VOGTLE ELECTRIC GENERATING PLANT

GEORGIA POWER COMPANY

DIESEL GENERATOR BUILDING

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 Plan (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 diesel generator building, in order to assist in planning and conducting a structural audit. Quantitative information is provided regarding the scope of the actual design computations and the final design results.

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

2.0 DESCRIPTION OF STRUCTURE

2.1 GENERAL DESCRIPTION

The diesel generator building is a two-story, reinforced concrete, shear wall, box type structure. The floor and roof slabs act as rigid diaphragms spanning between the walls. There are two diesel generator buildings, one for each unit. The primary function of each building is to house two diesel engines and generators and the corresponding auxiliary support equipment and systems. Each building is divided into two separate and isolated diesel engine trains by a 2-foot-thick reinforced concrete interior barrier wall at the centerline of the building. The exterior walls and the roof have openings for heating, ventilating, and air conditioning (HVAC) air intake and exhaust. There are two large openings in the level 1 south wall for engine installation, removal, and general equipment servicing. These openings are provided with removable concrete doors.

2.2 LOCATION AND FOUNDATION SUPPORT

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

The diesel generator buildings are located adjacent to the containment buildings; one to the east of Unit 1 containment, and the other to the west of the Unit 2 containment. The diesel engines and generators are connected to other systems in the power block through tunnels on the north and south sides of each building. For a more detailed location of the diesel generator buildings, refer to figure 1. Each diesel generator building basemat is founded on Category 1 backfill. The bottom of the diesel generator building basemat is at elevation 211'-0".

2.3 GEOMETRY AND DIMENSIONS

The diesel generator building plan dimensions are approximately 92 feet wide by 114 feet long, and the building is 60 feet high. The 9-foot-thick basemat has several 4-foot-deep trenches in which piping and electrical systems are routed to the diesel engines and auxiliary equipment. In the southeast corner of level 1, there is a small 1-foot-thick concrete wall and slab enclosure that houses the diesel fuel oil day storage tank. Building plan and equipment layout drawings are shown in figure 2.

2.4 KEY STRUCTURAL ELEMENTS

The key structural elements in the diesel generator building include the basemat, shear walls, roof and floor slabs, and deep beams. Listed below is a brief description of the function and design considerations for these elements.

2.4.1 Basemat

The basemat of the diesel generator building is 9 feet thick and has several 4-foot-deep trenches to accommodate piping and electrical systems. In the southwest corner of each engine train, there is a shaft that thickens the mat at that location to join tunnels on the south end of the building, with the bottom elevation being 202'-6". The foundations for the diesel generators and engines consist of short reinforced concrete pedestals that form an integral part of the basemat.

2.4.2 Shear Walls

The shear walls in the diesel generator building extend from elevation 220'-0" to elevation 280'-0". They are 2 feet thick and located as shown in figures 3 and 4.

2.4.3 Roof and Floor Slabs

The roof and floor slabs of the diesel generator building act as rigid diaphragms and are 2 feet thick. The slab located at elevation 255'-0" has many openings used for HVAC ducts and air handling equipment.

2.4.4 Deep Beams

The deep reinforced concrete beams in the diesel generator building are located in level 2 of the building as shown in figure 4. They extend from elevation 255'-0" to 280'-0". The beams located in the south half of the building are 1 foot, 6 inches thick and those located in the north half of the building are 2 feet thick.

2.5 MAJOR EQUIPMENT

Major equipment housed in the diesel generator building include the diesel engines and generators, the corresponding auxiliary skid equipment, and air filtering, exhaust and silencing equipment. A five-ton bridge crane, used to service the diesel engine and generator in each engine train, is hung from the bottom of the level 2 structural steel beams that support the floor slab. Electrical cable tray, HVAC ducts, bus ducts, conduits and piping systems are supported from structural steel framing or from embeded plates on walls and slabs.

2.6 SPECIAL FEATURES

Reinforced concrete barriers are provided, where necessary, for tornado missile protection for the openings in the exterior walls or roof.

3.0 DESIGN BASES

3.1 CRITERIA

The following documents are applicable to the design of the diesel generator building.

3.1.1 Codes and Standards

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

3.1.2 Regulations

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

3.1.3 General Design Criteria (GDC)

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

3.1.4 Industry Standards

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

3.2 LOADS

The diesel generator building is designed for all credible loading conditions. The loads are listed and defined in Appendix A. Abnormal loads, due to a high-energy pipe break accident, are not applicable to the diesel generator building. The magnitudes for the categories of loads applicable to the diesel generator building are provided in the following sections.

3.2.1 Normal Loads

3.2.1.1 Dead Loads (D)

These loads include the weights of structural steel, roof and floor slabs, walls, equipment, platforms, pipes, ducts, cable trays, and supports. The uniform loads used to account for equipment, mechanical, electrical, and piping loads are listed below by elevation and by category.

Category	Elev. 220'	Elev. 255'	Elev. 280'
Piping	75 psf	25 psf	50 psf
Cable trays/ structural steel	75 psf	25 psf	50 psf
HVAC ducts	50 psf	50 psf	75 psf
Equipment	-	50 psf	-

The weights of permanent major equipment located at the appropriate elevations were included as concentrated loads on the basemat or slabs. Refer to figure 2 for the equipment location. The equipment dead loads are listed below.

Item	Weight (lb)
Diesel generator/engine	314,075
Start air compressor and aftercooler package	2,650
Start air receiver	8,600
Diesel fuel oil day tank	15,200
Oily waste sump pump	2,368
Intake air filter	6,950
Exhaust silencer	15,800
Control panel	7,000

3.2.1.2 Live Loads (L)

Live loads considered include occupancy loads, laydown loads, movable equipment loads, and precipitation loads. The uniformly distributed live loads are listed below. The minimum roof live load of 30 psf envelops the effects of occupancy, snow, and 100-year rainwater ponding loads.

Item	Uniform Load (psf)
Roof	30
Platforms	100
Laydown area at grade	1000
Slab at grade, except laydown area	250
Snow load	30

3.2.1.3 Operating Thermal Loads (T_o)

The operating temperature inside the diesel generator building ranges from 50°F to 120°F.

3.2.1.4 Pipe Reactions (R_o)

The piping and equipment reactions during normal operating or shutdown conditions were considered to be negligible for the diesel generator building.

3.2.2 Severe Environmental Loads

3.2.2.1 Operating Basis Earthquake, OBE (E)

Based on the plant site geologic and seismologic investigations, the peak ground acceleration for OBE is established as 0.12g. The free-field response spectra and the development of horizontal and vertical floor accelerations and in-structure response spectra at the basemat and slab elevations are discussed in the Seismic Analysis Report. Table 1 shows the horizontal and vertical floor accelerations.

The OBE damping values, as percentages of critical, applicable to the diesel generator building design are as follows.

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

3.2.2.2 Design Wind (W)

The diesel generator building is designed for a wind velocity of 110 miles per hour, based on a 100-year mean recurrence interval (reference 1). Exposure C, applicable for flat open country, is used. The effective velocity pressure profile for the 110-mph wind is shown in figure 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 floor accelerations and in-structure response spectra at the basemat and slab elevations are discussed in the Seismic Analysis Report. Table 1 shows the horizontal and vertical floor acceleration.

The SSE damping values, as percentages of critical, applicable to the diesel generator building 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 feet
• Atmospheric pressure differential	-3 psi
• Rate of pressure differential change	2 psi/sec

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)
$W_t = W_{tq} + 0.5\ W_{tp}$
$W_t = W_{tq} + W_{tm}$
$W_t = W_{tq} + 0.5\ W_{tp} + W_{tm}$

The tornado effective velocity pressure profile used in the design (shown below and in figure 5) is in accordance with reference 2. The effective velocity pressure includes the size coefficient and is used in conjunction with the external pressure coefficient to determine the net positive and negative pressures. No reduction in pressure is made for the shielding effects that may be provided by adjacent structures.

Structural Element	Tornado Pressure (psf)
Level 1 wall E&W	232
Level 1 wall N&S	249
Level 2 wall E&W	232
Level 2 wall N&S	249
Roof	305

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

The tornado missiles that were considered in the design of the diesel generator building are defined in table 2.

3.2.3.3 Probable Maximum Precipitation, PMP (N)

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

3.2.3.4 Blast Load (B)

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

3.2.4 Abnormal Loads

There are no postulated high-energy pipe break accidents within the diesel generator building. Therefore, the abnormal loads do not apply.

3.3 LOAD COMBINATIONS AND STRESS/STRENGTH LIMITS

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

3.4 MATERIALS

The following materials and material properties are used in the design of the diesel generator building.

3.4.1 Concrete

- Compressive strength $f'_c = 4$ ksi
- Modulus of elasticity $E_c = 3834$ ksi
- Shear modulus $G = 1625$ 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 Structural Tubing - ASTM A500 Grade B

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

3.4.5 Foundation Media

3.4.5.1 General Description

See section 2.2

3.4.5.2 Category 1 Backfill

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

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

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

3.4.5.3 Modulus of Subgrade Reaction

- Static 60 kcf
- Dynamic 85 kcf

3.4.5.4 Net Bearing Capacities

- Ultimate 60.9 ksf
- Allowable static 20.3 ksf
- Allowable dynamic 30.5 ksf

4.0 STRUCTURAL ANALYSIS AND DESIGN

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

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

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

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

For computer analyses, the modeling techniques, boundary conditions, application of loads, and description of the computer model are provided to illustrate the overall method of analysis. In addition, for both manual and computer analyses and design, representative analysis and design results are provided to illustrate the response of the key structural elements for governing load combinations.

4.1 SELECTION OF GOVERNING LOAD COMBINATION

An evaluation of load magnitudes, load factors, and load combinations is performed to determine the load combinations that govern the overall response of the structure. It is determined that load combination equation 2 for steel design (Appendix B, Table B.1) and equation 3 for concrete design (Appendix B, Table B.2) containing OBE govern over all other load combinations, and hence form the basis for the overall structural analysis and design of the diesel generator building. All other load combinations, including the effects of tornado loads, are evaluated where applicable on a local area basis, (i.e., section 5.2). The localized response is combined with the analysis results of the overall structural response, as applicable, to confirm that design integrity is maintained.

4.2 VERTICAL LOAD ANALYSIS

The vertical load carrying elements of the diesel generator building consist of concrete slabs and steel beams that support the applied vertical loads, walls and deep beams that support the slabs, and the basemat which transmits the loads from the walls and columns to the foundation medium. Representative vertical load carrying elements are identified in figures 3 and 4.

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

4.3 LATERAL LOAD ANALYSIS

The lateral load carrying elements of the diesel generator building consist of concrete slabs acting as rigid diaphragms to resist applied lateral loads, the shear walls which transmit the loads from the slab diaphragm through the shear walls below to the basemat, and the basemat which transmits the loads from the walls and columns to the foundation medium. Representative lateral load carrying elements are identified in figures 3 and 4.

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

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

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

4.4 COMBINED EFFECTS OF THREE COMPONENT EARTHQUAKE LOADS

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

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

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

4.5 ROOF AND FLOOR SLABS

4.5.1 Analysis and Design Methodology

A representative slab panel framing plan (elevation 255'-0") of the diesel generator building is presented in figure 6, showing the structural elements provided for vertical and lateral support of the slab panels, which consist of deep beams and load bearing shear walls. Based on the panel configuration, the relative stiffness of the supporting members and type of fixity provided, slab panels are analyzed for one-way or two-way action using appropriate boundary conditions and standard beam and plate formulae.

Slab areas that are designed to act in composite action with the supporting steel beams via shear studs are analyzed for flange action parallel to the span direction of the beam, using conventional analysis techniques to determine the location of the composite neutral axis, the effective flange width, and the corresponding design forces in the flange. Refer to figure 7 for details of the composite design.

Equivalent uniformly distributed loads are applied to slab panels. The design vertical earthquake loads for slab panels in a level are obtained by multiplying the tributary mass from the applied loading (including its own mass) by the maximum floor acceleration at that level. The effects of the underhung bridge crane are considered by applying concentrated design loads and moments to the composite beams at the locations of the crane runway beams

Based on the floor flexibility study, it is concluded that the effects of vertical flexibility on the diesel generator building floor accelerations and response spectra are insignificant, as long as the fundamental floor (slab-beam) system frequency is equal to or higher than 20 cps. The evaluation of the floor systems in the diesel generator building demonstrates that their frequencies are higher than this value. The frequency calculations account for the stiffening effects of the structural steel columns that are selectively located to stiffen slab panels with very long spans. A representative detail of this column is provided in figure 7. The details of the floor flexibility study are provided in the Seismic Analysis Report.

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

The structural design is based on strength considerations and consists of sizing and detailing the reinforcing steel to meet the ACI 318 Code requirements. In general, the reinforcing requirements are determined for the governing face of the slab and conservatively provided on both faces. For the slab areas designed for composite action, the composite design is done in accordance with the requirements of the AISC Code. Shear studs are welded to the top flange of the supporting steel beams.

As appropriate, additional reinforcement is provided in the slab adjacent to large floor openings. Reinforcing steel at the periphery of the slab is investigated to ensure that reinforcement requirements are satisfied to resist flexure due to diaphragm action.

4.5.2 Design Results

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

4.6 DEEP BEAMS

4.6.1 Analysis and Design Methodology

A representative plan view showing the deep beams (elevation 280'-0") is presented in figure 4.

For the vertical loads analysis, the boundary conditions for the deep beams at the intersecting walls are assumed to be simply supported. For intersecting deep beams, where one deep beam is partially supported by another, compatibility of deflection is considered. Maximum moments and shears are determined using standard formulas for deep beams, satisfying the appropriate compatibility requirements. The deep beams function to support both the roof slab above and the slab below along their length.

Uniformly distributed floor loads are converted to an equivalent uniform linear load using the tributary load method. The design vertical earthquake load for the deep beams is obtained by multiplying the tributary mass from the applied loading (including the deep beam's own mass) by the maximum floor acceleration at the appropriate level.

For the horizontal loads analysis, the deep beams are analyzed for shear loads due to the differential displacement between the diaphragm at the bottom of the deep beams and the roof diaphragm at the top of the deep beams.

The structural design of deep beams is based on strength considerations and consists of sizing and detailing the reinforcing steel to meet the ACI 318 Code requirements. The design consists of both vertical and horizontal reinforcing bars. The horizontal reinforcement in the walls is designed to resist the maximum bending moments due to vertical design loads.

In general, the flexural steel reinforcing requirements are determined for the governing face and conservatively provided on both faces. This reinforcement is located in bands at the top and bottom edges of the deep beam.

Special consideration is given to large openings by evaluating the strips adjacent to the openings. The shear at the location of each opening is resisted by the top and bottom portions of the remaining deep beam. The principle bending moments are combined with the secondary bending moments due to local cantilever action of the remaining portions of the deep beam above and below the opening. The resulting local design moments and shears are used to determine reinforcing steel around the openings.

4.6.2 Design Results

The design results of representative deep beams for governing load combinations are summarized in table 4.

4.7 SHEAR WALLS

4.7.1 Analysis and Design Methodology

The location of shear walls is identified in figures 3 and 4 for representative levels.

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

The out-of-plane design loads are considered using the inertia loads on the walls due to the structural acceleration caused by the design earthquake. The seismic inertia loads are applied as uniform pressure loads.

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

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

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

B. The flexural capacity of the shear wall using the reinforcement determined is obtained using the Cardenas equation, (reference 3).

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

   1. The total vertical reinforcement required for the design moment is computed using the Cardenas equation (reference 3) and is distributed uniformly along the length of the wall.

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

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

4.7.2 Design Results

The design results of representative shear walls for governing load combinations are summarized in table 5.

4.8 BASEMAT

4.8.1 Analysis and Design Methodology

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

Two separate two-dimensional finite element models are prepared at key transverse sections through the basemat (figure 8). The finite element models are prepared using conventional modeling techniques. Each is modeled with beam elements. The three nodes in the model that correspond to the locations of the three north-south walls are restrained against translation. The first model is a representative section through the basemat near the center of the building, and includes the trenches. The second model is a representative section through the basemat near the end of the building where the thickness is uniform.

The basemat is determined to be rigid on the basis of relative stiffness between basemat and soil, and is therefore analyzed using a linear soil pressure distribution. The basemat is analyzed as a continuous two-span beam, simply supported at the three north-south walls. The foundation pressure profiles corresponding to the governing load combinations are determined manually. A finite element analysis is then performed to determine the moments and shears in the basemat (refer to figure 9 for representative analysis results).

All reinforcement in the basemat is designed and detailed in accordance with the ACI 318 Code.

For representative reinforcement details of the basemat, refer to figure 10.

The design of the basemat also considers the effects of the vibration of the diesel engines. The basemat is designed to avoid resonance by limiting the resonant frequency of the tributary mat foundation to no more than half of the operating frequency, as recommended in reference 4. The maximum operating amplitude of the foundation is determined and is within the acceptable range.

4.8.2 Design Results

The design results of representative basemat elements are presented in table 6.

5.0 MISCELLANEOUS ANALYSIS AND DESIGN

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

5.1 STABILITY ANALYSIS

The overall stability of the diesel generator building is evaluated by determining the factor of safety against overturning and sliding. Since the foundation level (elevation 211'-0") is above the high water table elevation (elevation 165'-0"), the diesel generator building is not subjected to flotation effects.

5.1.1 Overturning

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

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

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

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

5.1.2 Sliding

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

5.1.3 Analysis Results

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

5.2 TORNADO LOAD EFFECTS

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

Roof and exterior wall panels are evaluated for tornado load effects. The localized response of these panels 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 analysis are provided in table 8.

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

5.3 FOUNDATION BEARING PRESSURE

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

6.0 CONCLUSION

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

7.0 REFERENCES

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

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

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

4. NAVFAC DM-7, March 1971, Design Manual, Soil Mechanics, Foundations, and Earth Structures.

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

TABLE 1

DIESEL GENERATOR BUILDING SEISMIC ACCELERATION VALUES

Floor Accelerations (g's)[1]				
Elevation	OBE		SSE	
	Horizontal	Vertical	Horizontal	Vertical
220' (grade level)	C.19	0.19	0.31	0.31
255'	0.25	0.20	0.40	0.32
280'	0.25	0.20	0.40	0.32

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

TABLE 2

TORNADO MISSILE DATA

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

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

TABLE 3

DESIGN RESULTS FOR REPRESENTATIVE SLAB ELEMENTS

Element	Governing(1) Load Combination Equation	Design Forces Moments (k-ft/ft)	A_s Required (in.^{2}/ft)				A_s Provided (in.^{2}/ft)			
			N-S		E-W		N-S		E-W	
			Top	Bottom	Top	Bottom	Top	Bottom	Top	Bottom
Level 2 Midspan	3	N-S M_u = ±38.8	0.44	0.44	0.31 (3)	0.31 (3)	1.0	1.0	1.0	1.0
Level 2 End Span	3	E-W M_u = -94.0	0.31 (3)	0.31 (3)	0.98	0.31 (3)	1.0	1.0	1.0	1.0
Roof Composite	3	N-S M_u = ±21.7	0.31 (3)	0.31 (3)	0.31 (3)	0.31 (3)	1.0	1.0	1.0	1.0
Roof North End	8	N-S M_u = ±42.0 N-S M_u = ±42.0 (2)	0.45 (2)	0.45 (2)	0.45 (2)	0.45 (2)	1.0	1.0	1.0	1.0
Roof South End	3	E-W M_u = ±116.5	0.31 (3)	0.31 (3)	1.29	1.29	1.0	1.0	1.33	1.33

(1) Load combination equations correspond to equations in Appendix B.
(2) Based on load combination equation 3.
(3) Governed by minimum code reinforcement requirements.

TABLE 4

DESIGN RESULTS FOR REPRESENTATIVE DEEP BEAMS

(1) Element	(2) Governing Load Combination Equation	Design Forces Moment (k-ft)	A_s Required				A_s Provided			
			(3) Vertical (in.^{2}/ft)	Horizontal (in.^{2})			(3) Vertical (in.^{2}/ft)	Horizontal (in.^{2})		
				Top Band	Center Portion	Bottom Band		Top Band	Center Portion	Bottom Band
Wall C	3	$M_u = \pm$ 21,385	0.36 (5)	9.60	14.4	20.84	1.0	12.7	35.6	22.9
Wall B	3	$M_u = \pm$ 4,217	0.36 (5)	0.2	6.53	5.7	0.79	6.3	25.3	6.3
Wall A	8	$M_u = \pm$ 2,368 (4)	0.36 (4)(5)	5.21 (4)	0.88 (4)	5.61 (4)	1.0	6.0	14.0	14.0
Wall E	3	$M_u = \pm$ 21,740	0.27 (5)	12.62	12.88	22.37	1.0	15.6	43.6	25.0
Wall D	3	$M_u = \pm$ 9,976	0.27 (5)	5.0	7.4	10.74	0.60	6.3	23.8	12.6

(1) Location given in figure 4.
(2) Load combination equations correspond to equations in Appendix B.
(3) Reinforcement required at each face.
(4) Based on load combination equation 3.
(5) Governed by minimum code reinforcement requirements.

TABLE 5

DESIGN RESULTS FOR REPRESENTATIVE SHEAR WALLS

Element	Governing Load Combination Equation (1)	Design (5) In-Plane Forces		A_s Required (in.^{2}/ft)		A_s Provided (in.^{2}/ft)	
		V_u (k)	M_u (k-ft)	Horiz	Vert	Horiz	Vert
Level 1 South Wall	8	4,105 (3)	196,306 (3)	0.72 (4) (3)	2.70 (3)	1.27	3.12
Level 1 East Wall	8	3,190 (3)	143,716 (3)	0.72 (4) (3)	1.10 (3)	1.0	1.56
Level 2 (2) Barrier Wall	3	1,469	35,256	0.72 (4)	0.79	1.0	1.0
Level 2 South Wall	3	2,364	56,736	0.72 (4)	0.79	1.0	1.0

(1) Load combination equations correspond to equations in Appendix B.
(2) Interior barrier wall that isolates the two diesel engine trains.
(3) Based on load combination equation 3.
(4) Governed by minimum code reinforcement requirements.
(5) Axial load N_u in compression is conservatively assumed to be zero for determining the design shear capacity of the walls.

TABLE 6

DESIGN RESULTS FOR REPRESENTATIVE BASEMAT ELEMENTS

Element	Governing Load Combination Equation (1)	Moment Capacity (2) (k-ft/ft)		Design Forces (2) Moment (k-ft/ft)		A_s Provided ($in.^2/ft$)
		Positive	Negative	Positive	Negative	
Trench Region Thickened Section	3	2,398	2,954	1,838	2,011	Top layer = 3.12 Middle layer = 4.5 Bottom layer = 4.5
Trench Region Thinner Section	3	1,035	1,018	1,031	1,010	Top layer = 4.5 Bottom layer = 4.5
Non-Trench Region	3	2,112	1,996	1,897	1,900	Top Layer = 4.8 Bottom layer = 4.5

(1) Load combination equations correspond to equations in Appendix B.
(2) Positive moment indicates tension on top face of basemat.

TABLE 7

FACTORS OF SAFETY FOR STRUCTURAL STABILITY

Load(1)(3) Combination	Overturning Factor of Safety			Sliding Factor of Safety	
	Minimum Required	Calculated: Equivalent Static	Calculated: Energy Balance	Minimum Required	Calculated
D + H + E	1.5	2.39	See Note (2)	1.5	1.89
D + H + E'	1.1	1.49	916	1.1	1.1

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

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

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

TABLE 8

TORNADO MISSILE ANALYSIS RESULTS[1]

Panel Description and Location	Panel Size			Computed Ductility	Allowable Ductility
	Length (ft)	Width (ft)	Thickness (ft)		
South Wall Lvl 2 D_4-D_5	45.0	24.0	2.0	5.2	10.0
East Wall Lvl 1 D_A-D_D	112.0	34.0	2.0	8.9	10.0
Interior Barrier Wall Lvl 2 13' South of D_A	31.7	12.6	2.0	1.7	10.0
Roof Slab D_5-D_6 and D_B-D_C	45.0	7.0	2.0	0.5	10.0
Roof Slab $D_{4.6}$-D_4 and D_C-D_D	40.75	24.0	2.0	2.2	10.0

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

IMAGE EVALUATION
TEST TARGET (MT-3)

150mm

6″

IMAGE EVALUATION TEST TARGET (MT-3)

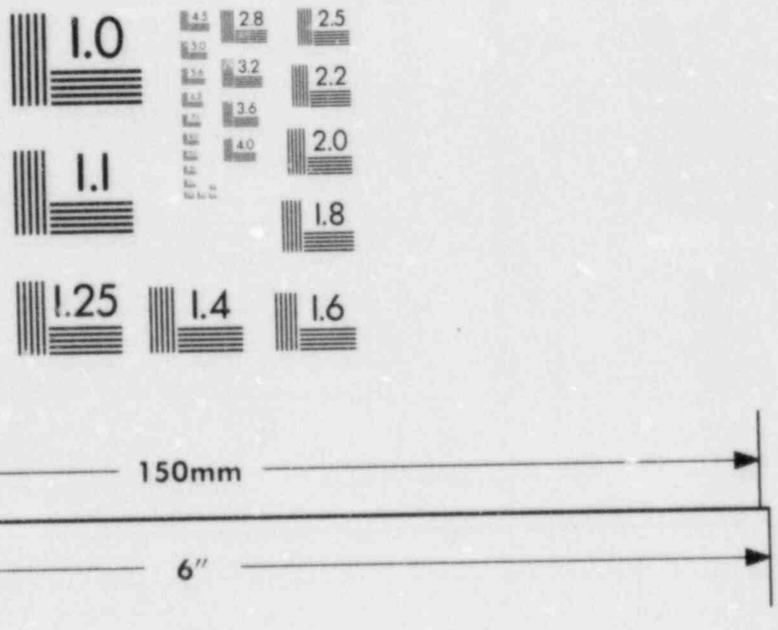


TABLE 9

MAXIMUM FOUNDATION BEARING PRESSURES[1]

Gross Static (ksf)	Net Static (ksf)	Gross Dynamic (ksf)	Net Dynamic (ksf)	Allowable Net[2] Bearing Capacity		Computed Factor[3] of Safety	
				Static (ksf)	Dynamic (ksf)	Static	Dynamic
3.8	2.7	13.6	12.5	20.3	30.5	22.6	4.9

(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.

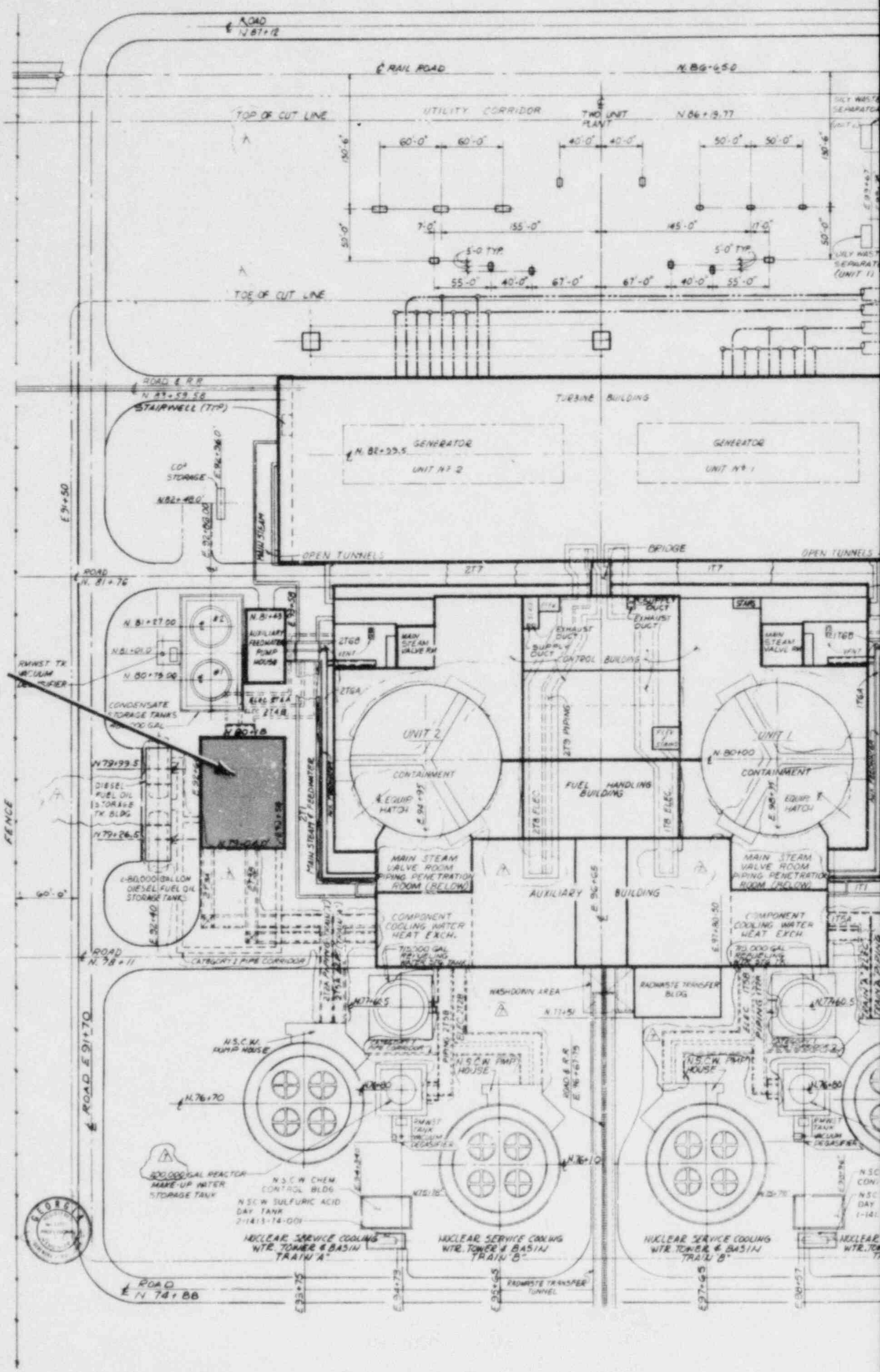
Gross Dynamic = Maximum soil pressure under dynamic loading conditions (i.e., unfactored SSE).

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.

DIESEL GENERATOR BUILDING



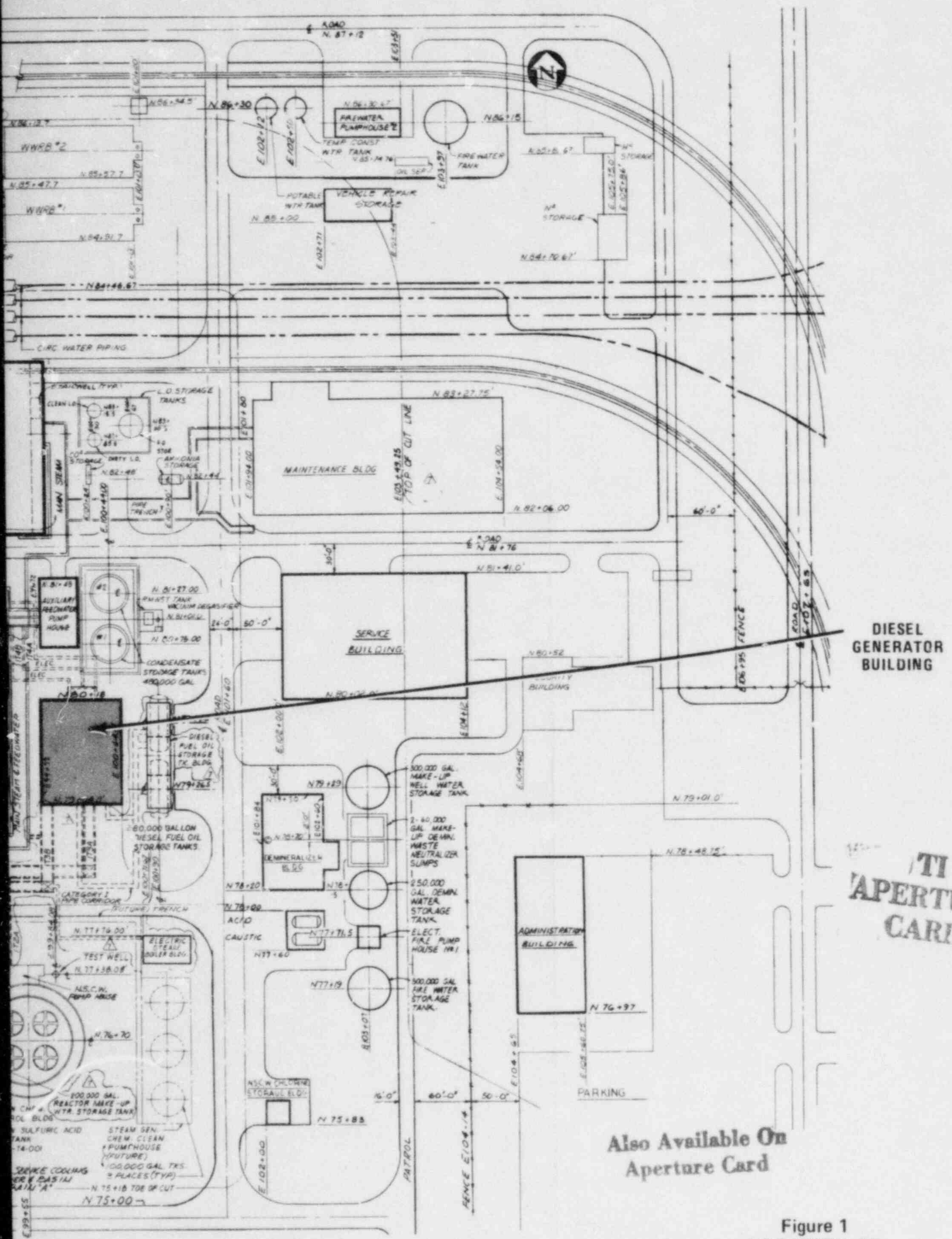
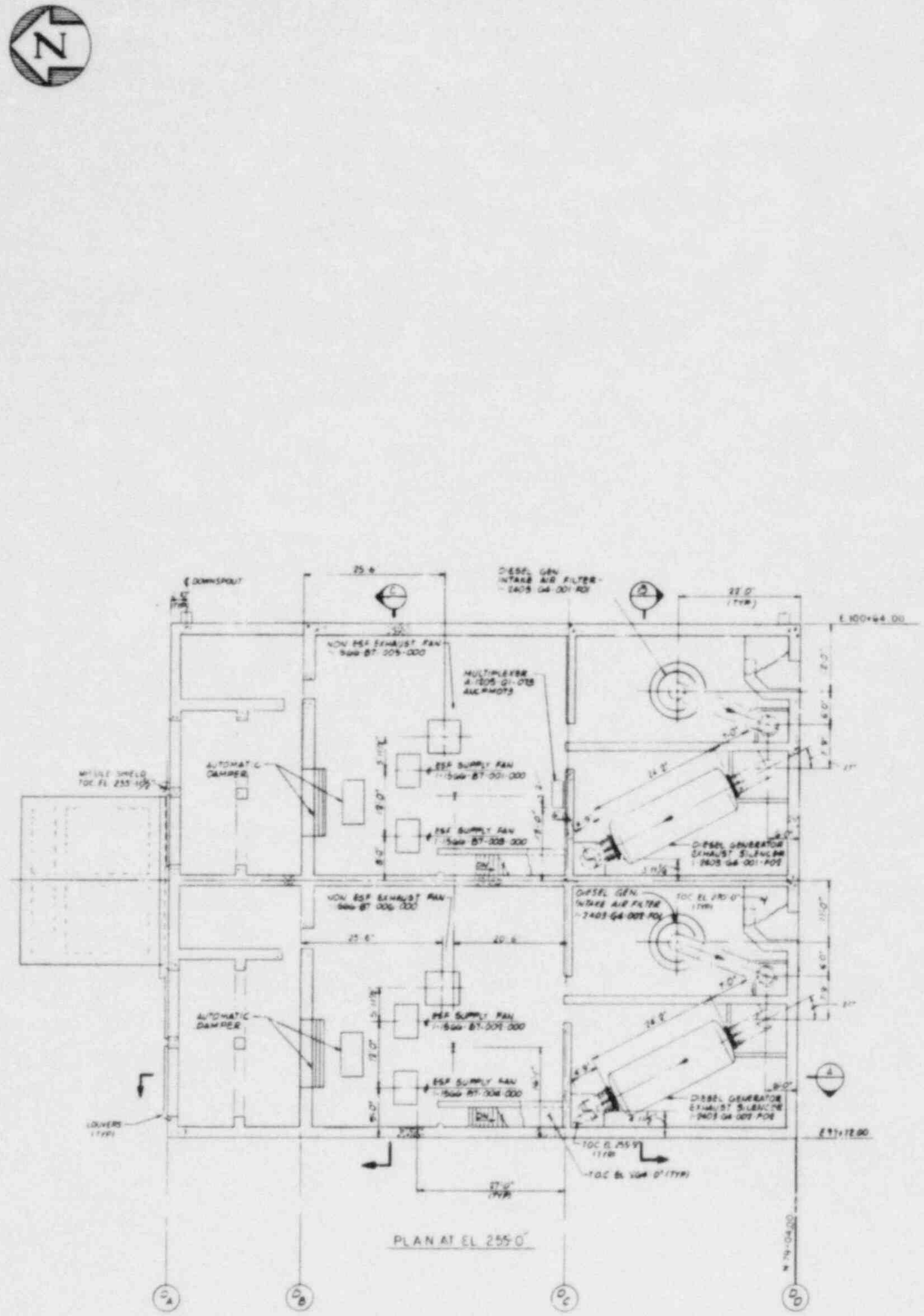


Figure 1
LOCATION OF
DIESEL GENERATOR BUILDINGS



PLAN AT EL 255'-0"

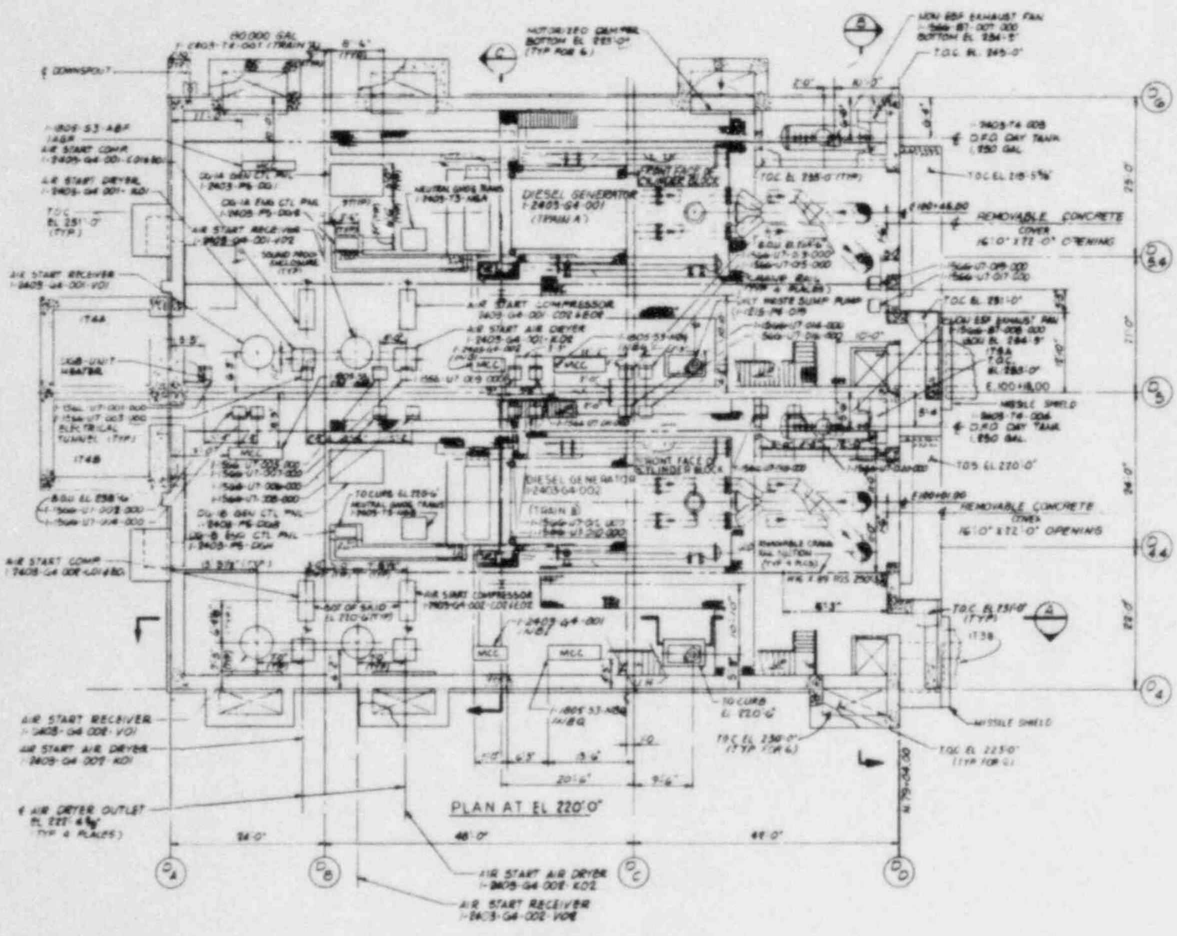
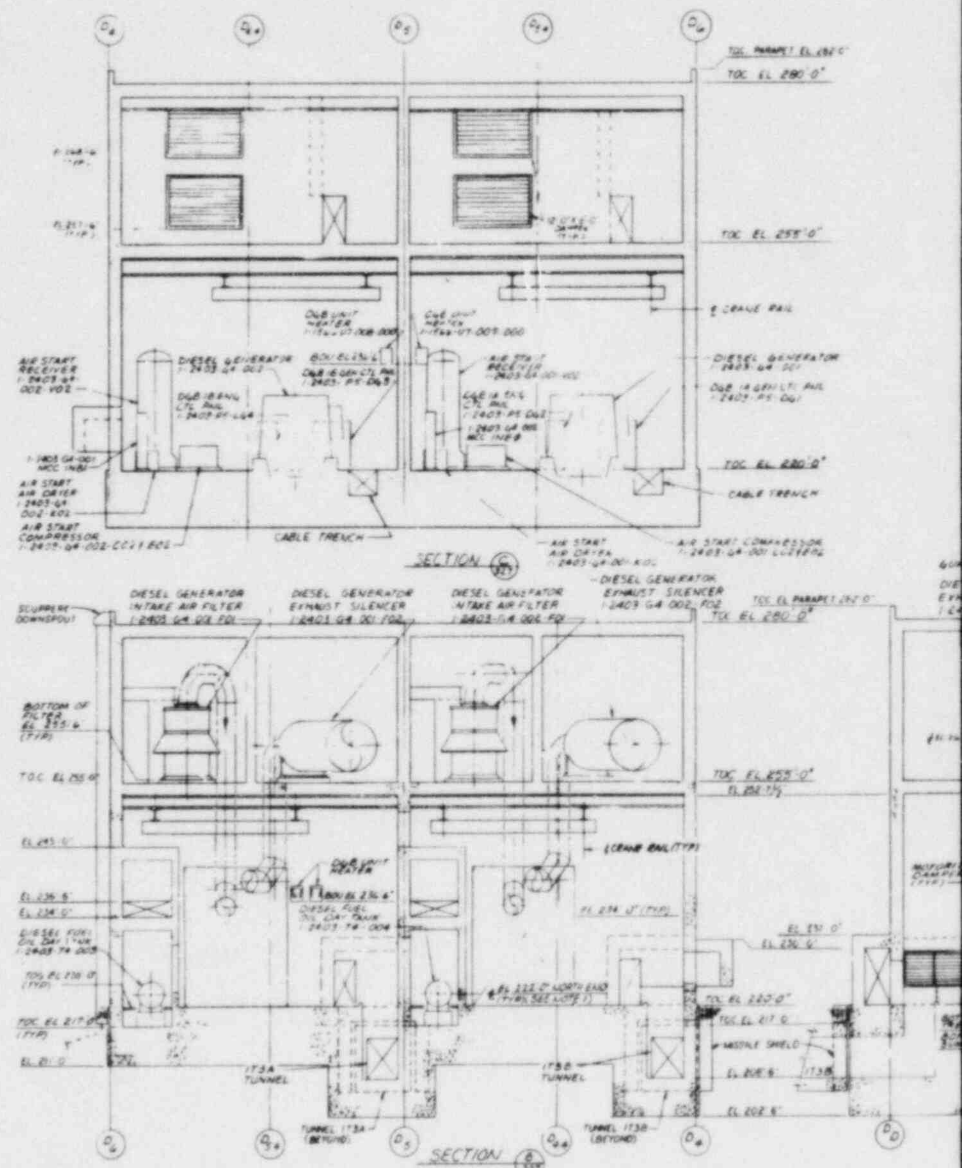


Figure 2
EQUIPMENT LOCATION LAYOUT
(Sheet 1 of 2)



TOC PARAPET EL 282'0"
TOC EL 280'0"
TOC EL 255'0"
℄ CRANE RAIL
DIESEL GENERATOR
AIR START RECEIVER
AIR START AIR DRYER
AIR START COMPRESSOR
TOC EL 220'0"
CABLE TRENCH
SECTION
DIESEL GENERATOR INTAKE AIR FILTER
DIESEL GENERATOR EXHAUST SILENCER
SCUPPERS DOWNSPOUT
BOTTOM OF FILTER
EL 245'0"
DIESEL FUEL OIL DAY TANK
℄ CRANE RAIL (TYP)
TUNNEL
MISSILE SHIELD
EL 202'6"
SECTION

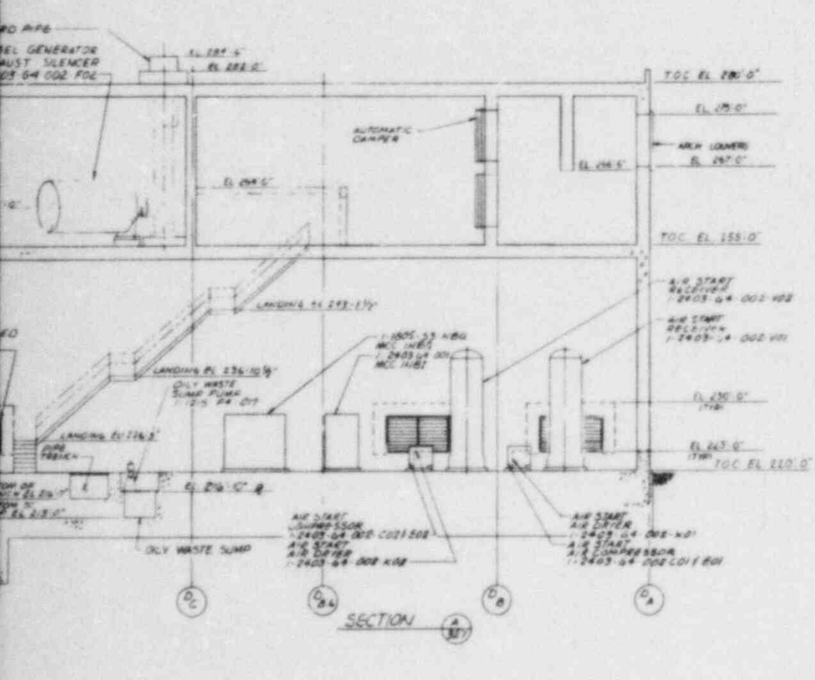


Figure 2
EQUIPMENT LOCATION LAYOUT
(Sheet 2 of 2)

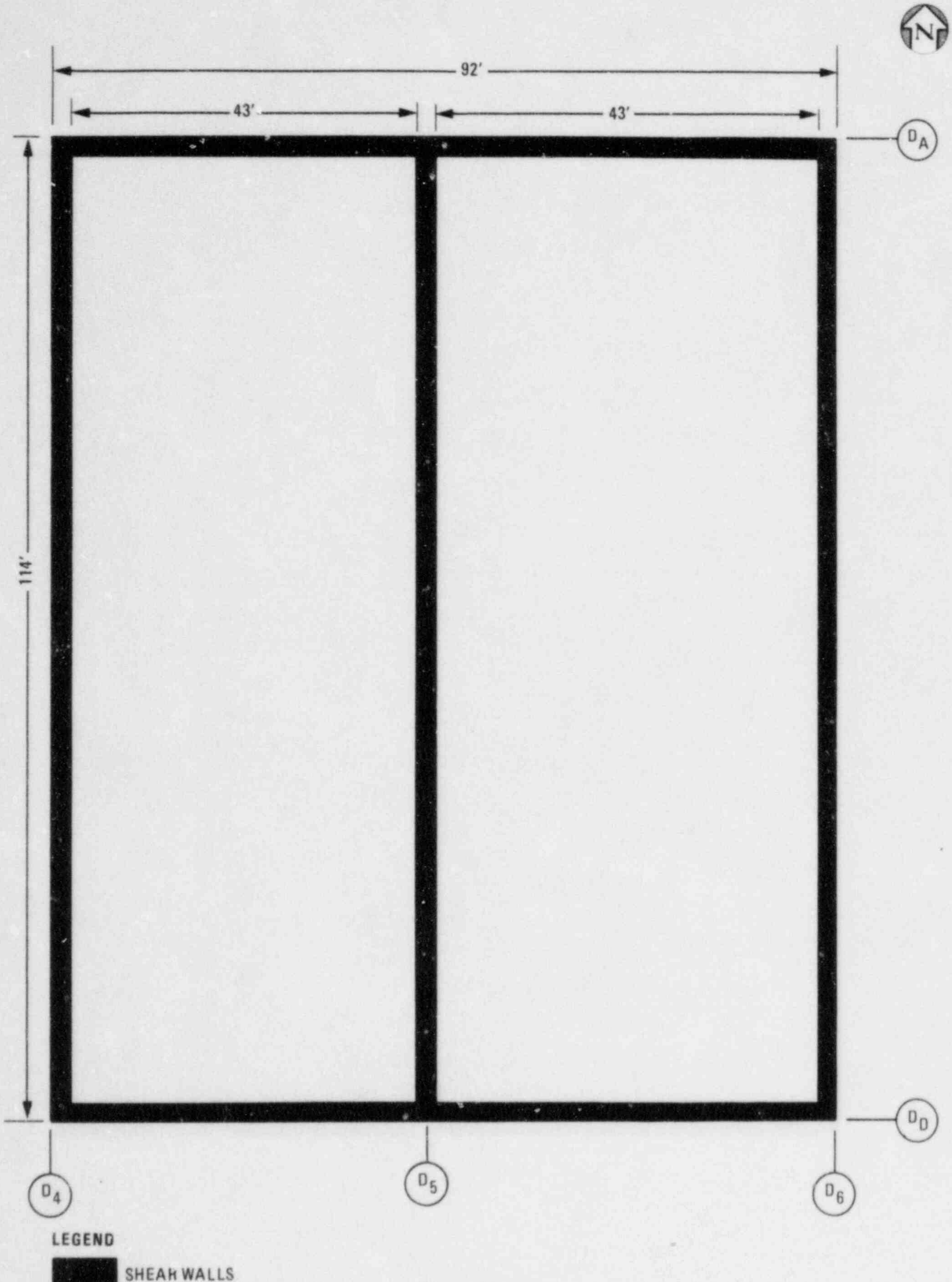


Figure 3
EL. 220'-255' LOCATION OF KEY
STRUCTURAL ELEMENTS

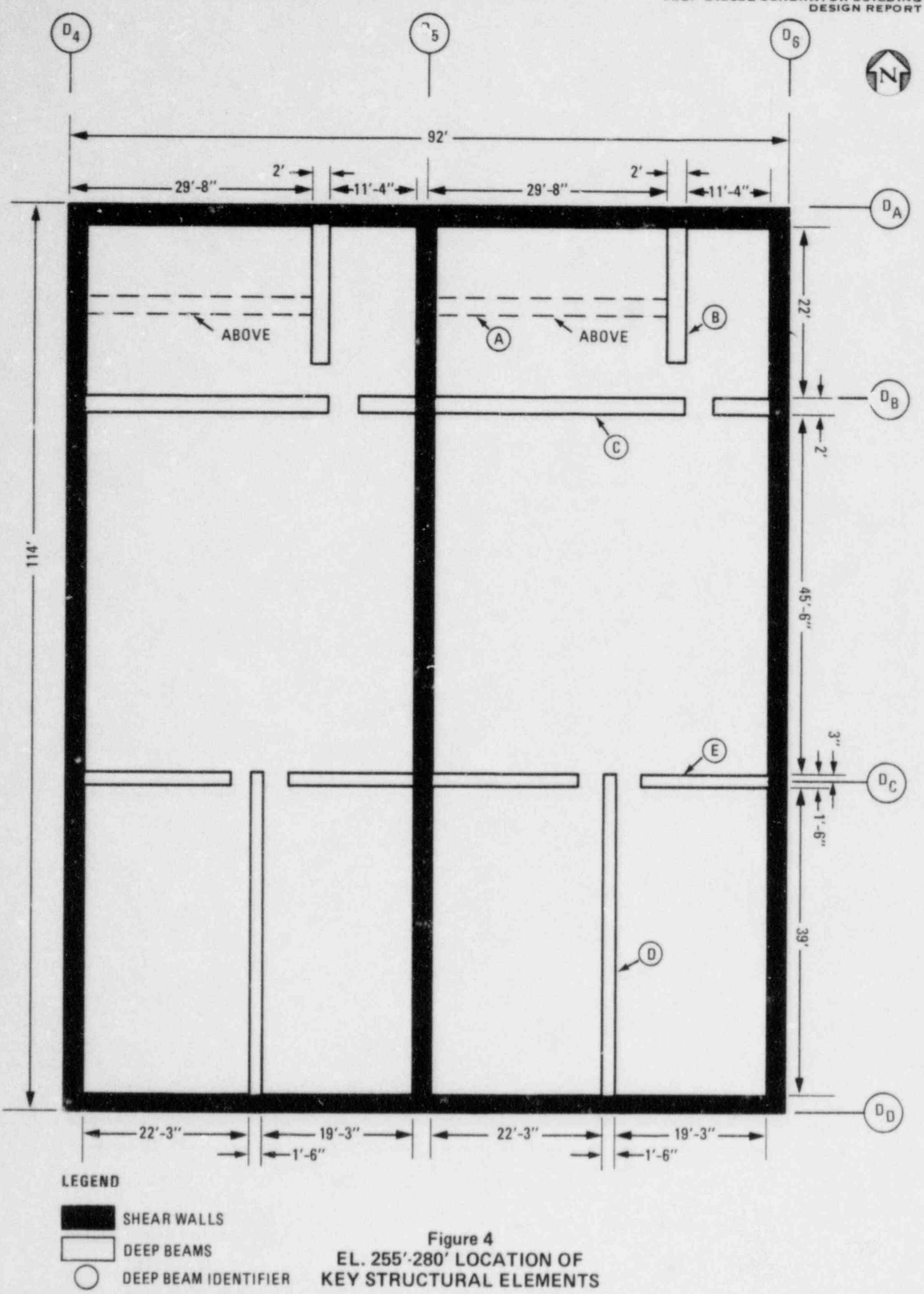
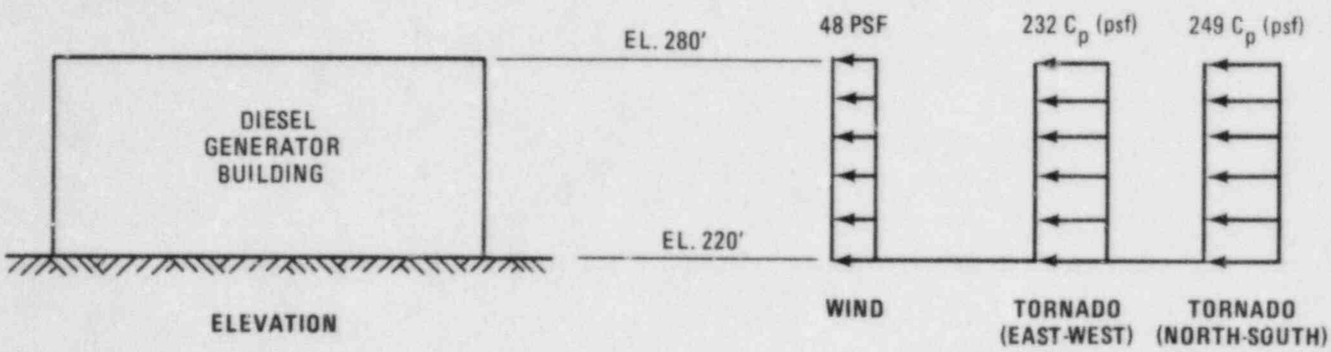


Figure 4
EL. 255'-280' LOCATION OF
KEY STRUCTURAL ELEMENTS



$P = C_s P_{max} C_p$

WHERE:

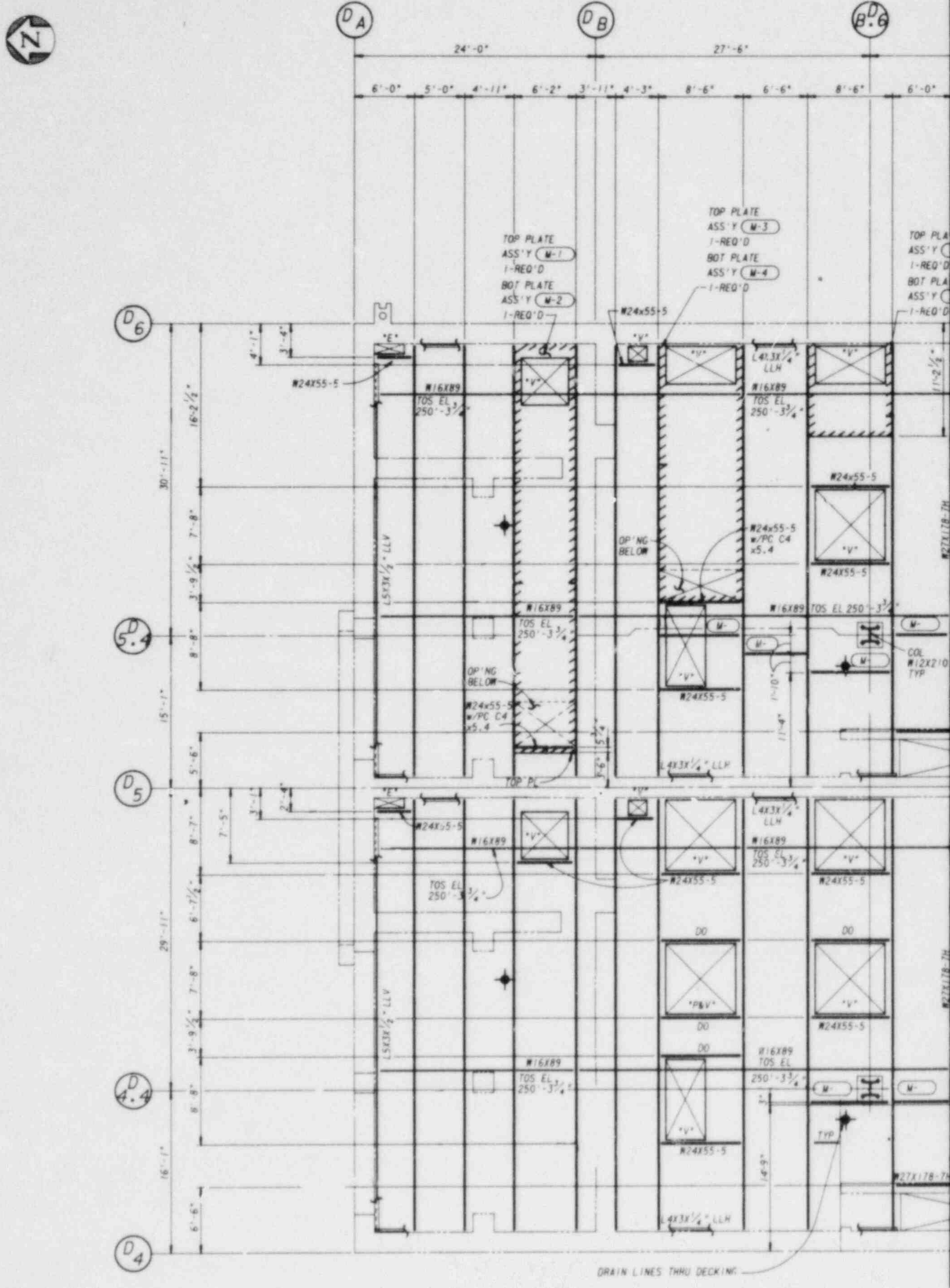
C_s = SIZE COEFFICIENT
= .75

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

C_p = EFFECTIVE EXTERNAL PRESSURE COEFFICIENT

$P = (.75)(332\text{ psf})\,C_p$
$= 249\,C_p$ (psf)

Figure 5
WIND AND TORNADO EFFECTIVE VELOCITY
PRESSURE PROFILES



DA
DB
B.6
24'-0"
27'-6"
6'-0"
5'-0"
4'-11"
6'-2"
3'-11"
4'-3"
8'-6"
6'-6"
8'-6"
6'-0"
TOP PLATE
ASS'Y W-1
1-REQ'D
BOT PLATE
ASS'Y W-2
1-REQ'D
TOP PLATE
ASS'Y W-3
1-REQ'D
BOT PLATE
ASS'Y W-4
1-REQ'D
D6
D5.4
D5
D4.4
D4
W24X55-5
W16X89
TOS EL 250'-3 3/4"
L4X3X1/4" LLH
L5X3X1/2" LLV
OP'NG BELOW
W24x55-5 w/PC C4 x5.4
COL W12X210 TYP
TOP PL
"E"
"V"
"P&V"
DO
TYP
W27X178-7H
30'-11"
16'-2 1/2"
7'-8"
3'-9 1/2"
8'-8"
15'-1"
5'-6"
8'-7"
7'-5"
6'-7 1/2"
29'-11"
16'-1"
6'-6"
14'-9"
11'-10"
11'-4"
DRAIN LINES THRU DECKING

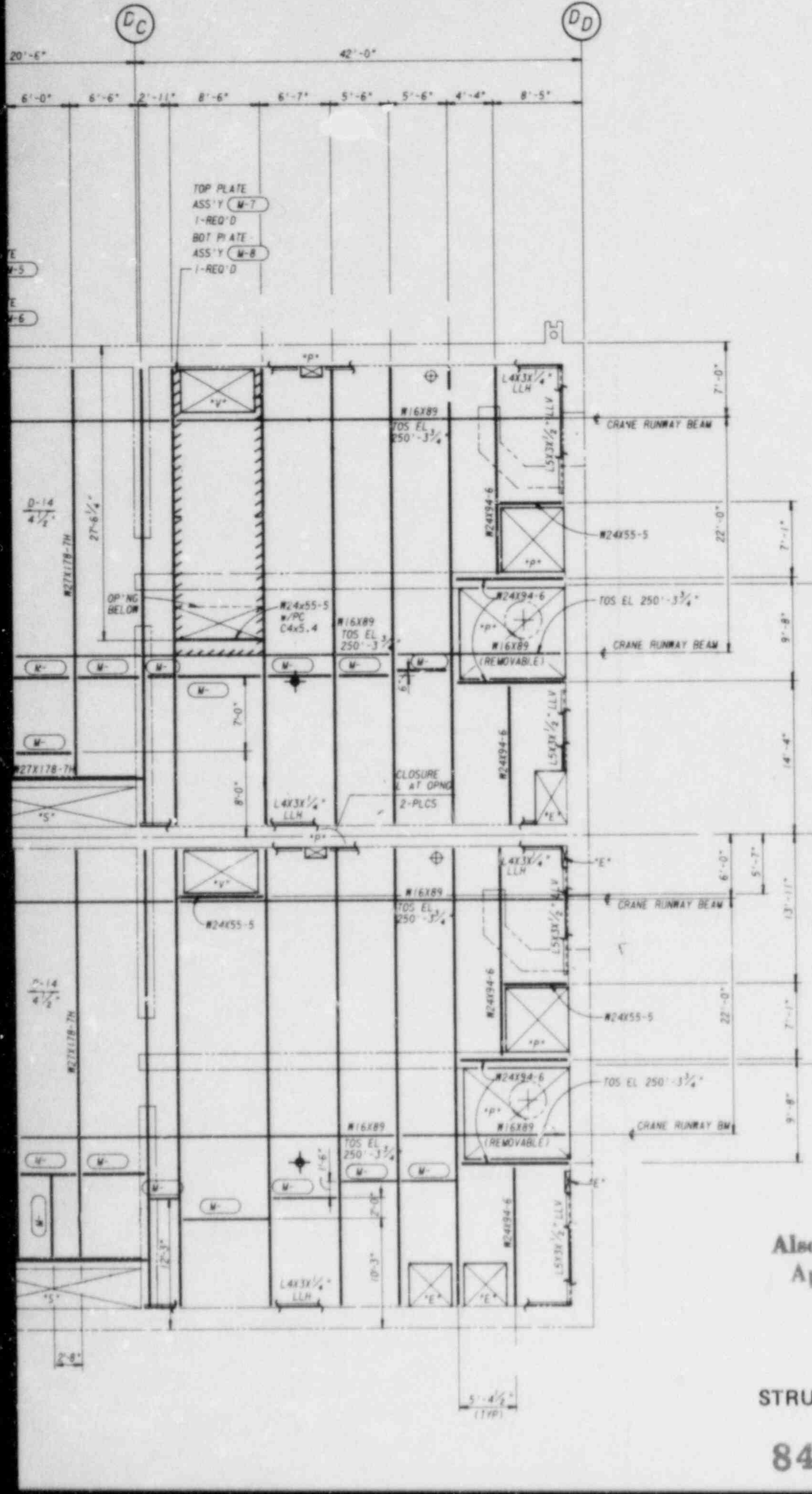
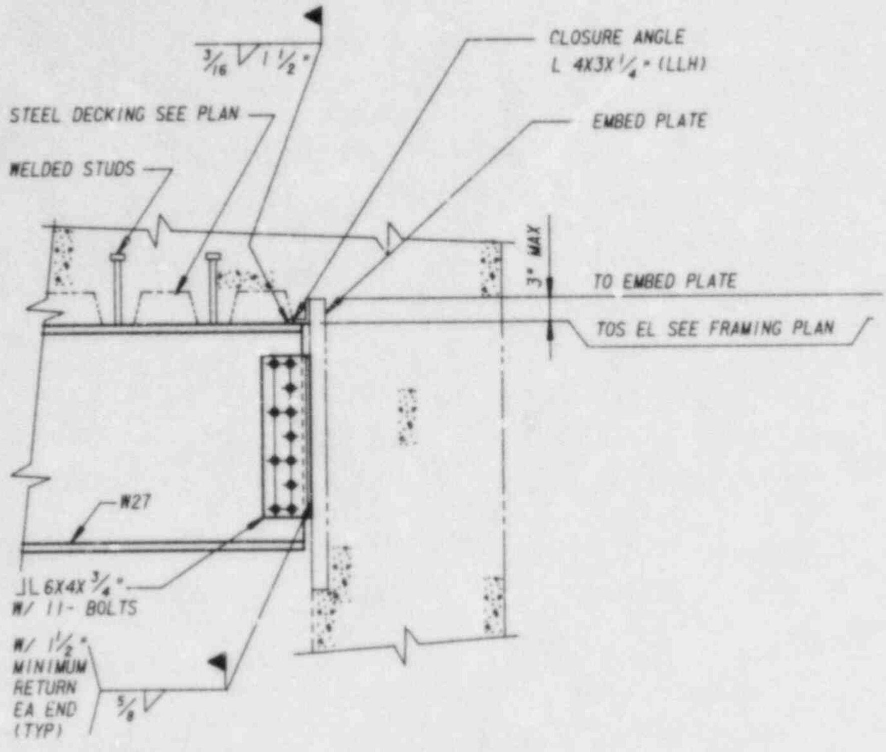
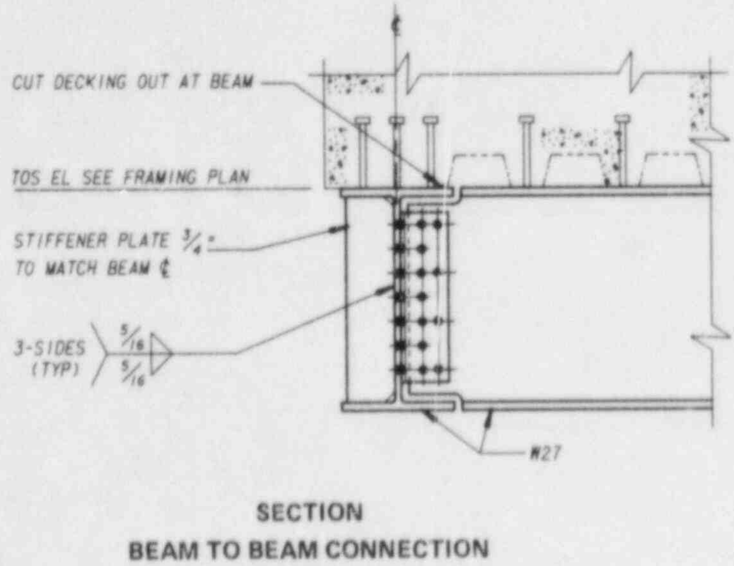


Figure 6
STRUCTURAL STEEL FRAMING
PLAN AT EL. 255'-0"

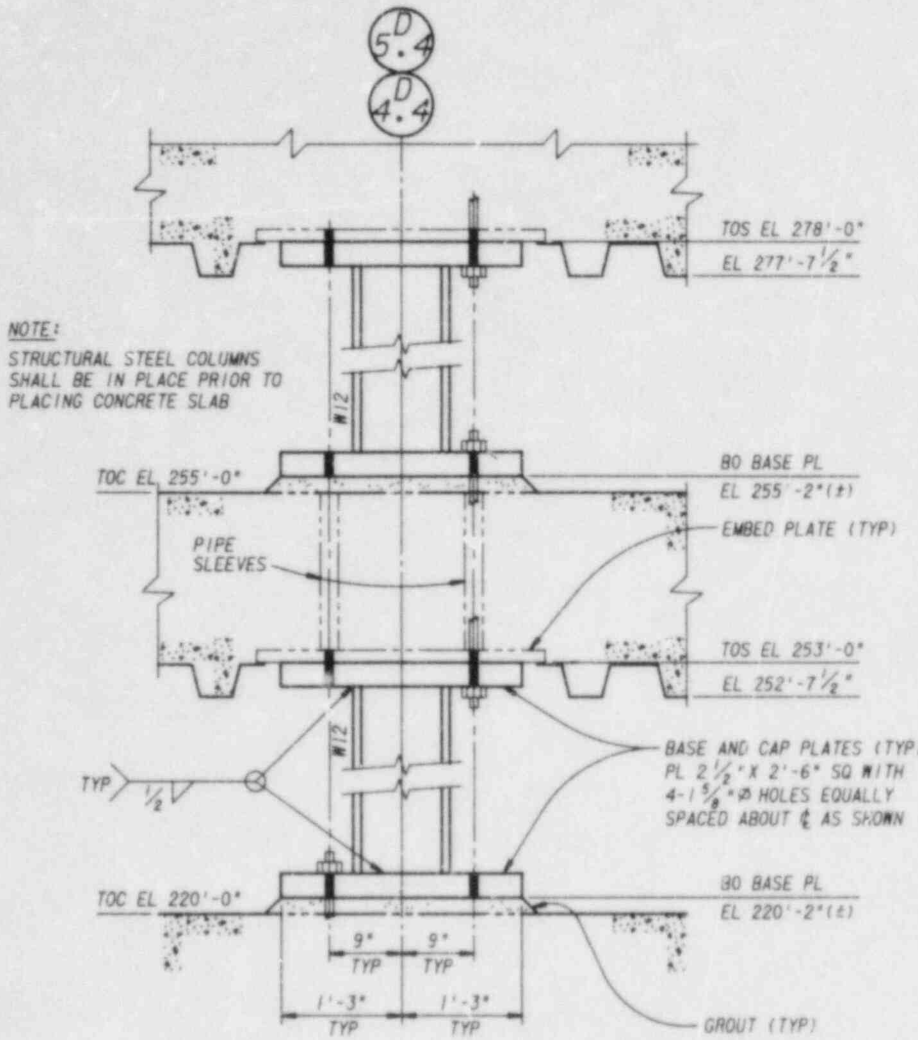


SECTION
SLAB/BEAM TO WALL CONNECTION

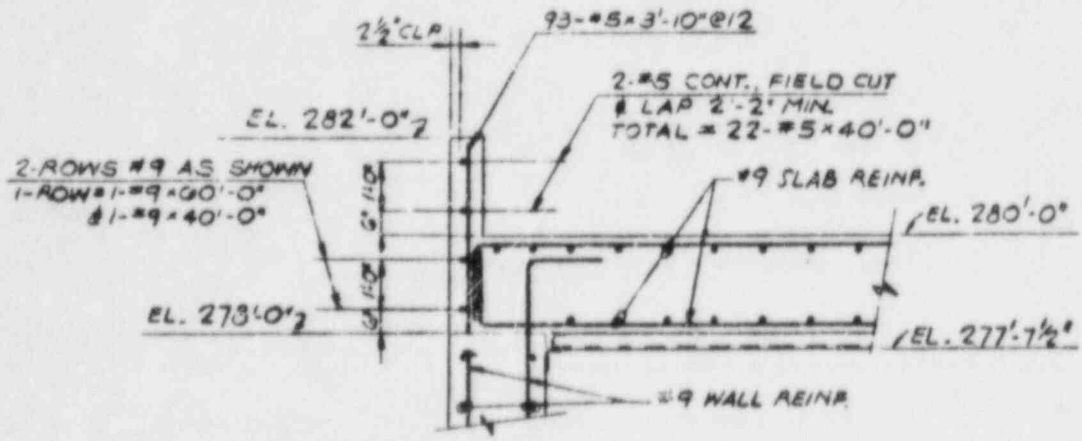


SECTION
BEAM TO BEAM CONNECTION

Figure 7
REPRESENTATIVE SLAB DETAILS
(Sheet 1 of 2)

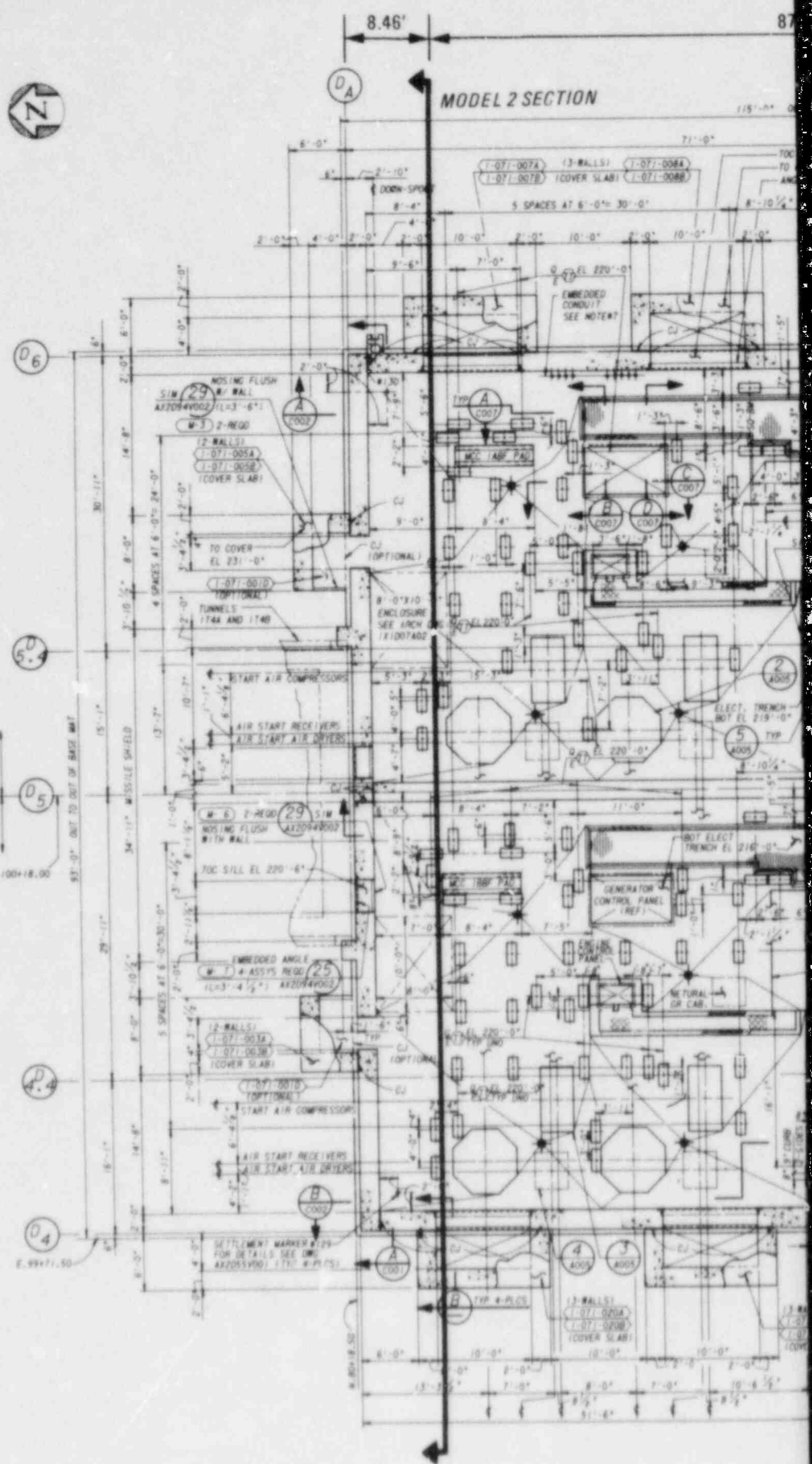


COLUMN USED TO VERTICALLY STIFFEN SLABS

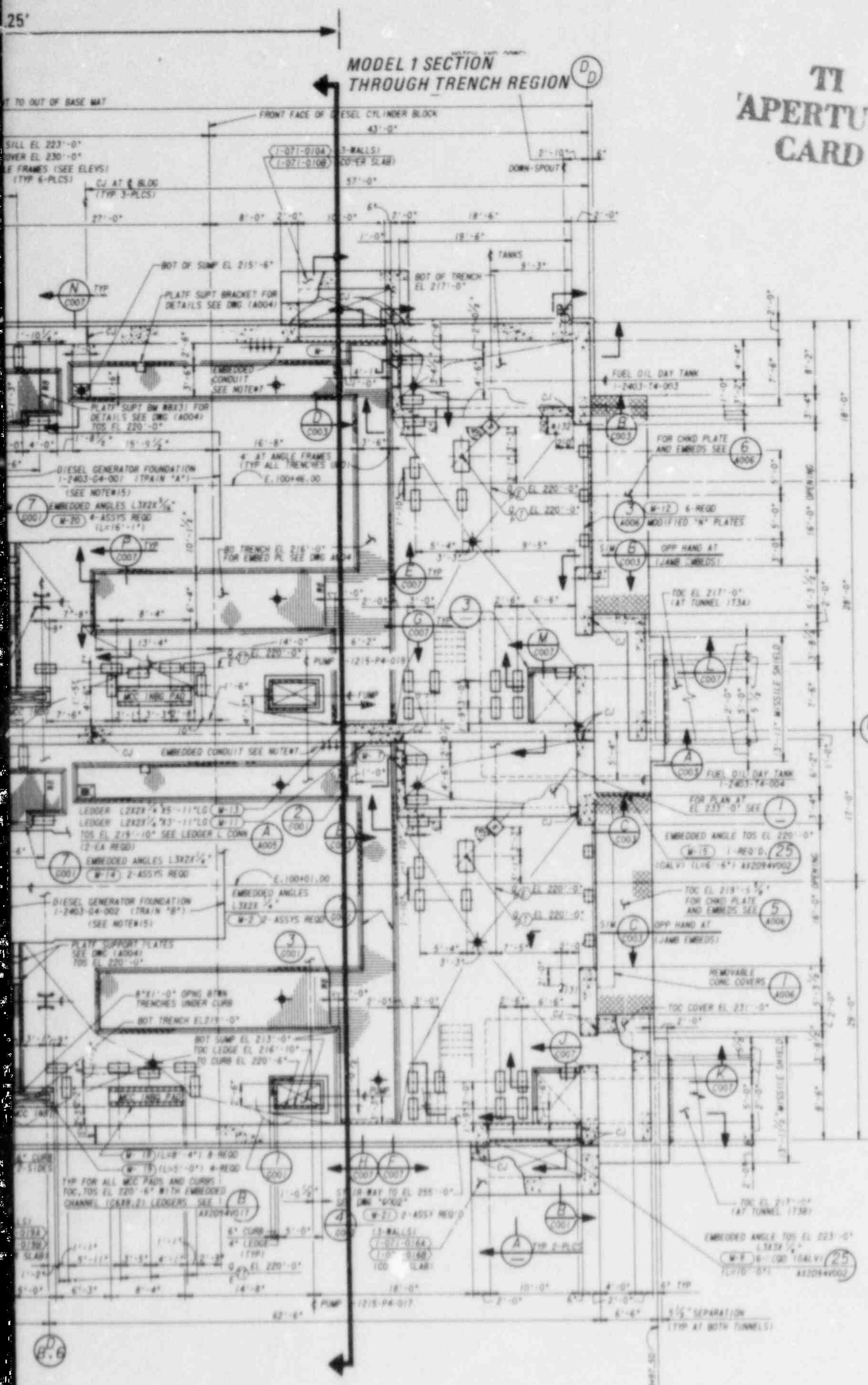


ROOF SLAB TO WALL JOINT

Figure 7
REPRESENTATIVE SLAB DETAILS
(Sheet 2 of 2)

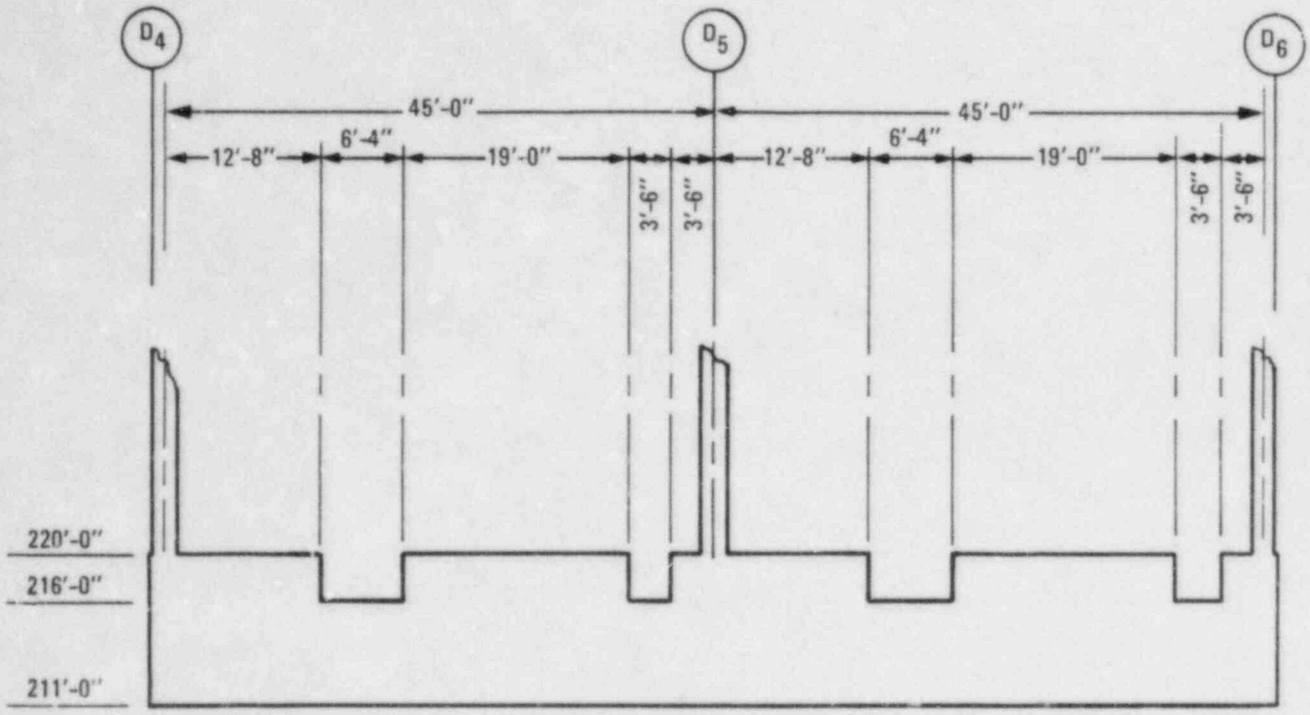

8.46'
MODEL 2 SECTION
NOSING FLUSH W/ WALL
TO COVER EL 231'-0"
TUNNELS 1T4A AND 1T4B
START AIR COMPRESSORS
AIR START RECEIVERS
AIR START AIR DRYERS
MISSILE SHIELD
TOC SILL EL 220'-6"
EMBEDDED ANGLE
GENERATOR CONTROL PANEL (REF)
ELECT. TRENCH
(COVER SLAB)
(OPTIONAL)
SETTLEMENT MARKER
E.100+18.00
E.99+71.50

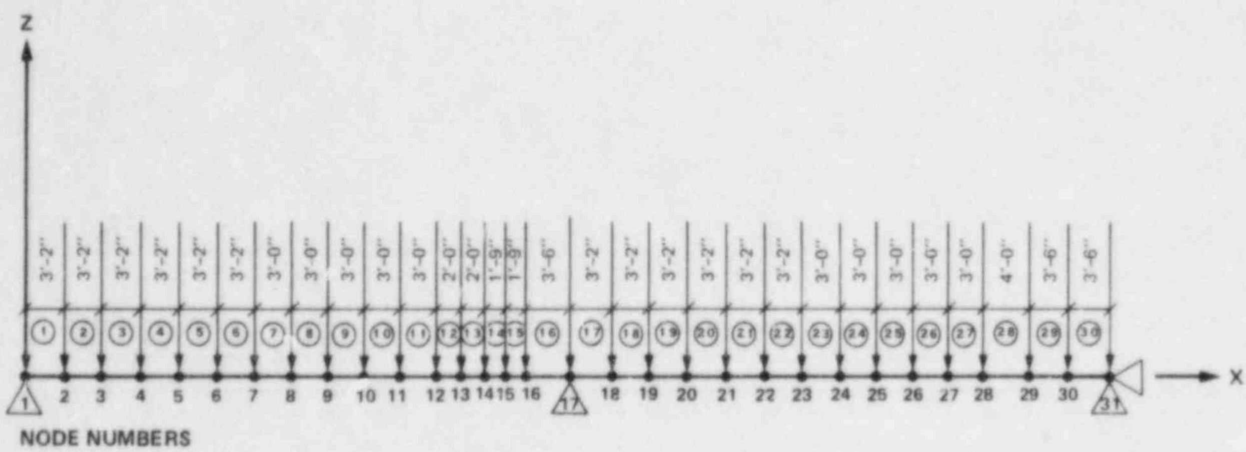
LOCATIO



N OF COMPUTER MODEL SECTIONS

Figure 8
BASEMAT COMPUTER MODEL
(Sheet 1 of 3)





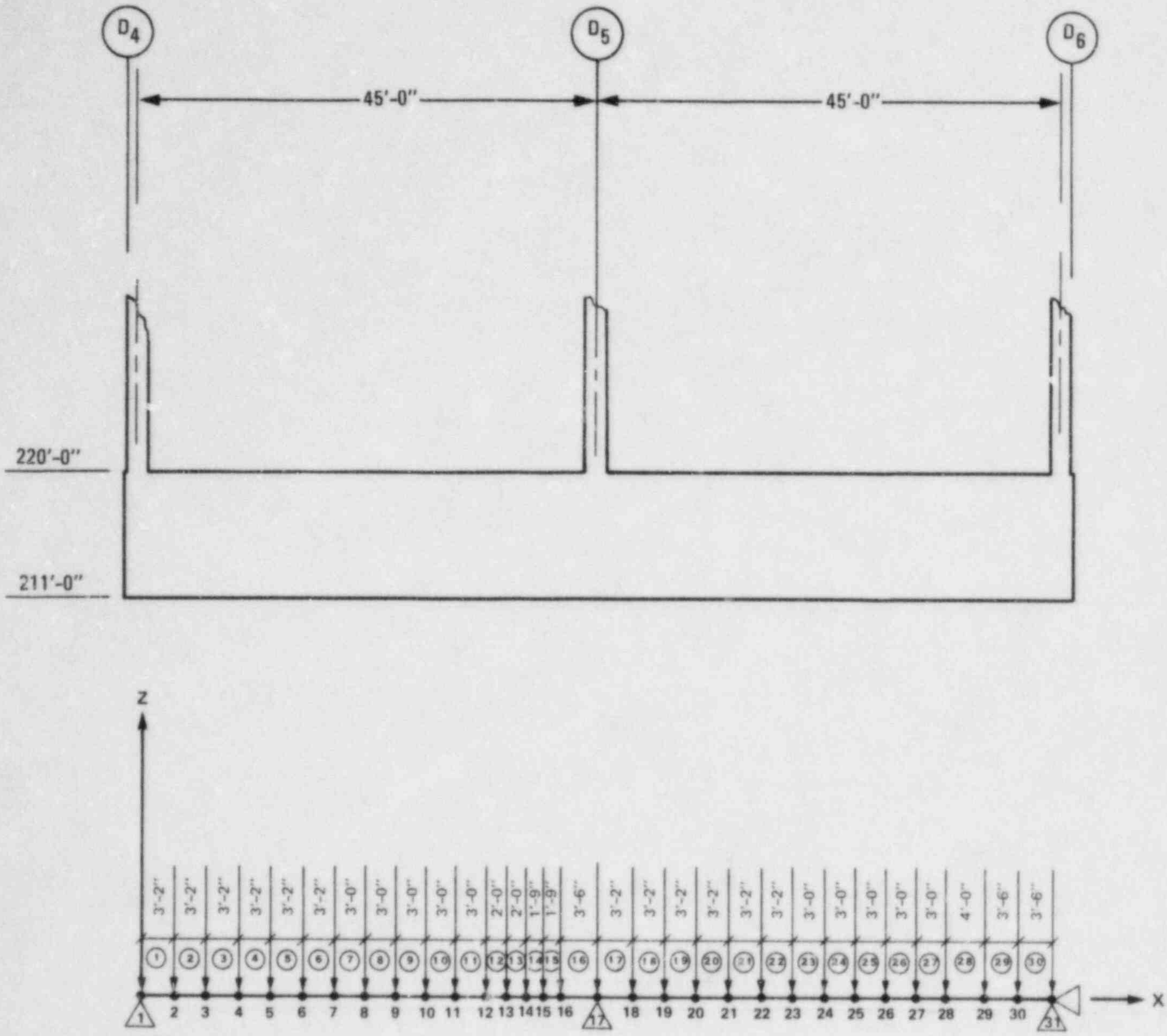
BASEMAT MODEL 1

NOTE:

△ – INDICATES TRANSLATIONAL RESTRAINT

○ – INDICATES BEAM ELEMENT NUMBERS

Figure 8
BASEMAT COMPUTER MODEL
(Sheet 2 of 3)



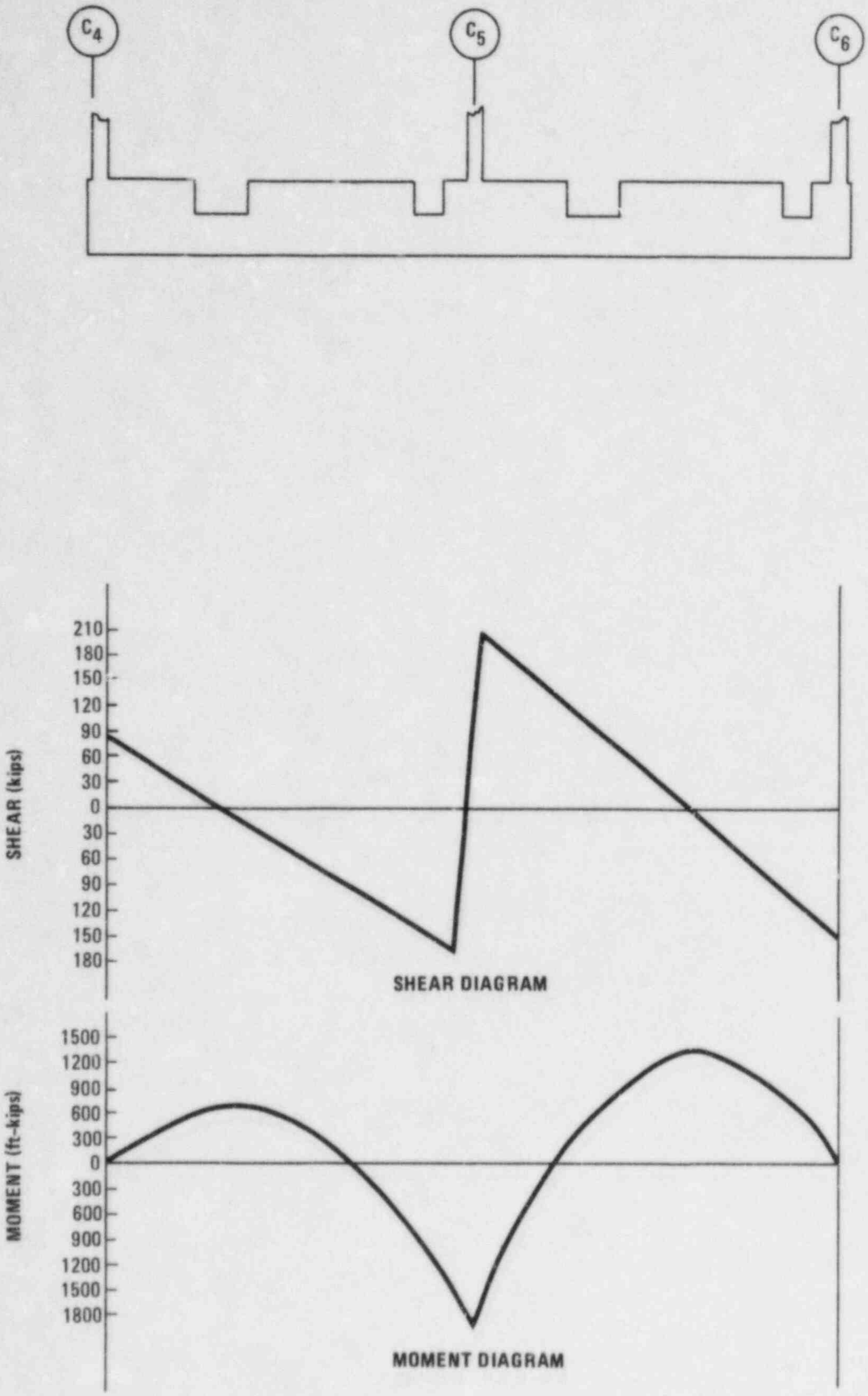
BASEMAT MODEL 2

NOTE:

△ – INDICATES TRANSLATIONAL RESTRAINT

○ – INDICATES BEAM ELEMENT NUMBERS

Figure 8
BASEMAT COMPUTER MODEL
(Sheet 3 of 3)



LOAD COMBINATION 3

Figure 9
REPRESENTATIVE BASEMAT ANALYSIS RESULTS

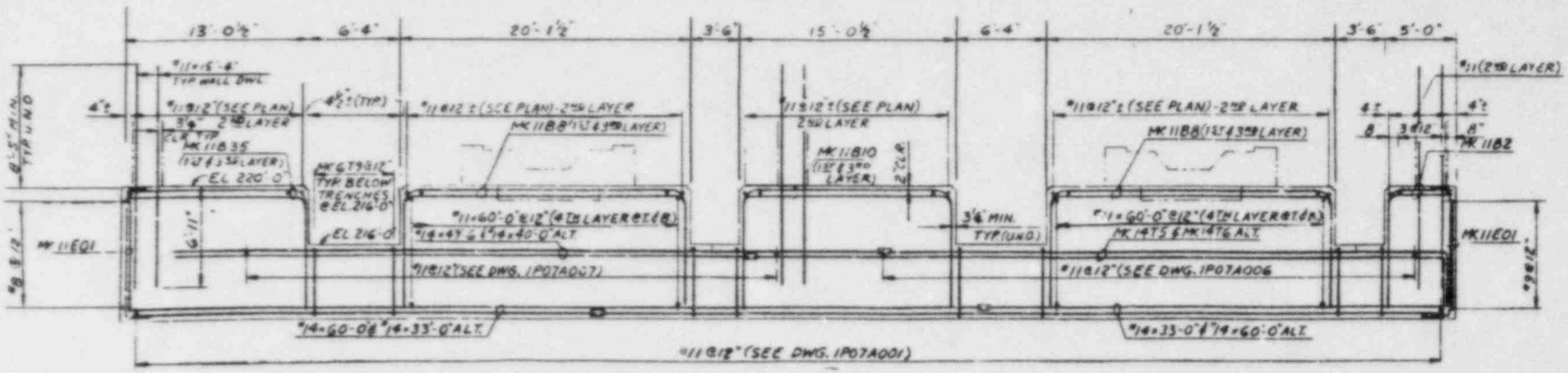


Figure 10
BASEMAT REINFORCING DETAILS

APPENDIX A

DEFINITION OF LOADS

APPENDIX A

DEFINITION OF LOADS

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

A.1 NORMAL LOADS

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

D — Dead loads or their related internal moments and forces, including hydrostatic loads and any permanent loads except prestressing forces.

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

T_o — Thermal effects and loads during normal operating or shutdown conditions, based on the most critical transient or steady-state condition.

R_o — Pipe reactions during normal operating or shutdown conditions, based on the most critical transient or steady-state conditions.

A.2 SEVERE ENVIRONMENTAL LOADS

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

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

A.3 EXTREME ENVIRONMENTAL LOADS

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

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

A.4 ABNORMAL LOADS

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

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

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

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

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

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

APPENDIX B

LOAD COMBINATIONS

APPENDIX B

LOAD COMBINATIONS

B.1 STEEL STRUCTURES

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

B.2 CONCRETE STRUCTURES

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

TABLE B.1[(a)]

STEEL DESIGN LOAD CO NATIONS
ELASTIC METH

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

---

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

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

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

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

TABLE B.2[a][f]

CONCRETE DESIGN LOAD COMBINATIONS
STRENGTH METHOD

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

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

APPENDIX C

DESIGN OF STRUCTURES FOR TORNADO MISSILE IMPACT

APPENDIX C

DESIGN OF STRUCTURES FOR TORNADO MISSILE IMPACT

C.1 INTRODUCTION

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

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

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

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

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

C.1.1 Procedures

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

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

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

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

d. Determining structural response considering other concurrent loading conditions.

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

C.2 LOCAL EFFECTS

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

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

C.2.1 Reinforced Concrete Elements

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

C.2.2 Steel Elements

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

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

where:

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

E_k = missile kinetic energy (ft-lb).

M_m = mass of the missile (lb-s^2/ft).

V_s = missile striking velocity (ft/s).

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

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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.

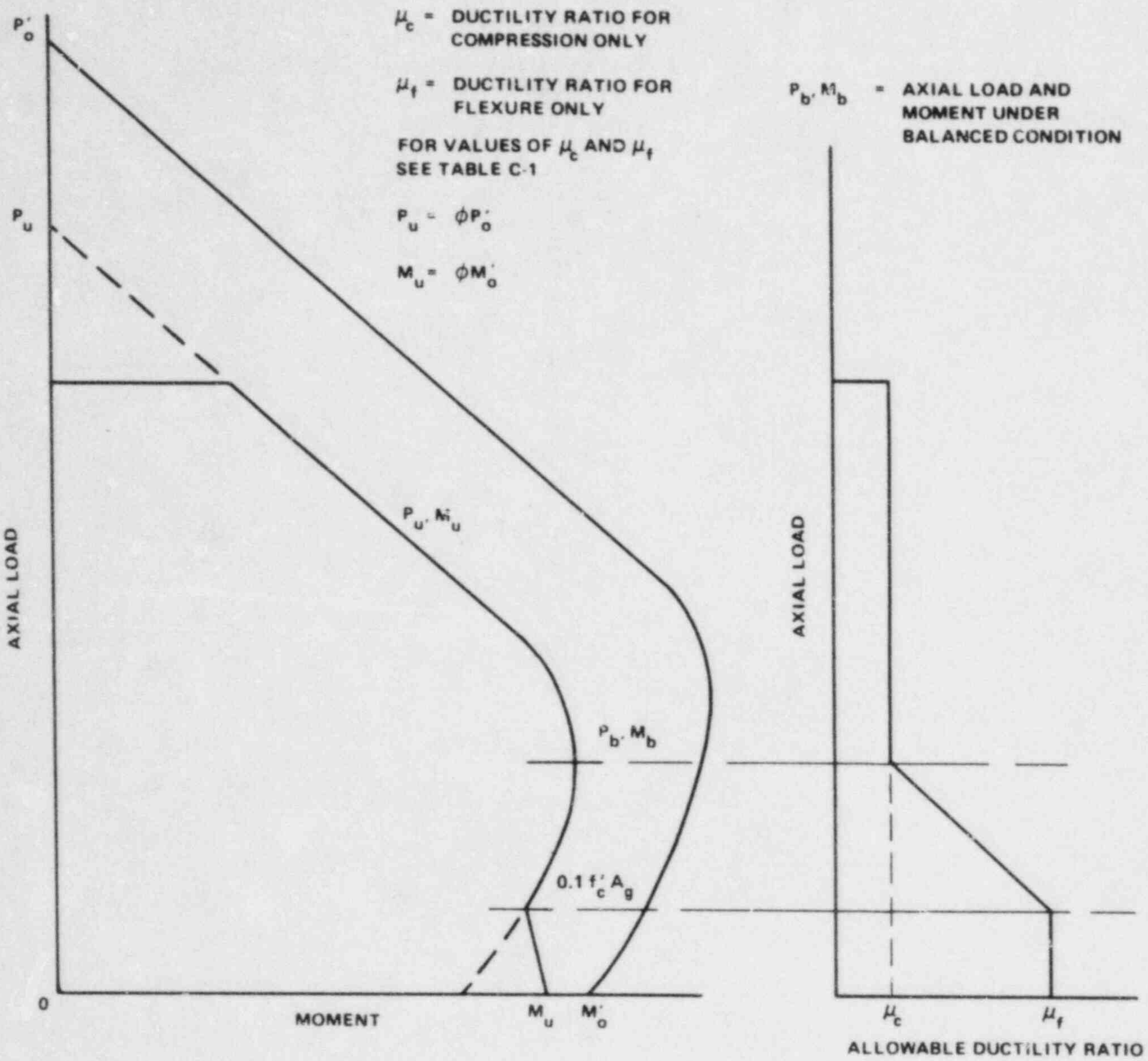
(3) Ductility ratio up to 10 can be used without an angular rotation check.

(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