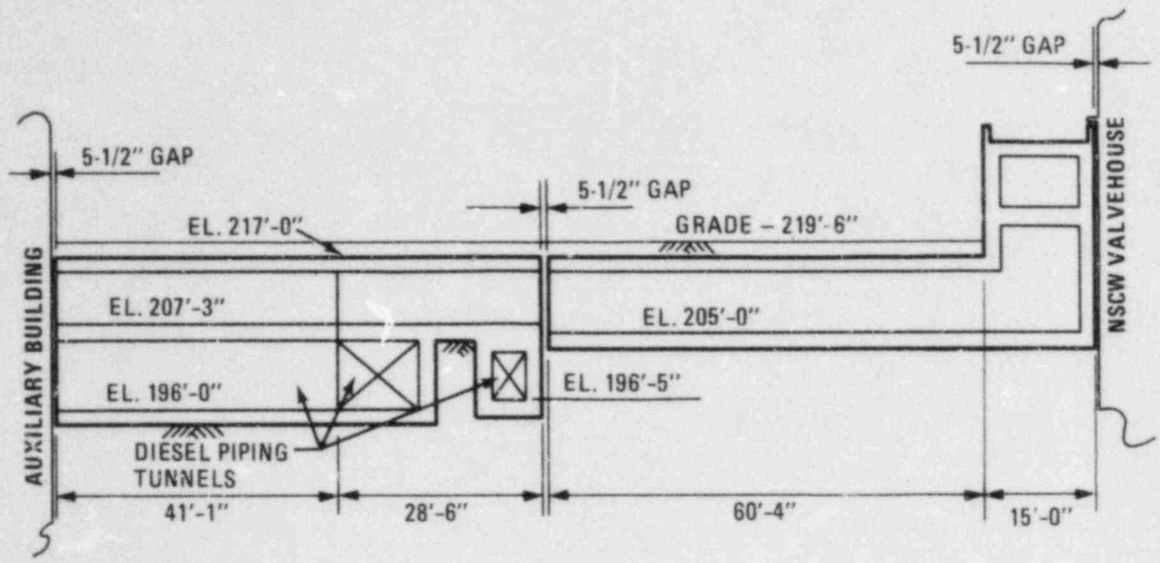
APERTURE
CARD

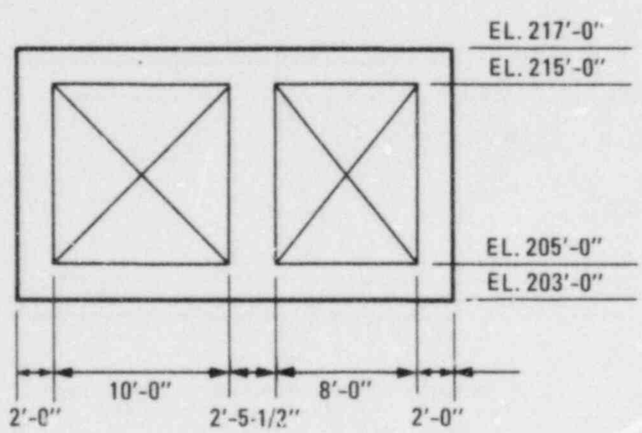
Also Available On
Aperture Card

Figure 2
LOCATION OF CATEGORY 1 TUNNELS – UNIT 2

8411050221-02

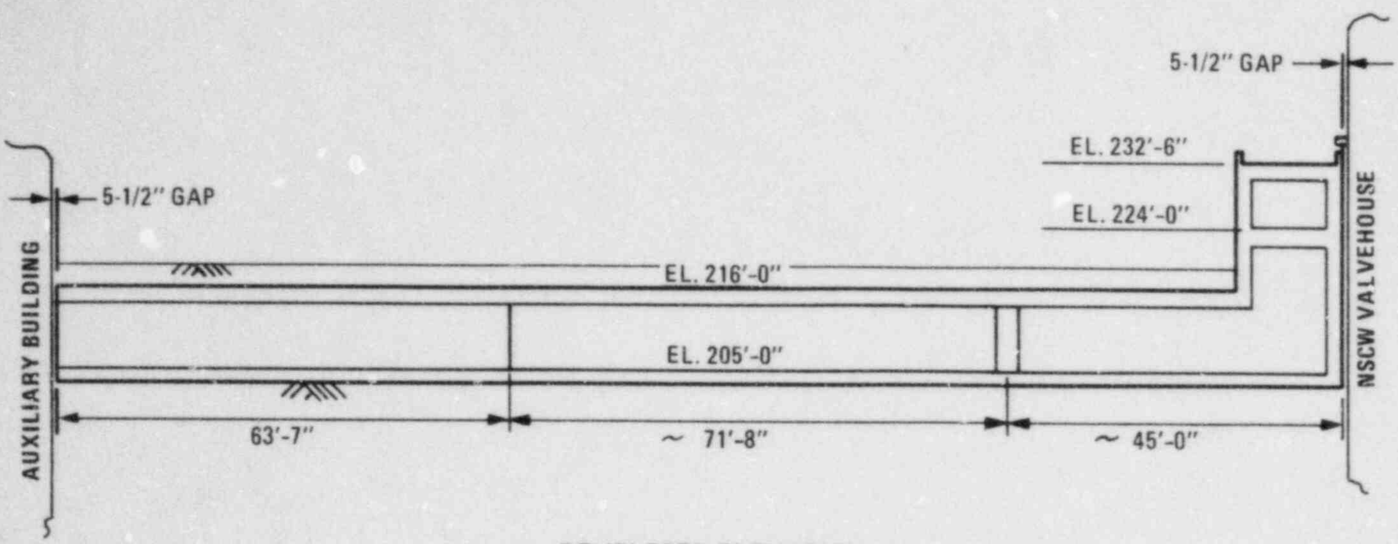


DEVELOPED ELEVATION

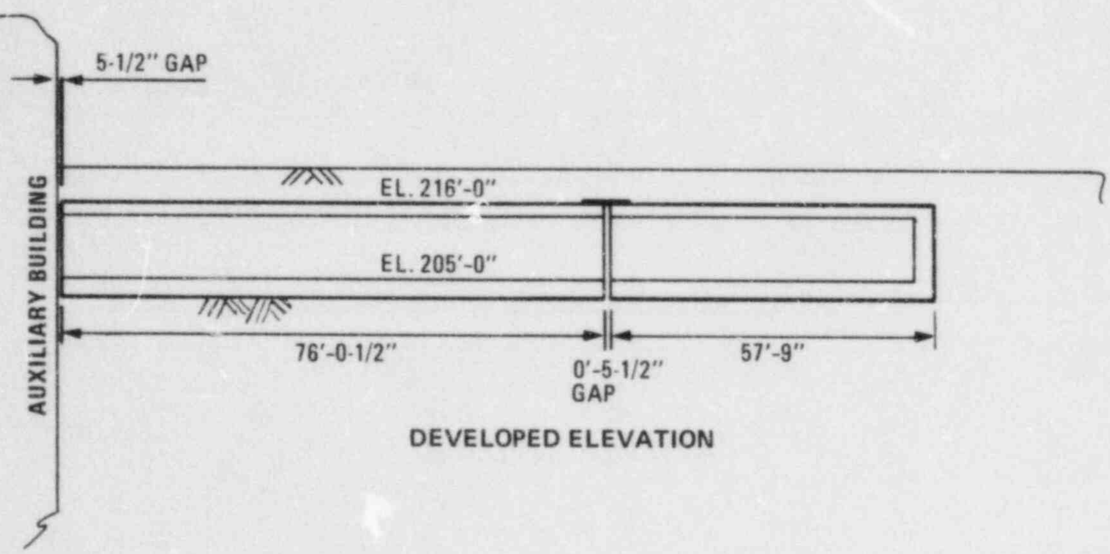


TYPICAL CROSS SECTION

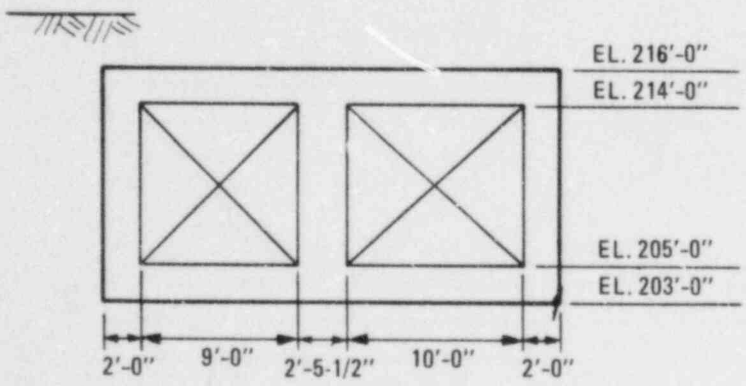
Figure 3
NSCW TUNNEL 1A
DEVELOPED ELEVATION AND CROSS SECTION



DEVELOPED ELEVATION

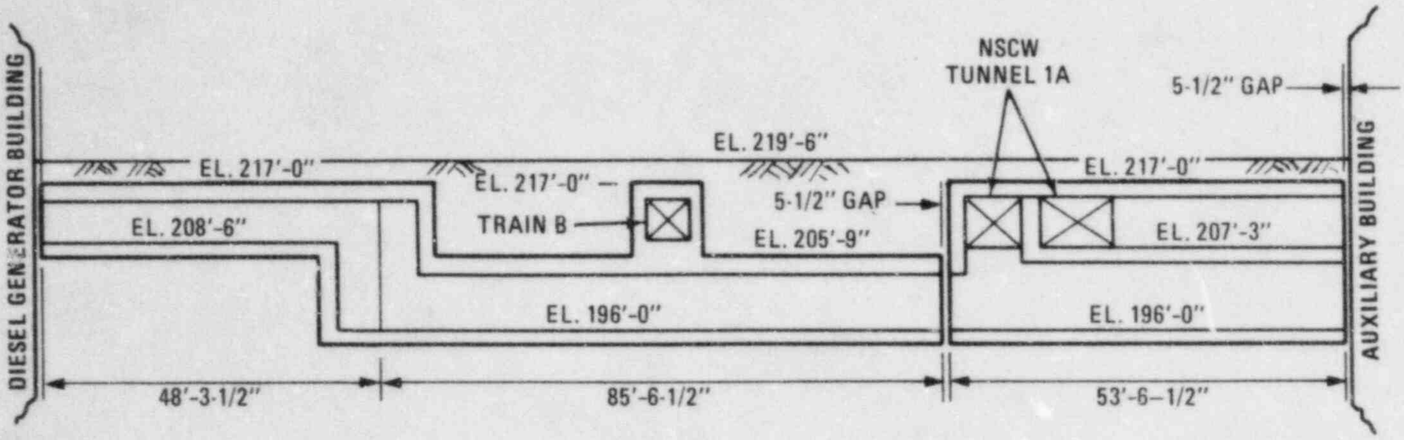


DEVELOPED ELEVATION

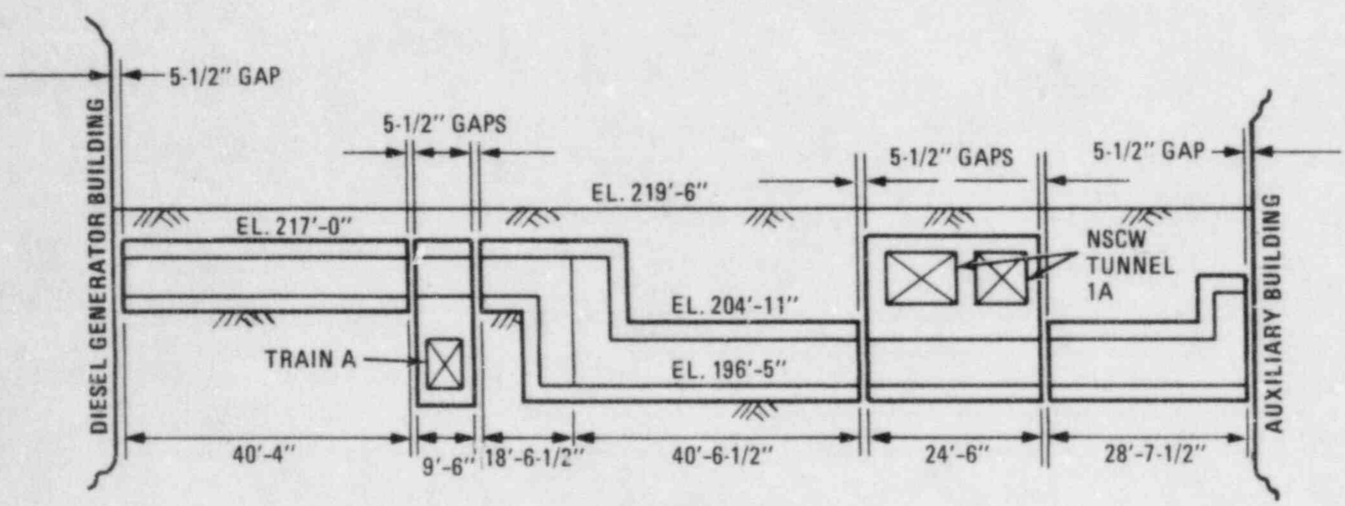


TYPICAL CROSS SECTION

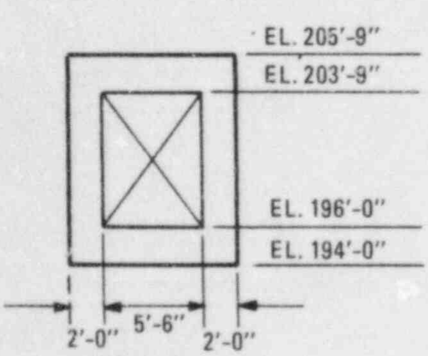
Figure 4
NSCW TUNNEL 1B
DEVELOPED ELEVATIONS AND CROSS SECTION



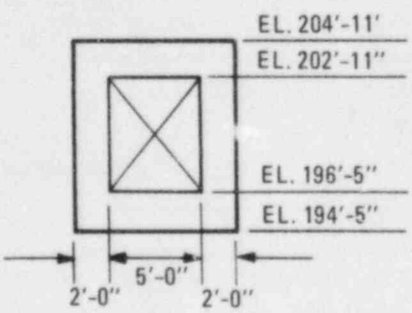
DEVELOPED ELEVATION
(TRAIN A)



DEVELOPED ELEVATION
(TRAIN B)

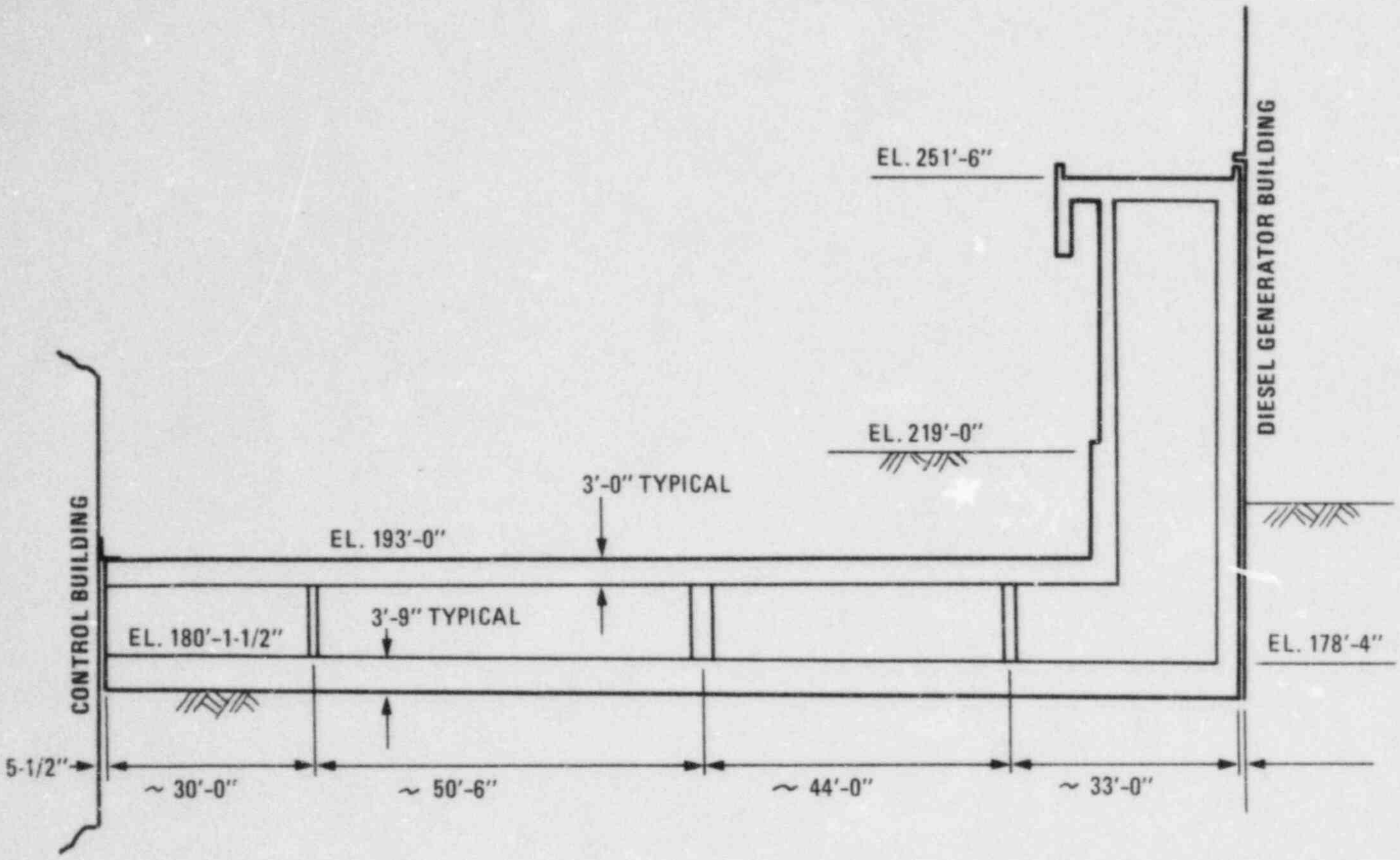


CROSS SECTION
(TRAIN A)

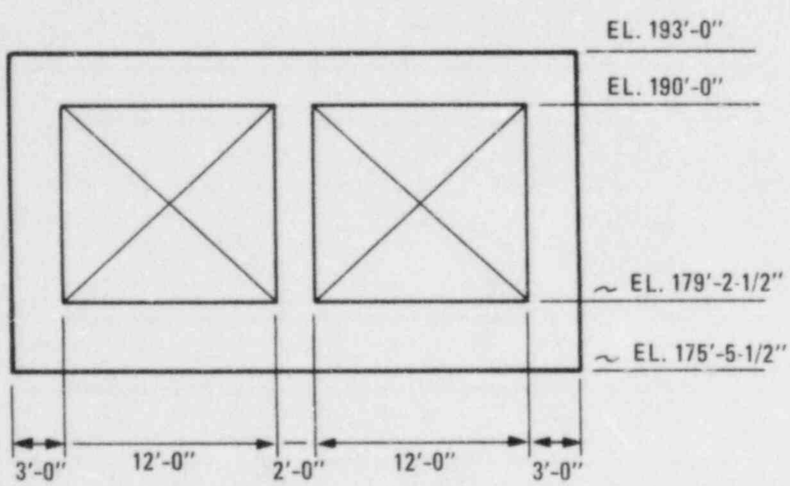


CROSS SECTION
(TRAIN B)

Figure 5
DIESEL GENERATOR PIPING TUNNELS,
DEVELOPED ELEVATIONS AND CROSS SECTIONS

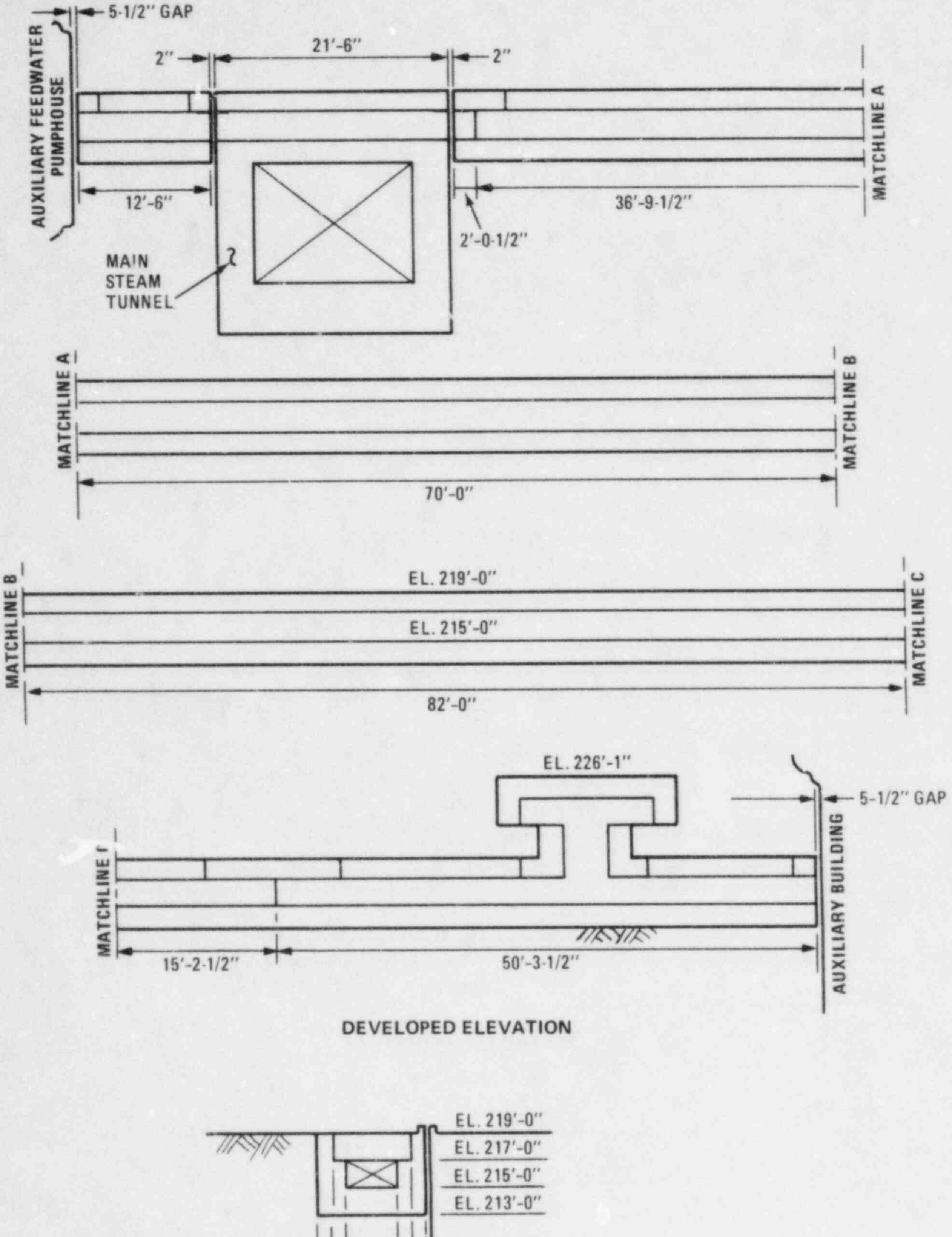


DEVELOPED ELEVATION

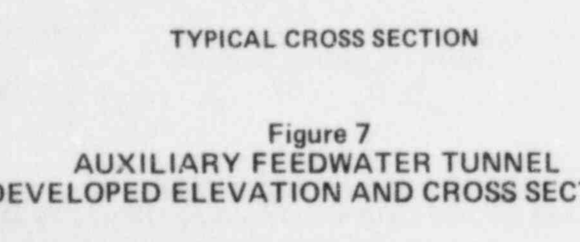


TYPICAL CROSS SECTION

Figure 6
DIESEL GENERATOR ELECTRIC TUNNEL
DEVELOPED ELEVATION
AND CROSS SECTION



DEVELOPED ELEVATION



TYPICAL CROSS SECTION

Figure 7
AUXILIARY FEEDWATER TUNNEL
DEVELOPED ELEVATION AND CROSS SECTION

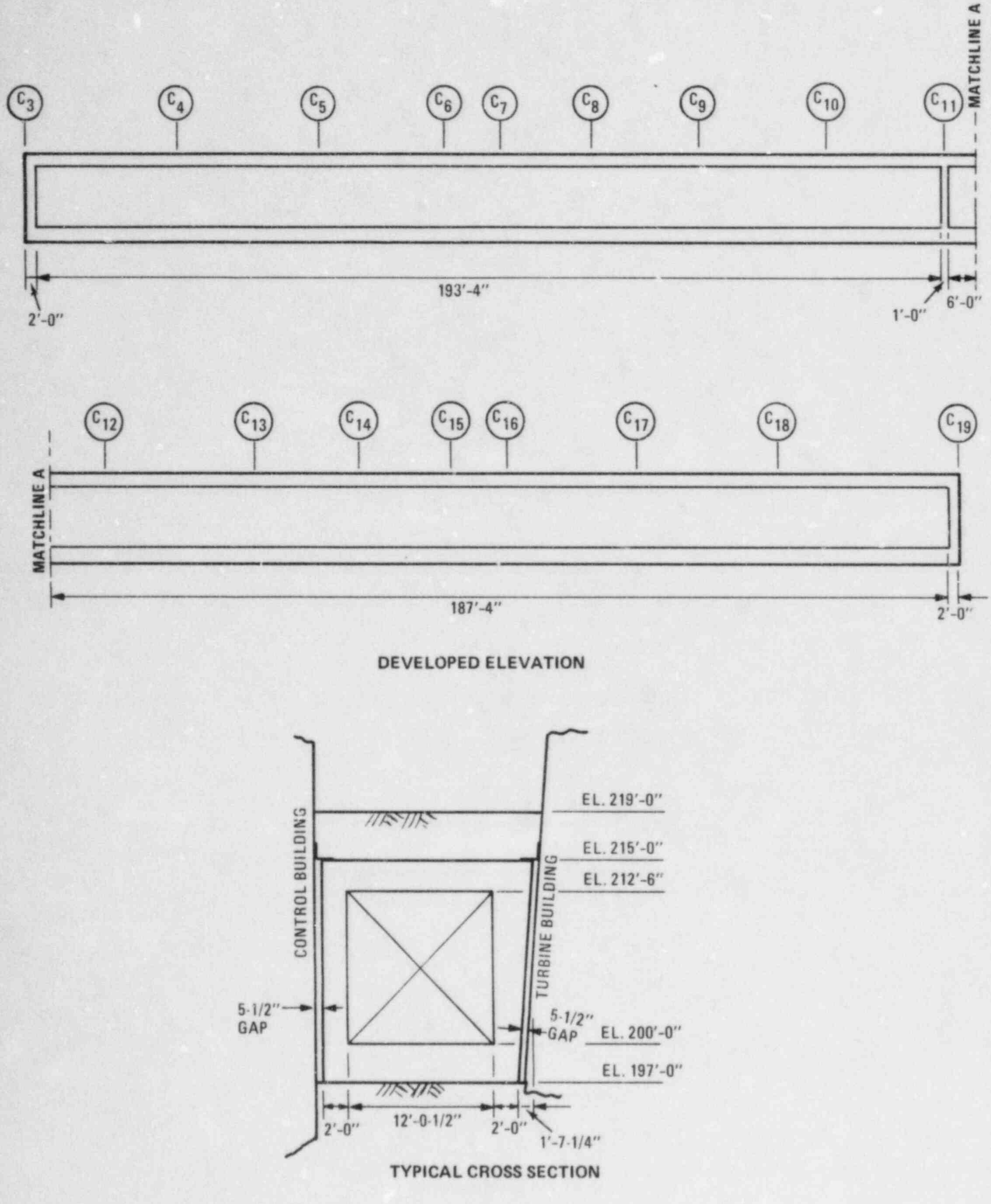
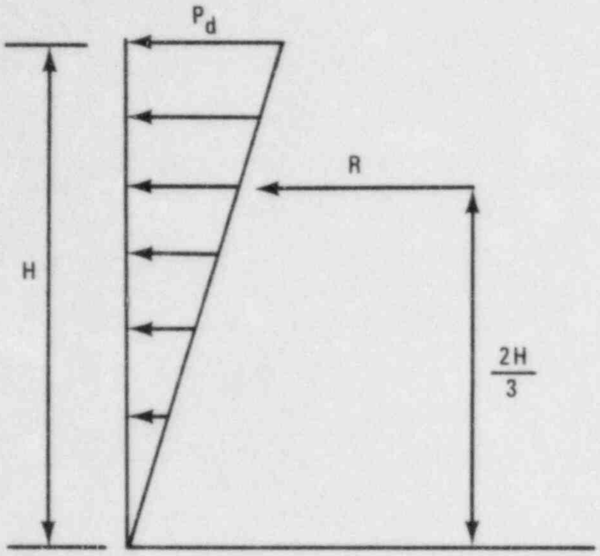


Figure 8
TURBINE ELECTRIC TUNNEL
DEVELOPED ELEVATION AND CROSS SECTION



H: HEIGHT FROM BASE OF STRUCTURE TO SOIL SURFACE

P_d = DYNAMIC INCREMENTAL SOIL PRESSURE

R: RESULTANT FORCE

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

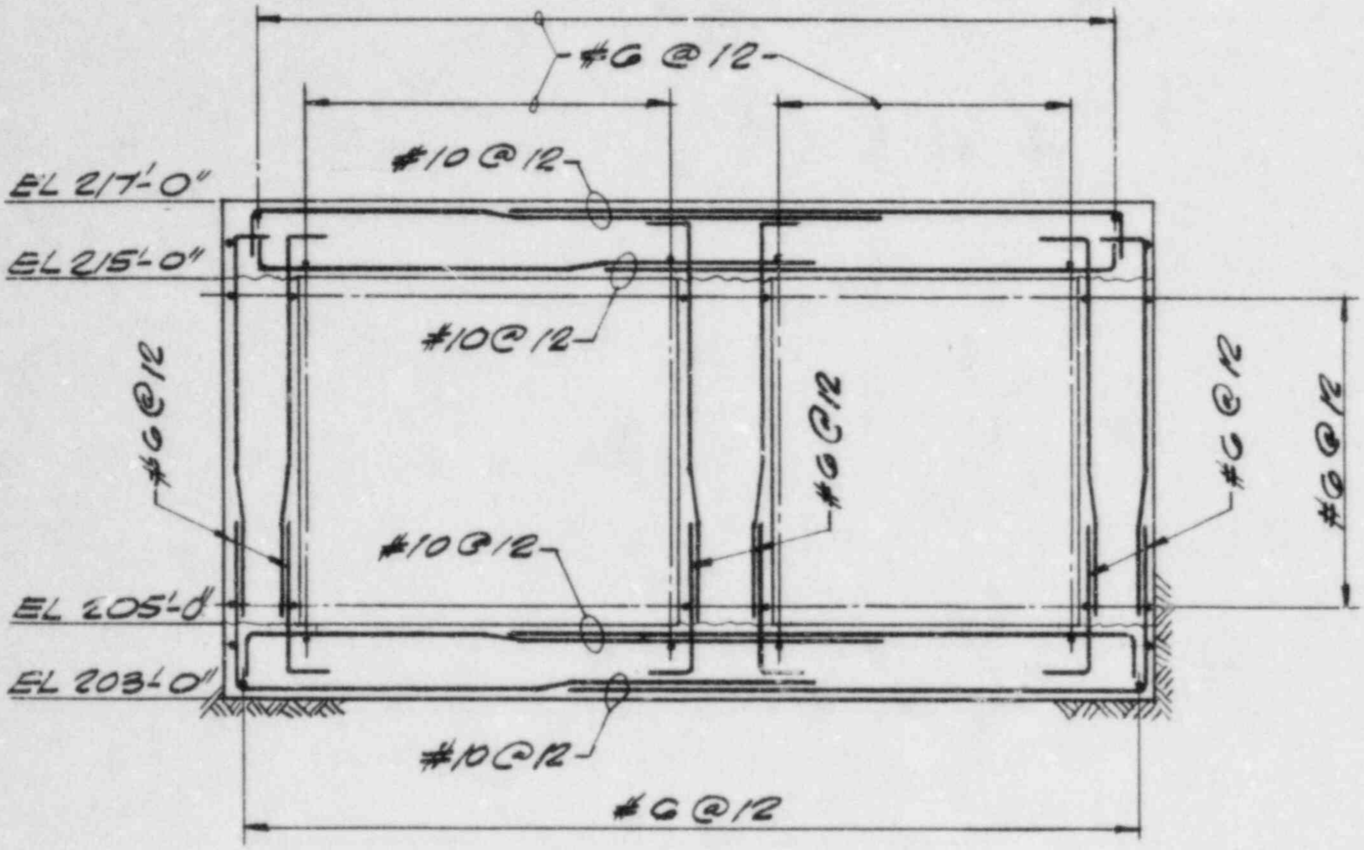
$= .045\ \gamma_m H^2$ (OBE)*

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

γ_m = SOIL MOIST UNIT WEIGHT, PCF

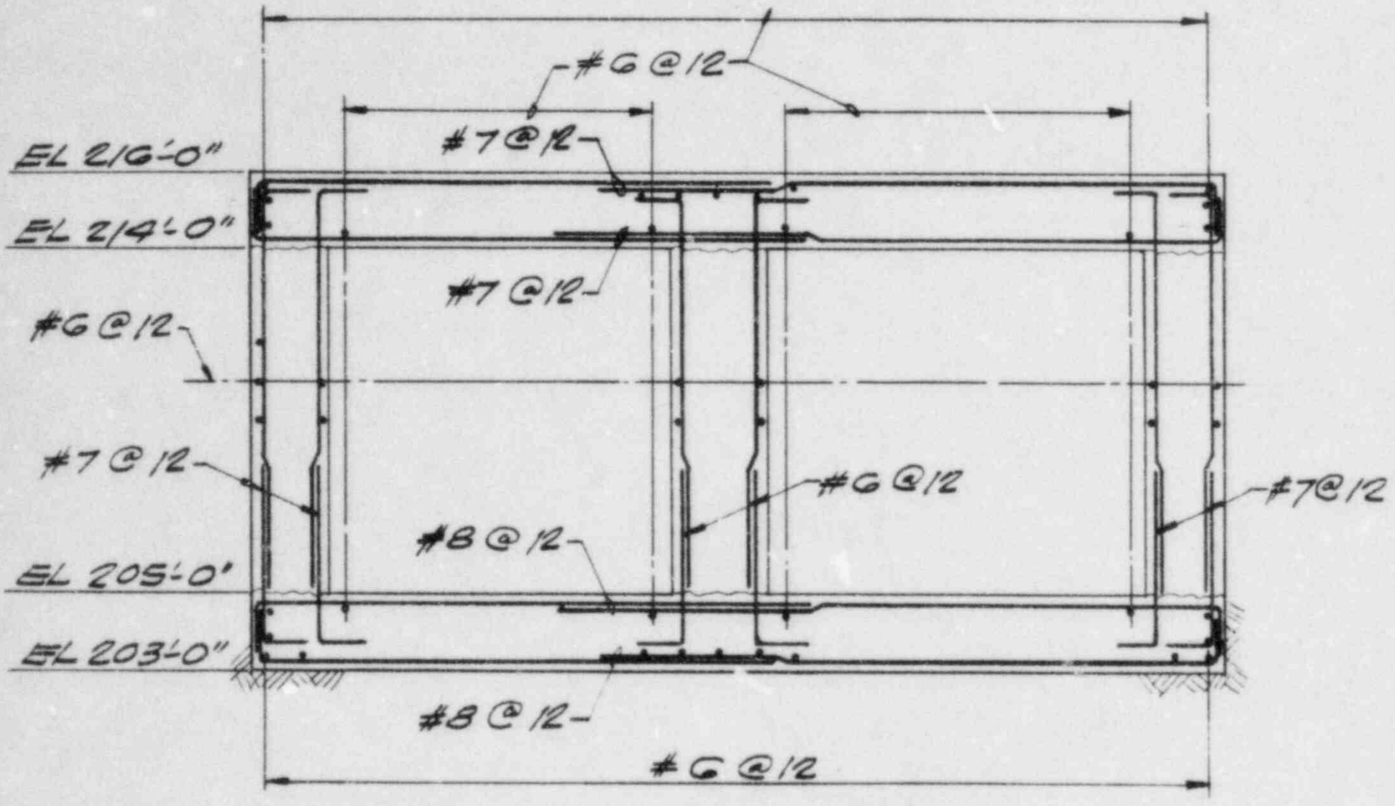
*DERIVED USING THE PEAK GROUND ACCELERATIONS OF 0.12g AND 0.20g FOR OBE AND SSE RESPECTIVELY.

Figure 9
DYNAMIC INCREMENTAL
SOIL PRESSURE PROFILE



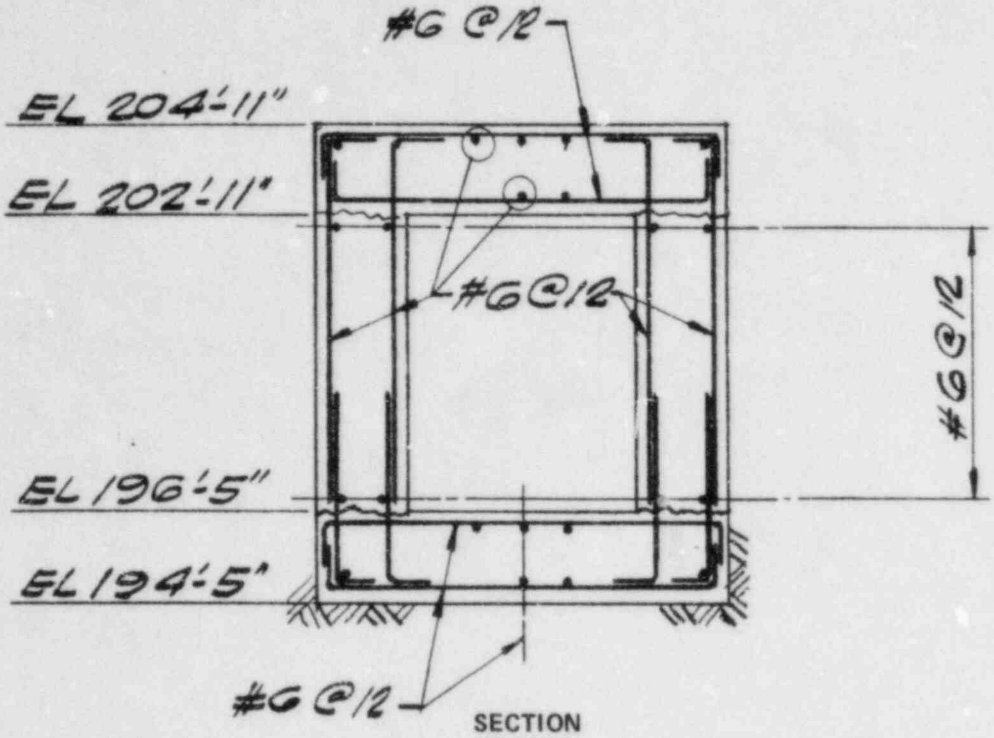
SECTION
TUNNEL 1A

Figure 10
NSCW TUNNEL – TYPICAL REINFORCING DETAILS
(Sheet 1 of 2)

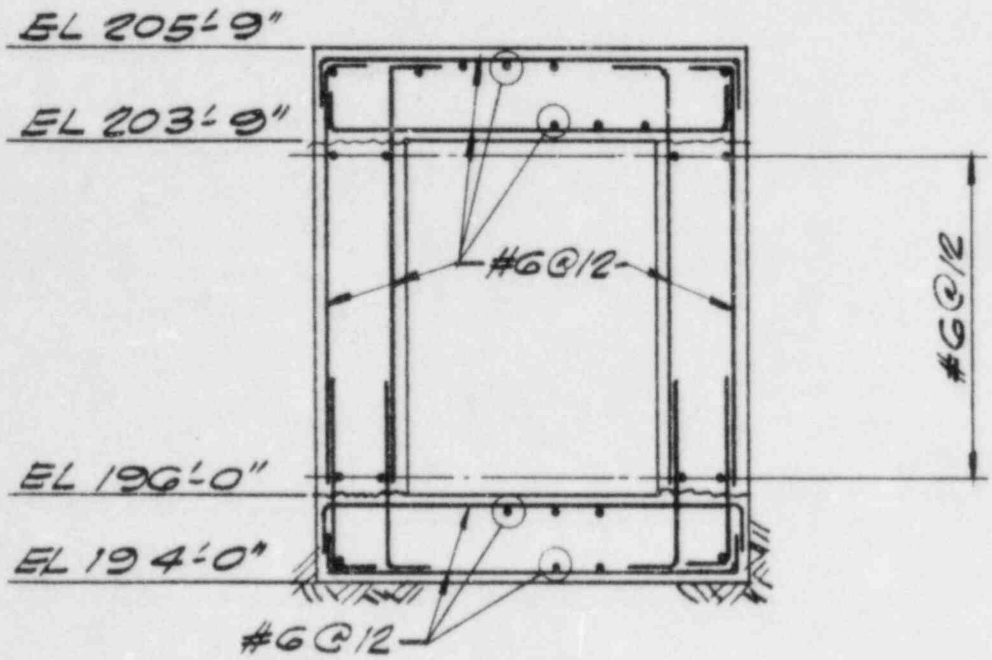


SECTION
TUNNEL 1B

Figure 10
NSCW TUNNEL – TYPICAL REINFORCING DETAILS
(Sheet 2 of 2)

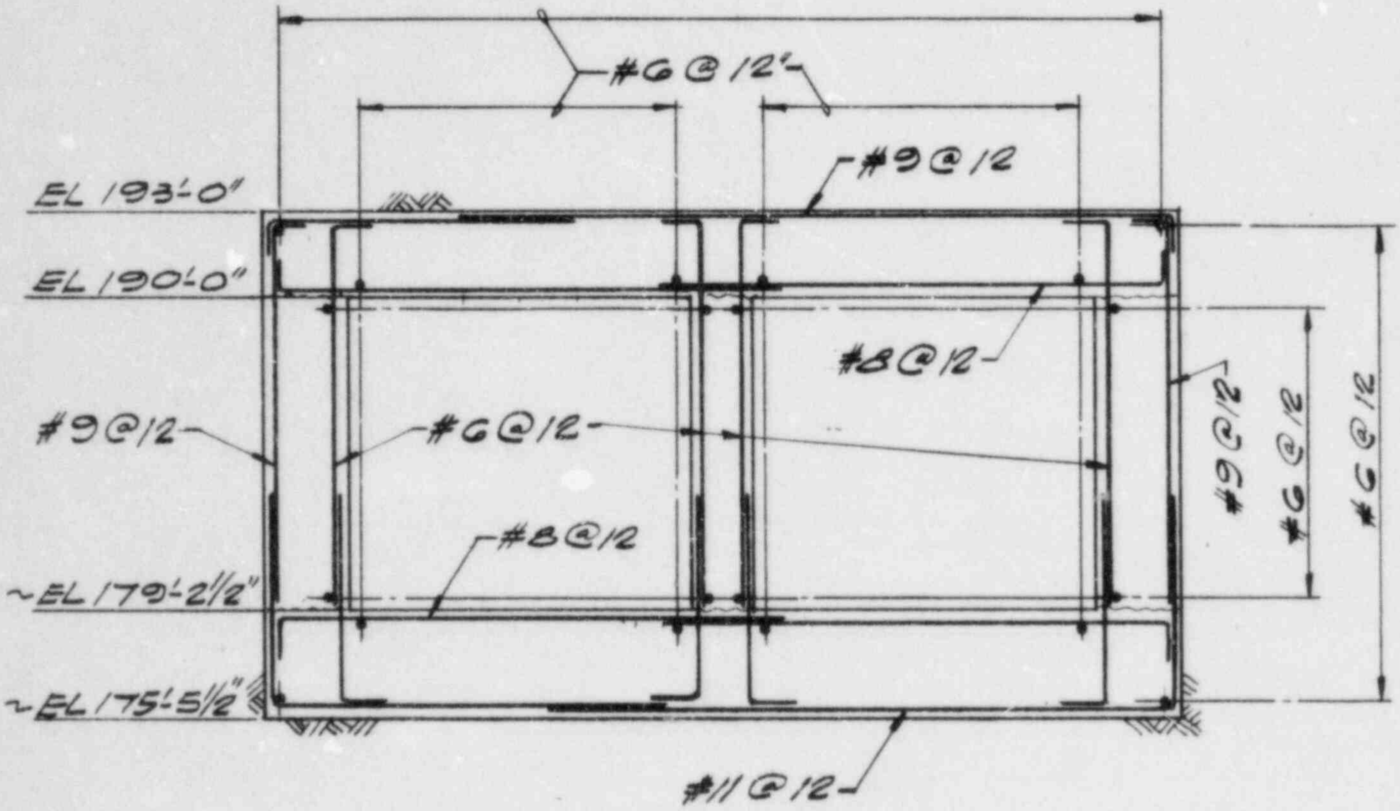


SECTION
TRAIN B



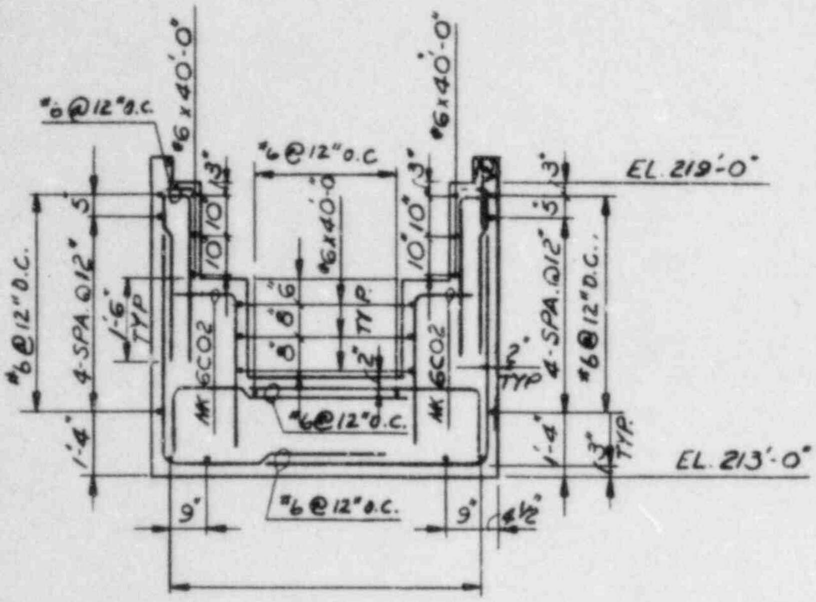
SECTION
TRAIN A

Figure 11
DIESEL GENERATOR PIPING
TUNNELS – TYPICAL REINFORCING DETAILS



SECTION

Figure 12
DIESEL GENERATOR ELECTRICAL
TUNNEL – TYPICAL REINFORCING DETAILS



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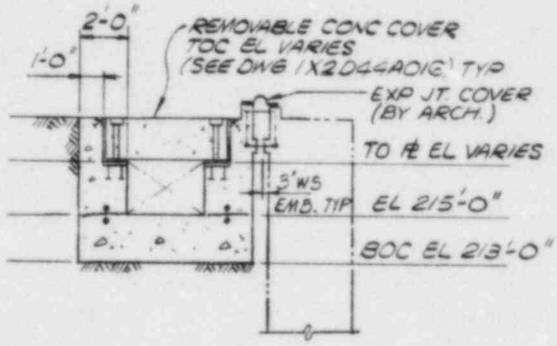

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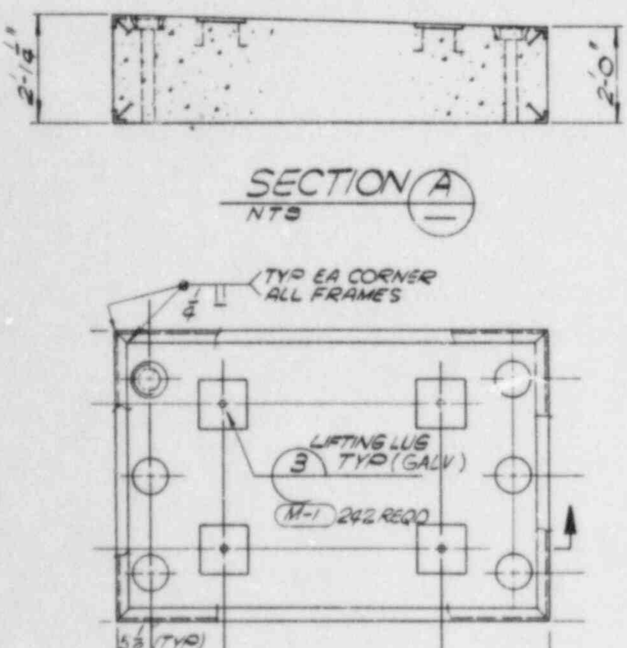

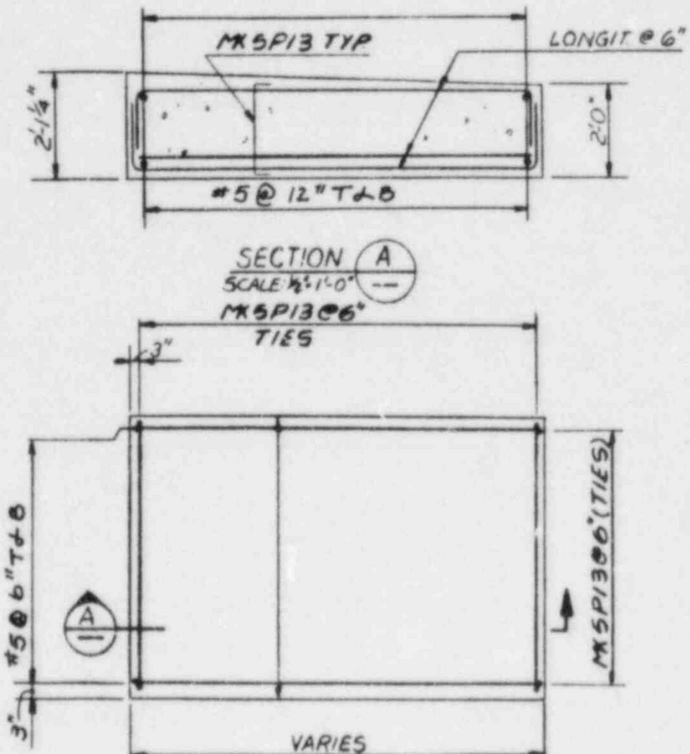

CONC. COVER PLAN

Figure 13
AUXILIARY FEEDWATER TUNNEL –
TYPICAL REINFORCING DETAILS

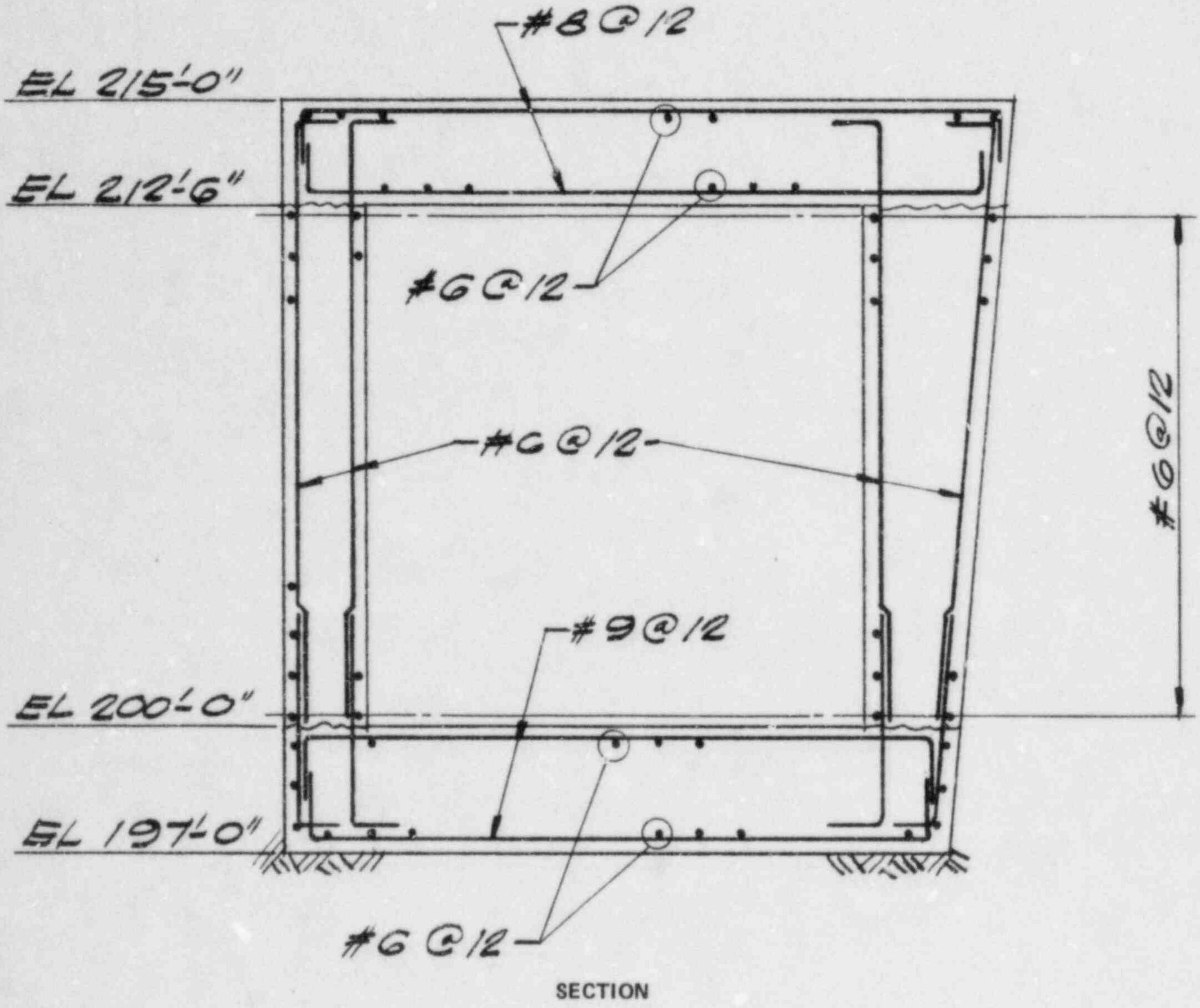

Figure 14
TURBINE ELECTRIC TUNNEL –
TYPICAL REINFORCING DETAILS

APPENDIX A

DEFINITION OF LOADS

APPENDIX A

DEFINITION OF LOADS

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

A.1 NORMAL LOADS

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

- D — Dead loads or their related internal moments and forces, including hydrostatic loads and any permanent loads except prestressing forces.
- L — Live loads or their related internal moments and forces, including any movable equipment loads and other loads which vary with intensity and occurrence, e.g., lateral soil pressures. Live load intensity varies depending upon the load condition and the type of structural element.
- T_o — Thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.
- R_o — Pipe reactions during normal operating or shutdown conditions, based on the most critical transient or steady-state conditions.

A.2 SEVERE ENVIRONMENTAL LOADS

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

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

A.3 EXTREME ENVIRONMENTAL LOADS

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

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

A.4 ABNORMAL LOADS

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

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

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

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

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

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

APPENDIX B

LOAD COMBINATIONS

APPENDIX B

LOAD COMBINATIONS

B.1 STEEL STRUCTURES

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

B.2 CONCRETE STRUCTURES

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

TABLE B.1[a]

STEEL DESIGN LOAD COMBINATIONS
ELASTIC METHOD

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

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

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

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

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

TABLE B.2[a][f]

CONCRETE DESIGN LOAD COMBINATIONS
STRENGTH METHOD

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

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

APPENDIX C

DESIGN OF STRUCTURES FOR TORNADO MISSILE IMPACT

APPENDIX C

DESIGN OF STRUCTURES FOR TORNADO MISSILE IMPACT

C.1 INTRODUCTION

This appendix contains methods and procedures for analysis and design of steel and reinforced concrete structures and structural elements subject to tornado-generated missile impact effects. Postulated missiles, and other concurrent loading conditions are identified in Section 3.? 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-s^2/ft).

V_s = missile striking velocity (ft/s).

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

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

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

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

where:

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

C.3 STRUCTURAL RESPONSE DUE TO MISSILE IMPACT LOADING

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

C.3.1 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.

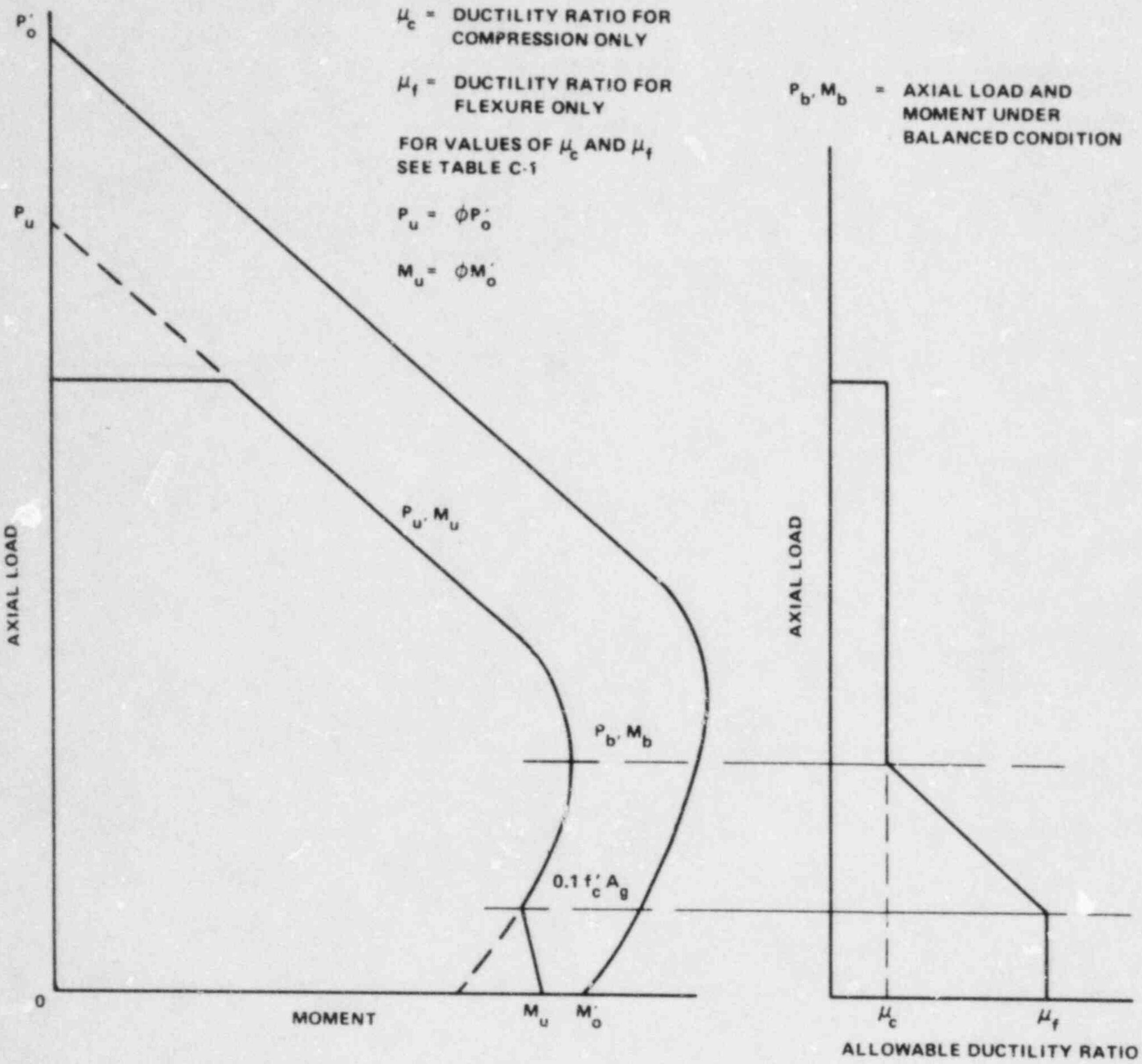
(3) Ductility ratio up to 10 can be used without an angular rotation check.

(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