VOGTLE ELECTRIC GENERATING PLANT

GEORGIA POWER COMPANY

CONTROL BUILDING DESIGN REPORT

Prepared

by

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TABLE OF CONTENTS

TABLE OF CONTENTS (cont)

LIST OF TABLES

LIST OF FIGURES

<u>Figure</u>

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 the 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 control 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.

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

2.0 DESCRIPTION OF STRUCTURE

2.1 GENERAL DESCRIPTION

The control building is a six-story, deeply embedded, reinforced concrete structure common to the two-unit plant. It is situated north of and adjacent to the fuel handling building and the two containment buildings, and south of the turbine building and the turbine electrical tunnel. It is separated from the surrounding structures by a 5-1/2 inch seismic gap and is supported on a mat foundation 40 feet below grade. The boxlike center section has three upper levels extending to 60 feet above grade and a partial fourth level extending an additional 20 feet. Penetration areas east and west of the center section provide access to the two containment buildings. These are the primary areas for routing of electrical and control system cables into the containment. Directly north of each containment building is the main steam isolation valve (MSIV) room which extends 40 feet above grade.

The floor at grade (level 1) is principally occupied by the control room, technical support center (TSC), office areas, equipment building, and MSIV room. The floors immediately above (level 2) and below (level A) in the center section house the cable spreading rooms. The lowest level (level B) houses switchgear and heating, ventilating, and air conditioning (HVAC) equipment. The third and fourth floors mainly contain HVAC equipment. The fourth floor, TSC, and areas between column lines C4 to C8 above elevation 220'-0", are primarily occupied by nonsafety-related components.

Access shafts number 3, providing access to the containment tendon gallery and one buttress for each unit, are formed by portions of the control building.

Figure 1 shows the location of the control building with respect to the other plant structures while figures 2 through 5 show the general layout.

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 control building is supported on a reinforced concrete mat foundation (basemat) 40 feet below grade (top of basemat elevation 180'-0"). The appproximate plan dimension of the basemat is 169 feet wide by 525 feet long. The basemat is a minimum of 7 feet thick with an increase in thickness to 10 feet adjacent to the containment building and localized increases around basemat penetrations. The basemat is founded on approximately 40 feet of Category 1 backfill placed on the marl bearing stratum. The 10-foot-thick portion of the basemat is founded approximately 5 feet above the high water table.

The location of adjacent structures with respect to the control building, and their basemat elevations, are summarized as follows (also refer to figure 1):

- Turbine Building - Located approximately 27 feet to the north; top of basemat elevation 195'-0", bottom of basemat elevation 186'-0".

- Containment Building - Located immediately adjacent and south of the penetration (wing) areas; top of basemat elevation 169'-0", bottom of basemat elevation 158'-6".
- Fuel Handling Building - Located immediately adjacent and south of the center section; top of basemat elevation 179'-0", bottom of basemat elevation 173'-0".
- Turbine Electrical Tunnel - Located immediately adjacent and to the north between the turbine building and the north wall; top of tunnel elevation 215'-0", bottom of tunnel elevation 197'-0".

2.3 GEOMETRY AND DIMENSIONS

Figures 2 through 5 show the outline dimensions of the control building. The geometry of the Unit 2 portion of the structure is mirror image to the Unit 1 portion except between column lines C_4-C_8 and $C_{A.6}$-C_G (Unit 2, penetration area) where there is an additional level to the structure. For the Unit 1 penetration area of the structure the roof slab is at elevation 240'-0", while this area for the Unit 2 side extends up to a roof slab at elevation 260'-0".

The approximate maximum plan outline dimensions for each level of the structure are as follows:

Level - Elevation	Maximum Plan Dimensions (E-W x N-S)	Remarks
B - 180'-0" (Basemat)	525 ft x 169 ft	Note 1: Dimension includes Unit 2 penetration area and Unit 1&2 center portion.
A - 200'-0"	522 ft x 169 ft	
1 - 220'-0"	520 ft x 155 ft	Note 2: Dimension is for the MSIV room. There is one room for each unit.
2 - 240'-0"	520 ft x 155 ft	
3 - 260'-0"	237 ft x 155 ft (1) 2 - 50 ft x 60 ft (2)	
4 - 280'-0"	152 ft x 155 ft	
Roof - 301'-0"	152 ft x 86 ft	

2.4 KEY STRUCTURAL ELEMENTS

The key structural elements of the control building are the roof and floor slabs, structural steel girders, reinforced concrete columns, shear walls, and the basemat. The structural analysis and design for each of these elements is described in section 4 of this design report.

The roof and floor slabs are generally formed with metal decking and are 24 inches and 18 inches thick respectively (including decking) in areas housing safety-related equipment. The structural steel girders consist of standard rolled sections and built-up plate members that vary in depth from 24 to 84 inches. The reinforced concrete columns vary in size from 24 inches square to 60 inches square. The shear walls vary in thickness from 24 inches to 66 inches. The location of the roof and floor slabs, the structural steel girders, the concrete columns, and the shear walls for representative floor levels is shown in figures 6 through 8.

2.5 MAJOR EQUIPMENT

In the design of the control building a minimum dead load of 50 psf is considered to account for permanently attached small equipment, piping, conduits and cable trays. The use of this minimum uniform 50 psf dead load conservatively envelops the weight of equipment with an individual weight less than 25 kips, and thus "major equipment" is defined as equipment with a weight of 25 kips or more. Listed below are the major equipment considered in the structural analysis and design of the control building.

Level	Equip. Description	Equipment Design Weight (kips)
B (Basemat)	None	-
A	4160 V Switchgear (4 Units)	71
1	CTB Normal Purge Exhaust Unit (2 Units)	30
2	None	-
3	Filter Unit W/AC (4 Units)	55
	ESF Chiller (4 Units)	37
4	Normal Chiller with Compressor (3 Units)	59
Roof	None	-

3.0 <u>DESIGN BASES</u>

3.1 CRITERIA

The following documents are applicable to the design of the control 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 of Testing 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 control building is designed for all credible loads and load combinations. The load terms are listed and defined in Appendix A.

3.2.1 Normal Loads

3.2.1.1 Dead Load (D)

The equipment dead loads considered range from 50 to 400 psf and as noted in section 2.5 the minimum uniform 50 psf dead load conservatively envelops the weight of equipment with an individual weight less than 25 kips. Additionally, the dead weight of equipment with an individual weight greater than 25 kips and the dead weight of structural members is considered.

3.2.1.2 Live Load (L)

The live loads considered range from 0 psf (in areas where the equipment weight is included in the dead load) to 250 psf. A minimum roof live load of 30 psf envelops the effects of occupancy, snow, and 100 year rainwater ponding loads. Static lateral earth pressure due to the Category 1 backfill, the lateral earth pressure due to a 264 psf surcharge to account for incidental surface loads, and adjacent tunnel surcharges are also considered as live loads.

3.2.1.3 Thermal Loads (T_o)

During normal operating conditions, the control building inside temperature is a maximum of 100°F.

3.2.1.4 Pipe Reactions (R_o)

Significant loads due to pipe reactions during normal operating or shutdown conditions include pipe support reactions and occur only in the MSIV and main feedwater isolation valve (MFIV) areas.

3.2.2 <u>Severe Environmental Loads</u>

3.2.2.1 Operating Basis Earthquake, OBE (E)

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

The OBE damping values, as percentage of critical, applicable to the control building are as follows:

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

The dynamic incremental lateral earth pressures due to the OBE are based on the Mononobe-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 control building is designed for loads due to wind velocity of 110 miles per hour, based on a 100 year mean recurrence level of annual extreme fastest mile speed 30 feet above the ground. The wind effective velocity pressure profile used in the design (see figure 10) is in accordance with reference 1. Coefficients are based on Exposure C, applicable for flat open country. The pressure values take into account the dynamic response due to gusts and ignore any shielding effects that may be provided by adjacent structures.

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 response spectra at the basemat and selected elevations of the structure are discussed in the Seismic Analysis Report. Table 1 provides the SSE horizontal and vertical floor accelerations.

The SSE damping values, as percentage of critical, applicable to the control building are as follows:

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

The dynamic incremental lateral earth pressures due to the SSE are based on the Mononobe-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 (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

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 differential 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 (see figure 10) 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.

Although the control building is a partially vented structure, it is conservatively designed for an atmospheric pressure differential of ±3 psi.

Additionally, the control building is designed to withstand tornado missile impact effects from airborne objects transported by the tornado. The tornado missile design parameters are listed in table 2. Missile trajectories up to and including 45° off the horizontal use the listed horizontal velocities. Those trajectories greater than 45° use the listed vertical velocities.

3.2.3.3 Probable Maximum Precipitation (PMP) (N)

The load due to probable maximum precipitation is applied to the control 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. It is conservatively taken as a peak positive incident overpressure of 2 psi (acting inwards or outwards) applied statically.

3.2.4 Abnormal Loads

Abnormal loads generated by a postulated high-energy pipe break accident occur only in the MSIV and MFIV areas of the control building. The loads due to the various postulated pipe breaks, including pressure loads (P_a), pipe and equipment reactions (R_a), impulse generated by jet impingement (Y_j), impact load generated by pipe impact (Y_m), impulse reaction generated by pipe whip restraints (Y_r), and thermal loads generated by the pipe break (T_a) are considered in the structural design evaluation of the MSIV and MFIV areas.

3.3 LOAD COMBINATIONS AND STRESS/STRENGTH LIMITS

The load combinations and stress/strength limits for the control building 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 control building.

3.4.1 Concrete

- Compressive strength — $f_c' = 4$ ksi
- Modulus of elasticity — $E_c = 3605$ ksi
- Shear modulus — $G = 1540$ 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 Bolts

3.4.4.1 ASTM A325: (1/2 inch to 1 inch diameter inclusive)

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

3.4.4.2 ASTM A325: (1 1/8 inch to 1 1/2 inch diameter inclusive)

- Minimum yield stress F_y = 81 ksi
- Minimum tensile strength F_{ult} = 105 ksi

3.4.4.3 ASTM A307

- Minimum yield stress F_y is not applicable
- Minimum tensile strength F_{ult} = 60 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 γ_m = 126 pcf
- Saturated unit weight γ_t = 132 pcf
- Shear modulus

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

- Angle of internal friction ϕ = 34°
- Cohesion c = 0

3.4.5.3 Modulus of Subgrade Reaction

- Static 60 kcf
- Dynamic 85 kcf

3.4.5.4 Net Bearing Capacities

- Ultimate 57.8 ksf
- Allowable static 19.3 ksf
- Allowable dynamic 28.9 ksf

4.0 STRUCTURAL ANALYSIS AND DESIGN

This section provides the methodologies employed to analyze the control 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 radiation shielding and 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 or computer methods. In the manual analysis, the building structure or sub-structure is considered as an assemblage of slabs, girders, walls and columns, and the analysis is performed using standard structural analysis techniques. In the computer analysis, the building structure or sub-structure 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 analysis 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 combination that governs 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 forms the basis for the overall structural analysis and design of the control building.

All other load combinations, including the effects of abnormal loads and tornado loads, are evaluated where applicable on a local area basis, (i.e., sections 5.2 and 5.3). The localized response is combined with the analysis results of the overall structural response, as applicable, to confirm that design integrity is maintained.

4.2 VERTICAL LOAD ANALYSIS

The vertical load carrying elements of the control building consist of concrete slabs and steel girders that support the applied vertical loads, walls and columns that support the slabs and steel girders, 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 6 through 8.

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 and girders are analyzed for the vertical loads applied to them. The total vertical load on a wall or column 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. The basemat is analyzed for the effects of the total cumulative vertical loads from the walls and columns.

4.3 LATERAL LOAD ANALYSIS

The lateral load carrying elements of the control 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 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 6 through 8.

Since the building structure utilizes the slab diaphragms for horizontal shear distribution, the lateral load analysis is performed by a conventional rigidity and mass analysis. In this analysis, the maximum horizontal design forces for earthquake loads and soil pressure loads are applied 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. The design horizontal soil pressure load at each level of the building below grade is obtained from the lateral earth pressure with due consideration to the seismic effects and the surcharge effects from the adjacent structures (i.e., turbine electric tunnel and main steam tunnel). 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.

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

4.4 COMBINED EFFECTS OF THREE COMPONENT EARTHQUAKE LOADS

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

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

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

4.5 ROOF AND FLOOR SLABS

4.5.1 Analysis and Design Methodology

A representative slab panel framing plan (elevation 260'-0") of the control building is presented in figure 8. The figure shows the structural elements provided for vertical and lateral support of the slab panels, which consist of structural steel girders, load bearing walls and load bearing shear walls. Based on the panel configuration, the relative stiffness of the supporting members and the type of fixity provided, the slab panels are analyzed for one way or two way slab action using appropriate boundary conditions and standard beam and plate formulas.

Equivalent uniformly distributed loads are applied to the slab panels. The design vertical earthquake load at a particular level is obtained by multiplying the effective mass from the applied loading (including the slab panel's own mass) by the maximum floor acceleration at that level.

Based on the floor flexibility study, it is concluded that the effects of vertical flexibility on the control building floor accelerations and response spectra are insignificant, as long as the fundamental floor (slab-girder) system frequency is equal to or higher than 8 cps. The evaluation of the floor systems in the control building demonstrates that their frequencies are higher than this value. 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 controlling 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. Refer to figure 11. As appropriate, additional reinforcement is provided in the slab adjacent to large floor openings and in slab areas where walls above the slab are not directly supported by walls or girders below the slab.

4.5.2 Design Results

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

4.6 STRUCTURAL STEEL GIRDERS

4.6.1 Analysis and Design Methodology

A representative girder framing plan (elevation 260'-0") is presented in figure 8. Stud shear connectors are welded to the top flange of all girders to provide composite action with the slab.

Appropriate support boundary conditions are selected consistent with the support conditions of each girder. Girder moments and shears are determined using standard beam formulas.

In accordance with the stiffness criterion established for the vertical flexibility of slabs (section 4.5.1), the floor system frequency is required to be greater than or equal to 8 cps. To satisfy this criterion, the girders (in addition to strength requirements) are sized to have an appropriate natural frequency.

Uniformly distributed floor loads are converted to an equivalent uniform linear load using the tributary load method. The design vertical earthquake load for girders is obtained by multiplying the tributary mass from the applied loading (including the girder's own mass) by the maximum floor acceleration at that level. To provide additional design margin, a 5 kip load is applied to each girder to maximize the design shear and moment.

To standardize the design of the girders, they are grouped on the basis of design load intensity, girder span, and slab panel configuration. The girder selected for design to represent each group is chosen on the basis of the controlling combination of design load intensity and span. The girder is proportioned to satisfy the requirements of strength and deflection as specified in the AISC Code, and stiffness where applicable. Increase in allowable stresses is permitted in accordance with Appendix B, Table B.1.

Three major built-up steel plate girders, located at elevations 240'-0", 260'-0", and 280'-0" (between column line C_9 and C_{13}, on column line C_c), are approximately seven feet deep and are continuous over a center support with two equal spans of approximately 52 feet each. For these girders appropriate design consideration is given to web openings and full section splices.

For the design of web openings, the shear at the location of the opening is resisted by the top and bottom portions of the remaining web. The principal bending stresses in the girder are combined with the secondary bending stresses due to local cantilever action of the remaining portions of flange and web above and below the opening. Horizontal and vertical web stiffener plates are added as appropriate. Refer to figure 12.

Full girder splices are designed for the maximum shear and moment at the splice location, using bolted splice plates. Refer to figure 12.

Girder end support conditions are either web clip angle pinned connections per the AISC specification or bottom flange bearing plates secured with anchor bolts. Refer to figure 12.

4.6.2 Design Results

The design results for governing load combinations are presented in table 4 for representative structural steel girders.

4.7 REINFORCED CONCRETE COLUMNS

4.7.1 Analysis and Design Methodology

The location of columns for a representative level of the control building (between elevations 240'-0" and 260'-0") are presented in figure 8. The columns are generally square and range in width from 24 to 60 inches.

Uniformly distributed loads acting on floor slabs are applied to each column on the basis of the framing system and floor area tributary to the column. Concentrated equipment loads are appropriately distributed to adjacent columns. The vertical loads from discontinuous shear walls supported by columns are applied to the supporting columns based on the tributary shear wall length. The overturning moment from the discontinuous shear walls is applied to the columns as axial compression and tension.

The design vertical earthquake load for columns is obtained by multiplying the tributary mass from the applied loading (including the column's own mass) by the maximum floor acceleration at that level.

The structural design of reinforced concrete columns is based on strength considerations, and consists of sizing and detailing the main reinforcing steel and lateral column ties to meet the ACI 318 Code requirements. Refer to figure 13. The governing minimum eccentricity of 0.1h (h = maximum column width) is used to compute the design moment since the unbalanced moments acting on the column are insignificant. Slenderness effects are also considered in accordance with the provisions of the ACI 318 Code.

The design consists of determining the maximum ultimate capacity of each column in the form of an interaction diagram, and verifying that the applied factored design loads, P and M, fall within the design envelope.

The design capacity of each column section is determined through the use of the BIAX computer program, which determines the ultimate capacity of reinforced concrete members subjected to combined axial force and biaxial moments. The program tabulates limiting values of moment and axial force combinations.

4.7.2 Design Results

The design results for governing load combinations are presented in figures 14 through 18 for representative reinforced concrete columns.

4.8 SHEAR WALLS

4.8.1 Analysis and Design Methodology

The location of shear walls is identified in figures 6 through 8 for representative levels.

The details of the analysis methodology used to compute the total in-plane design 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 soil pressure loads on the exterior walls, as applicable, and the inertia loads on the walls due to the structural acceleration caused by the design earthquake. Soil pressure loads are applied as triangular and uniform pressure loads. The seismic inertia loads are applied as uniform pressure loads.

The design in-plane shear force and the overturning moment acting on a shear wall at a given level is 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 shear forces resulting from the out-of-plane design loads. At critical 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 in-plane shear loads is determined.

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

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

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

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

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

4.8.2 Design Results

The design results for governing load combinations are presented in table 5 for representative shear walls. Refer to figure 19 for representative shear wall design details.

4.9 BASEMAT

4.9.1 Analysis Methodology and Computer Model

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 a three-dimensional finite-element model.

The finite element model is prepared using conventional modeling techniques. The basemat and first story walls are modeled using plate elements, and boundary (spring-type) elements are used to characterize the stiffness effects of the soil.

The boundary (spring-type) elements are used in two applications, i.e., (1) as boundary conditions to characterize the translational stiffness effect of the soil as a set of elastic soil springs in the three global directions and (2) to eliminate singularity conditions by providing boundary conditions that prevent in-plane rotation of the walls that are not orthogonal to one of the horizontal global axes. The vertical stiffness of each soil spring is determined by multiplying the nodal tributary area by the modulus of subgrade reaction. The horizontal springs stiffnesses are computed to model the stiffness effect of the soil in the horizontal direction. The plate elements are used to model the structural walls in the first story above the basemat (to represent the stiffness interaction effects at the wall/basemat junction), and to model the basemat. There are a total of 1366 boundary elements, and 934 plate elements used to mathematically model the basemat.

Figure 20 shows the computer plots of the basemat model indicating node numbers and element numbers for the portion of the basemat modeled. Only one half of the basemat is modeled by accounting for the approximate symmetry of the control building in the east-west direction about column line C_{11}.

4.9.2 Application of Loads

The magnitude and distribution of loads applied to the basemat model are obtained from the cumulative results of the vertical and lateral load analyses of the overall building structure. As described in the other sections of this report, the loads include dead load, live load, vertical and horizontal seismic loads, and lateral soil pressure loads.

The resultant column loads are applied to the basemat as vertical nodal loads. The resultant wall loads are distributed to the basemat nodes at the wall-basemat junction. These vertical nodal loads are based on the ratio of the wall length tributary to a basemat node to the total wall length. The uniformly distributed loads acting on the basemat itself are applied as a uniform pressure on all basemat plate elements.

Vertical seismic loads are computed for each floor level by multiplying the dead load by the applicable maximum floor acceleration value. To simplify the computer analysis, a weighted average acceleration value was computed to determine the cumulative vertical seismic load applied to the basemat as a fraction of the dead load applied to the basemat nodes. The vertical seismic loads are thus distributed to the surface of the basemat at the base of walls and columns in direct proportion to the cumulative dead load distribution.

The cumulative horizontal seismic shear and accompanying overturning moments, which are obtained from the shear wall analysis of the structure (as described in section 4.8) are applied as nodal forces to the basemat nodes that correspond to the base of the shear walls. Since the bottom half of the first floor walls is not included in the manual computation of overturning moments, the contribution to the lateral shear and overturning moment is modeled by assigning a mass density and a horizontal acceleration to the corresponding plate elements.

4.9.3 Design Methodology

The design of the basemat, including the sizing and detailing of main reinforcing steel, is done in accordance with the ACI 318 Code. The size and spacing of steel reinforcing is determined on the basis of preliminary design. The design consists of determining the ultimate capacity of different basemat zones (refer to figure 21), in the form of interaction diagrams, and verifying that the maximum factored design moment and membrane forces for that zone fall within the design envelope.

The capacity of the basemat zones is determined through the use of the computer program OPTCON. For each zone, the maximum design moment and membrane force are determined from the basemat finite element analysis results. These values are plotted on a graph showing the membrane force-moment interaction (see figures 24 through 27). The basemat design is verified to be adequate by confirming that the plotted values lie within the interaction diagram for the maximum capacity of the section. The need for shear reinforcing is checked. The maximum design shear is determined on the basis of moment gradient.

4.9.4 Design Results

Representative results of the basemat analysis are provided in figures 22 and 23. In addition, table 6 shows elements with maximum moment due to dead loads. Table 7 shows elements with maximum moment due to north-south seismic response. Table 8 shows elements with maximum moment due to east-west seismic response. From these tables representative elements are selected and their analysis and design results are shown in figures 24 through 27.

4.10 DESIGN DETAILS

Representative design details for roof and floor slabs, structural steel girders, reinforced concrete columns, and shear walls, are provided in figures 11 through 13, and 19.

5.0 MISCELLANEOUS ANALYSIS AND DESIGN

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

5.1 STABILITY ANALYSIS

The overall stability of the control building is evaluated by determining the factor of safety against overturning, sliding, and flotation.

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 buoyancy and 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 4.

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 Flotation

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

5.1.4 Analysis Results

The minimum required factors of safety and the calculated factors of safety for stability are provided in table 9.

5.2 TORNADO LOAD EFFECTS

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

Controlling roof and exterior wall panels are evaluated for tornado load effects, and the localized response is combined with the analysis results of the overall structural response, as applicable, to confirm that design integrity is maintained. Additional reinforcing steel is provided in accordance with the ACI 318 Code, if necessary, to satisfy design requirements. In addition, barriers are provided for the openings in the exterior walls or roofs unless the systems or components located in the exterior rooms are nonsafety related. In this case, the interior walls and slabs are treated as barriers for the safety-related systems or components located in the interior rooms. Any openings in the exterior walls or slabs and the interior walls or slabs that may be susceptible to missile entry are evaluated to ensure that no safety-related systems or components are located in a potential path of the missile.

The methodology used to analyze and design the structural elements to withstand the tornado load effects is described in reference 2. Specific procedures used for analysis of missile impact effects are described in Appendix C. Representative results of the tornado missile analysis are provided in table 10.

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 ABNORMAL LOADS EFFECTS

Abnormal loads generated by a postulated high-energy pipe break occur in the MSIV and MFIV areas, and adjacent vented areas.

The MSIV/MFIV areas subjected to the abnormal loads consist of the "break exclusion zone" between the containment building and the five-way restraints. The MSIV/MFIV areas are located north of the containment building between column lines C_{18} and C_{19} for Unit 1 and C_3 and C_4 for unit 2. The zones subjected to the abnormal loads are bounded by levels 1 and 3 for the MSIV area, and levels A and 1 for the MFIV area.

The MSIV/MFIV area is analyzed using the BSAP computer program, utilizing a finite element model. Conventional modeling techniques are used to model the structural walls and slabs in the MSIV/MFIV area. The loads applied to the model include dead loads, live loads, vertical and horizontal OBE/SSE loads, pressure loads, and thermal loads. Load combination equations 9, 10, and 11 of Appendix B, Table B.2 are considered in determining the design forces.

To ensure that the requirements of the ACI 318 Code are satisfied, the reinforcing steel provided on the basis of overall structural response (per the design methodology described in section 4) is evaluated using OPTCON, for the design forces resulting from the effects of abnormal loads.

5.4 FOUNDATION BEARING PRESSURE

The maximum calculated bearing pressures under the governing design load conditions are provided in table 11.

6.0 CONCLUSION

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

TABLE 1

CONTROL BUILDING SEISMIC ACCELERATION VALUES

Elevation	Floor Accelerations (g's)[1]					
	OBE			SSE		
	Horizontal		Vertical	Horizontal		Vertical
	East-West	North-South		East-West	North-South	
180'-0"	0.15	0.17	0.24	0.26	0.26	0.40
200'-0"	0.17	0.18	0.24	0.28	0.26	0.40
220'-0" (grade level)	0.18	0.19	0.25	0.29	0.27	0.42
240'-0"	0.24	0.33	0.44	0.37	0.49	0.67
260'-0"	0.30	0.40	0.53	0.45	0.58	0.72
280'-0"	0.35	0.53	0.69	0.52	0.73	0.88
301'-0"[2]	0.45	0.70	0.85	0.65	0.90	1.00

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

(2) The acceleration values shown for elevation 301'-0" are used in the design of the structure and are higher than the values obtained from the seismic analysis.

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 OF FLOOR SLABS

Slab Panel Location(1)		Governing Load Combination Equation	Design Force M_u (ft-k)	A_s Required by Design (in.^{2}/ft)	A_s Provided (in.^{2}/ft)
Col Line	Slab Thickness				
C_6-C_8 $C_{A.5}$-$C_{B.6}$	24 in. (roof)	3	59.0	0.71	1.00
C_9-C_{10} C_F-C_G	18 in.	3	35.0	0.60	1.00
C_{11}-C_{12} C_B-C_C	18 in.	3	9.8	0.30	1.00
C_{11}-C_{12} C_D-C_E	18 in.	3	54.0	0.97	1.00
C_3-C_4 South of $C_{C.8}$	36 in.	10	156.0	1.10	2.54

(1) All representative slab panels are for the level 260'-0" slab.

TABLE 4

DESIGN RESULTS OF STRUCTURAL STEEL GIRDERS

Girder Location[1]		Governing Load Combination Equation	Design Forces		Actual Stress		Allow. Stress		Remarks
Col. Line	Size		M (ft-k)	V (k)	f_b (ksi)	f_v (ksi)	F_b (ksi)	F_v (ksi)	
$C_{B.6}$ C_4-C_5	Plate Girder 42" x 550 lb/ft	2	1,169.6	158.0	6.27	2.51	24.0	14.4	Web = 1-1/2" x 36" Flange = 3" x 18"
C_C C_9-C_{13}	Plate Girder 42" x 965 lb/ft	2	7,830.0	680.4	12.00	5.40	24.0	14.4	Web = 1-1/2" x 77" Flange = 3-1/2" x 24"
C_C C_8-C_9	W36X170	2	515.9	84.7	10.67	3.44	24.0	14.4	
C_D C_{10}-C_{11}	W36X300 with 1/2" x 14" top and bottom flange plates	2	2,925.2	357.1	23.50	10.02	24.0	14.4	
C_B C_{10}-C_{11}	W36X300 with 1/2" x 14" top and bottom flange plates	2	2,073.6	270.2	22.42	7.58	24.0	14.4	

(1) All representative girders shown are for level 260'-0".

TABLE 5

DESIGN RESULTS OF SHEAR WALLS

Wall Designator			Governing Load Combination Equation	Design Forces(1)			A_s Required(2)			A_s Provided(2)	
Col. Line	Floor Elev.	Wall Thick.		V_u	N_u	M_u	M_o	Horiz.	Vert.	Horiz.	Vert.
C_A $C_1 - C_{21}$	180'	48"	3	12,400	1,428	1,971,837	381	1.44(3)	4.56	1.58	4.59
C_1 $C_A - C_{F.2}$	180'	66"	3	27,675	2,575	976,871	504	1.98(3)	6.62	6.24	8.00
East of C_4 $C_{A.5} - C_{B.6}$	200'	24"	3	1,208	163	24,160	5.5	0.72(3)	0.75	0.88	0.88
East of C_{18} $C_{B.5} - C_{C.4}$	180'	24"	3	1,752	2,445	96,902	4.6	0.72(3)	0.74	0.88	0.88(4)
C_G $C_7 - C_{15}$	180'	24"	3	12,648	2,311	1,320,447	4.6	0.72(3)	0.70	1.20	1.20

(1) V_u = In-plane shear force (kips)
N_u = Axial force (kips). . . . (-) tension
M_u = In-plane overturning moment (ft-kips)
M_o = Out-of-plane bending moment (ft-kips)

(2) A_s required and A_s provided are total reinforcement (in.^{2}/ft of wall) for both faces of the wall.

(3) Governed by minimum Code reinforcement requirements.

(4) Additional bars added as end reinforcement.

TABLE 6

BASEMAT ANALYSIS RESULTS; ELEMENTS WITH MAXIMUM MOMENT DUE TO DEAD LOAD

Element No.	Zone	Direction	Dead Load(1)		Vertical Seismic(1)		N-S Seismic(1)		E-W Seismic(1)	
			Membrane	Moment	Membrane	Moment	Membrane	Moment	Membrane	Moment
25	1	East/West	-4	-327	-1	-105	5	49	10	-3
43	1	East/West	-5	-339	-2	-109	5	56	16	-6
140	1	East/West	12	285	4	92	14	144	10	30
157	1	East/West	8	321	2	103	2	177	16	42
220	2	East/West	-6	-246	-2	-79	-86	-9	60	-33
345	3	East/West	-11	-340	-4	-110	-57	-266	-32	-359
918	3	East/West	6	-437	2	-141	-129	-336	-38	-430
927	2	East/West	-2	-210	-1	-68	-4	54	1	-21
23	1	North/South	-13	-283	-4	-91	16	182	37	5
26	1	North/South	-28	-252	-9	-81	10	31	45	5
50	1	North/South	-23	-265	-7	-85	-24	-347	25	1
59	1	North/South	-16	-313	-5	-101	10	201	15	4
196	2	North/South	-47	-470	-15	-151	-6	-926	-44	-10
200	2	North/South	-14	-218	-5	-70	12	184	2	3
920	3	North/South	8	-223	3	-72	-29	-570	24	-127
925	2	North/South	-42	-586	-14	-189	-5	-892	-38	-21

(1) Sign conventions:

Membrane forces (kips) . . . . (+) Tension . . . . (-) Compression
Moments (ft-kips). . . . . . . (+) Tension on bottom of basemat . . (-) Compression on bottom of basemat

TABLE 7

BASEMAT ANALYSIS RESULTS; ELEMENTS WITH MAXIMUM MOMENT DUE TO N-S SEISMIC RESPONSE

Element No.	Zone	Direction	Dead Load[1]		Vertical Seismic[1]		N-S Seismic[1]		E-W Seismic[1]	
			Membrane	Moment	Membrane	Moment	Membrane	Moment	Membrane	Moment
68	1	East/West	-6	-181	-2	-58	-21	-223	9	-9
159	1	East/West	4	309	1	100	17	221	1	40
260	4	East/West	-1	157	0	50	-104	168	-5	-10
322	3	East/West	-5	-349	-2	-113	-105	-236	-43	-357
334	3	East/West	12	-372	4	-120	-91	-372	-14	-423
344	3	East/West	-29	-327	-9	-105	-39	-192	-42	-326
347	3	East/West	14	-313	5	-101	-56	-283	7	-359
377	3	East/West	3	-45	1	-15	2	225	4	-126
3	1	North/ South	16	-59	5	-19	4	354	41	-9
69	1	North/ South	-11	-181	-4	-58	-15	-400	14	0
106	1	North/ South	-6	-84	-2	-27	-21	-360	6	-1
196	2	North/ South	-47	-470	-15	-151	-6	-926	-44	-10
262	2	North/ South	1	-87	0	-28	9	261	0	12
292	1	North/ South	1	-84	0	-27	11	257	-1	9
347	3	North/ South	8	-179	3	-58	-31	-485	14	-96
375	3	North/ South	5	-175	-2	-57	-9	-515	0	-99

(1) Sign conventions:

Membrane forces (kips) . . . . (+) Tension . . . . (-) Compression
Moments (ft-kips). . . . . . . (+) Tension on bottom on basemat . . (-) Compression on bottom of basemat

TABLE 8

BASEMAT ANALYSIS RESULTS; ELEMENTS WITH MAXIMUM MOMENT DUE TO E-W SEISMIC RESPONSE

Element No.	Zone	Direction	Dead Load(1)		Vertical Seismic(1)		N-S Seismic(1)		E-W Seismic(1)	
			Membrane	Moment	Membrane	Moment	Membrane	Moment	Membrane	Moment
332	3	East/West	-27	-330	-9	-106	-53	-183	-45	-324
333	3	East/West	-10	-354	-3	-114	-72	-252	-36	-367
334	3	East/West	12	-372	4	-120	-91	-372	-14	-423
346	3	East/West	1	-325	0	-105	-50	-263	-17	-362
347	3	East/West	14	-313	5	-101	-56	-283	7	-359
359	3	East/West	0	-271	0	-88	-41	-263	-14	-329
364	1	East/West	-8	-159	-3	-51	25	65	-13	-128
918	3	East/West	6	-437	2	-141	-129	-336	-38	-430
334	3	North/ South	8	-63	3	-20	-35	-285	4	-44
347	3	North/ South	8	-179	3	-58	-31	-485	14	-96
359	3	North/ South	-6	-111	-2	-36	-4	-370	-2	-77
361	3	North/ South	0	-138	0	-45	-14	-506	10	-99
373	3	North/ South	-15	-117	-5	-38	8	-364	-12	-84
374	3	North/ South	-10	-166	-3	-54	-2	-469	-6	-99
920	3	North/ South	8	-223	3	-72	-29	-570	24	-127
921	3	North/ South	6	-246	2	-79	-19	-622	24	-136

(1) Sign conventions:

Membrane forces (kips) . . . . (+) Tension . . . . (-) Compression
Moments (ft-kips). . . . . . . (+) Tension on bottom of basemat . . (-) Compression on bottom of basemat

TABLE 9

FACTORS OF SAFETY FOR STRUCTURAL STABILITY

Load[1][3] Combination	Overturning Factor of Safety			Sliding Factor of Safety		Flotation Factor of Safety	
	Minimum Required	Calculated: Equivalent Static	Calculated: Energy Balance	Minimum Required	Calculated	Minimum Required	Calculated
D + H + E	1.5	See Note (2)	See Note (2)	1.5	1.8	-	-
D + H + E'	1.1	1.6	1710	1.1	1.3	-	-
D + F'	-	-	-	-	-	1.1	24

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

(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 10

TORNADO MISSILE ANALYSIS RESULTS[1]

Panel Description and Location	Panel Size Length (ft)	Panel Size Width (ft)	Panel Size Thickness (ft)	Computed Ductility	Allowable Ductility
Exterior Wall Level 4; C_8-C_9 and C_C-C_D	7.7	9.0	2.0	10.0	10.0
Exterior Wall Level 2; C_4 and $C_{A.5}$ - $C_{B.6}$	24.3	18.0	3.0	See Note (2)	10.0
Exterior Wall Level 3; C_8 and C_E-C_F	14.0	18.0	4.0	1.2	10.0
Roof slab elevation 301'-0" C_{13}-C_{14} and C_E-C_F	24.5	22.0	2.0	1.5	10.0
Roof slab Level 4; C_{13}-C_{14} and $C_{A.6}$-C_B	14.5	11.8	1.8	1.2	10.0

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

(2) Remains elastic

TABLE 11

MAXIMUM FOUNDATION BEARING PRESSURES(1)

Gross Static (ksf)	Net Static (ksf)	Gross Dynamic (ksf)	Net Dynamic (ksf)	Allowable Net(2) Bearing Capacity		Computed(3) Factor of Safety	
				Static (ksf)	Dynamic (ksf)	Static	Dynamic
4.3	-1.3	13.4	7.8	19.3	28.9	-(4)	7.4

Note:

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

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

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

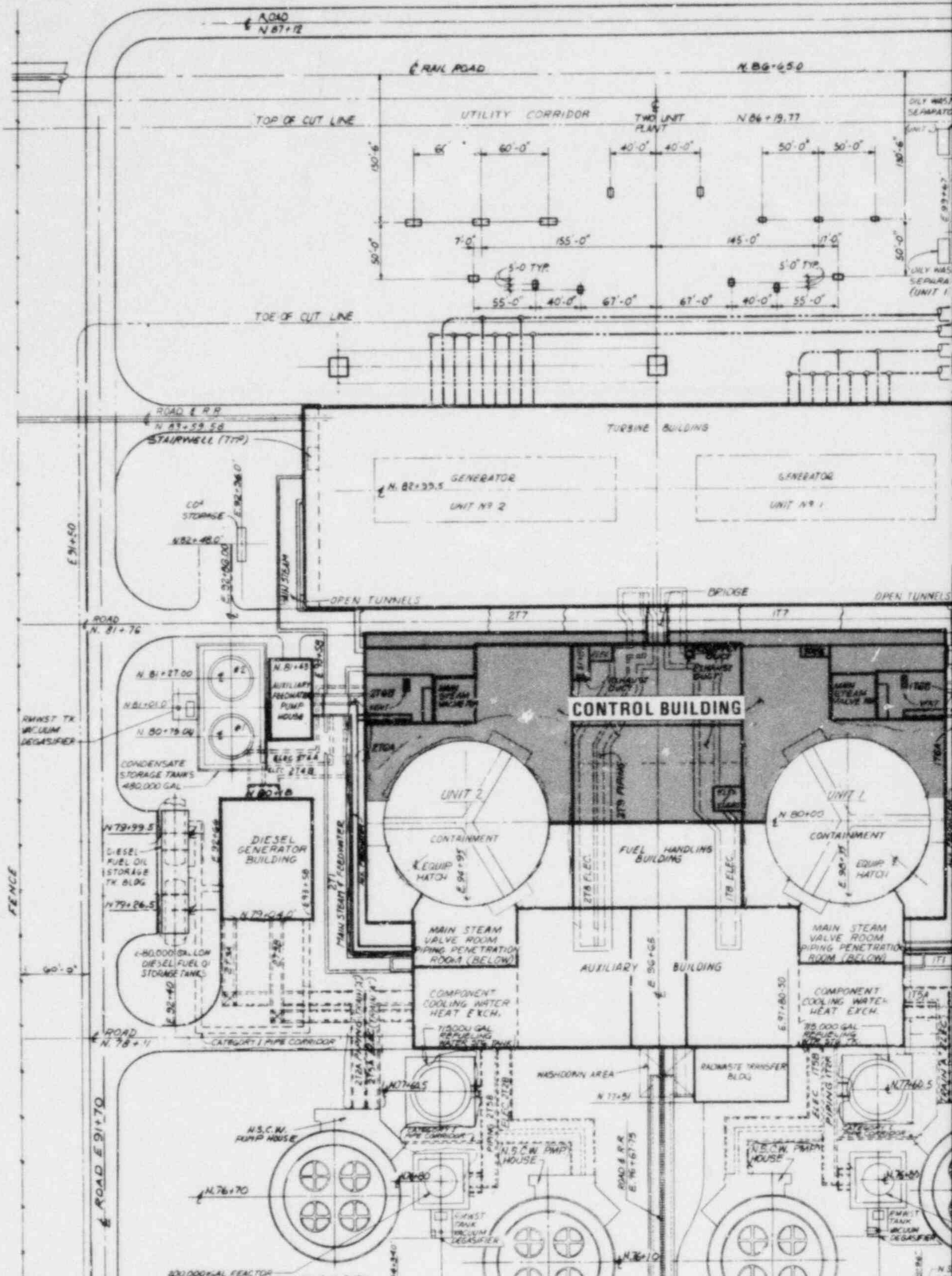
Gross Dynamic = Maximum soil pressure under dynamic loading conditions (i.e. unfactored SSE).

Net Dynamic = The dynamic pressure in excess of the overburden pressure at the base of the structure.

(2) The allowable net static and dynamic bearing capacities are obtained by dividing the ultimate net bearing capacity by factors of 3 and 2 respectively. The ultimate net bearing capacity is the pressure in excess of the overburden pressure at the foundation level at which shear failure may occur in the foundation stratum.

(3) The computed factor of safety is the ultimate net bearing capacity divided by the net static or net dynamic bearing pressure.

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



ROAD N 87+12
RAIL ROAD
N 86+650
TOP OF CUT LINE
UTILITY CORRIDOR
TWO UNIT PLANT
N 84+19.77
TOE OF CUT LINE
ROAD & R.R. N 83+59.58
STAIRWELL (TYP)
TURBINE BUILDING
GENERATOR UNIT No 2
GENERATOR UNIT No 1
N 82+33.5
CO2 STORAGE
OPEN TUNNELS
BRIDGE
ROAD N 81+76
CONTROL BUILDING
RMWST TK VACUUM DEGASIFIER
CONDENSATE STORAGE TANKS 480,000 GAL
AUXILIARY FEEDWATER PUMP HOUSE
DIESEL FUEL OIL STORAGE TK BLDG
DIESEL GENERATOR BUILDING
UNIT 2
UNIT 1
CONTAINMENT
EQUIP HATCH
N 80+00
FUEL HANDLING BUILDING
MAIN STEAM VALVE ROOM PIPING PENETRATION ROOM (BELOW)
AUXILIARY BUILDING
COMPONENT COOLING WATER HEAT EXCH.
FENCE
ROAD N 78+11
CATEGORY I PIPE CORRIDOR
WASHDOWN AREA
RADWASTE TRANSFER BLDG
N.S.C.W. PUMP HOUSE
ROAD E 91+70
N 76+70
N.S.C.W. CHEM CONTROL BLDG
N.S.C.W. SULFURIC ACID DAY TANK 2-1413-T4-001
NUCLEAR SERVICE COOLING WTR. TOWER & BASIN TRAIN "A"
NUCLEAR SERVICE COOLING WTR. TOWER & BASIN TRAIN "B"
ROAD N 74+88
RADWASTE TRANSFER TUNNEL

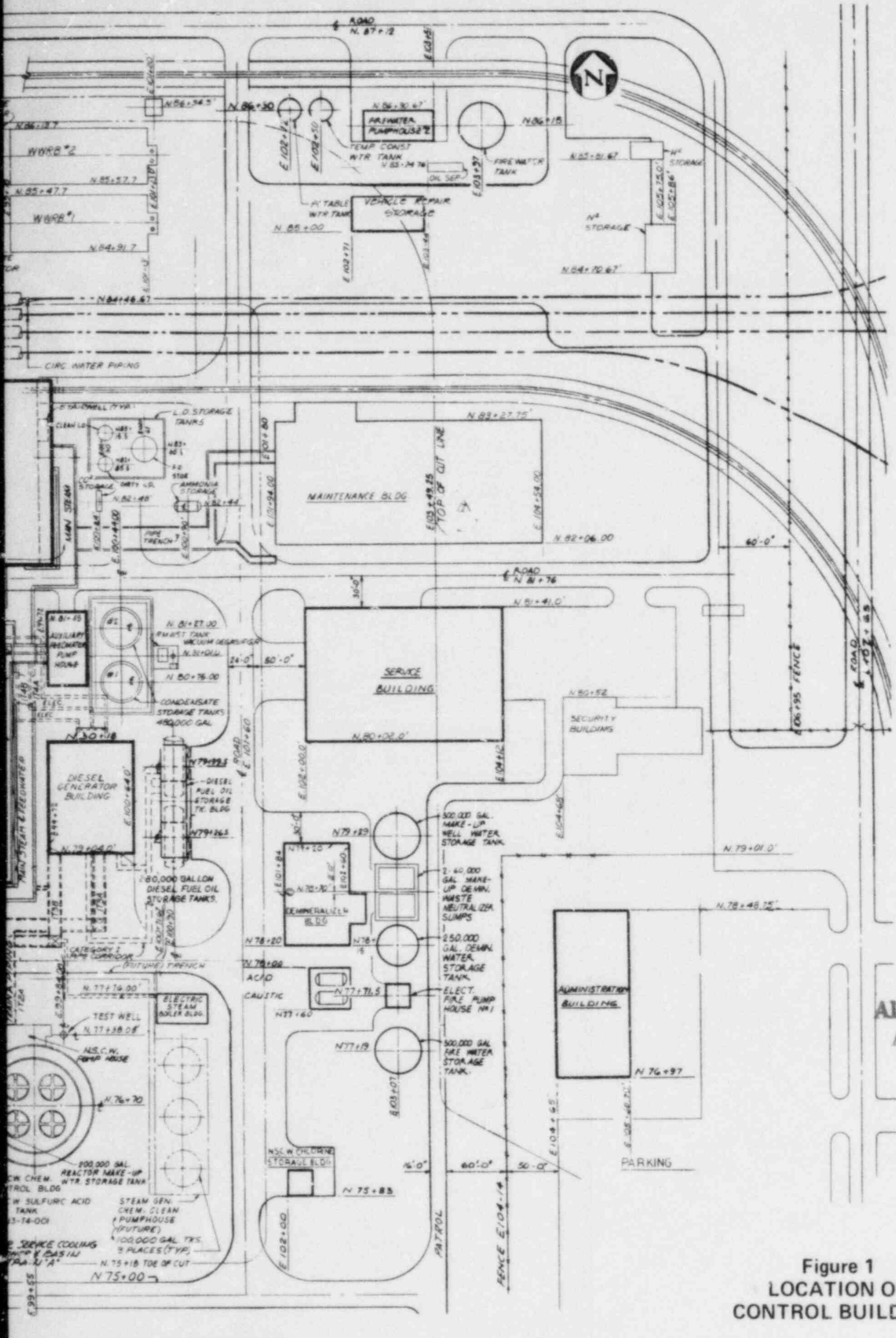
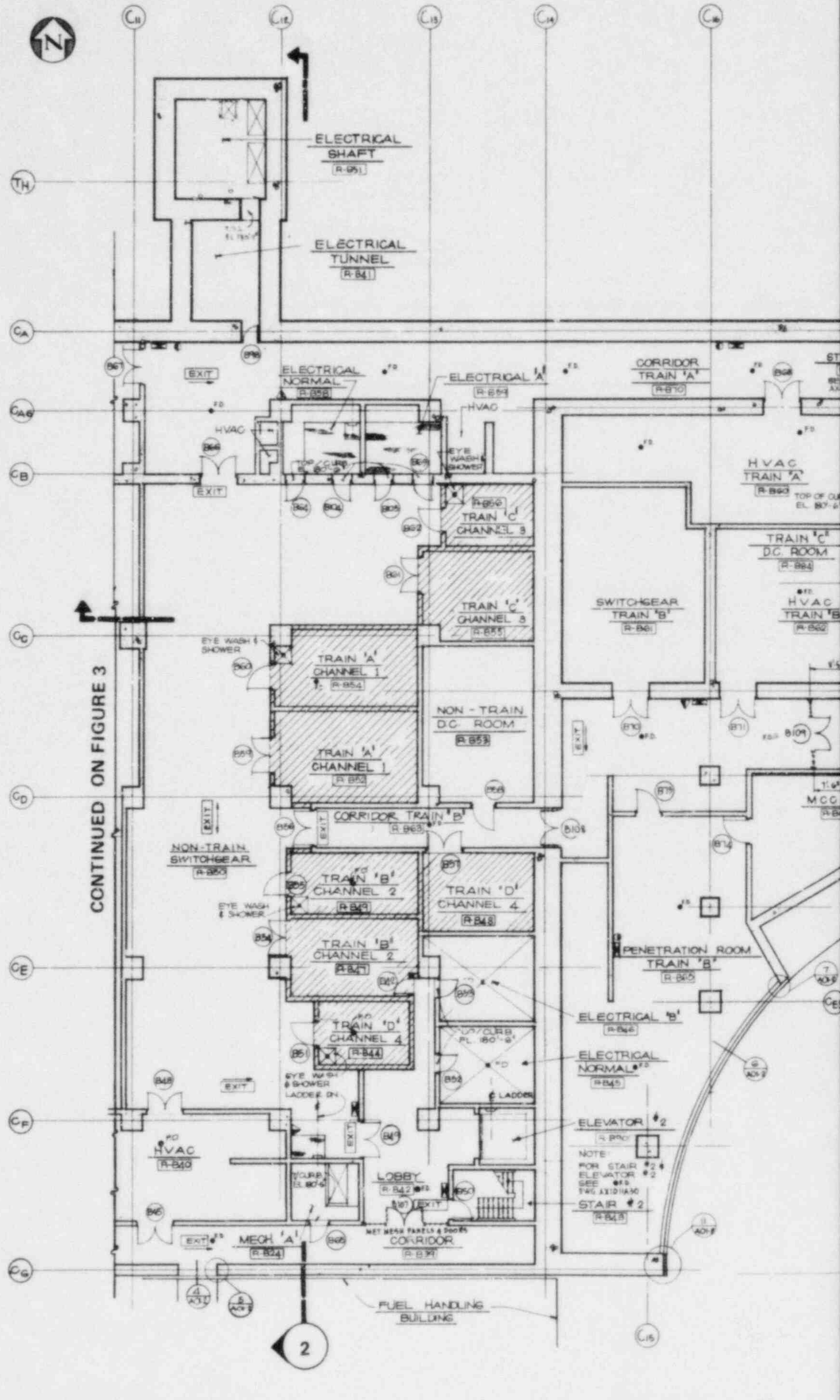
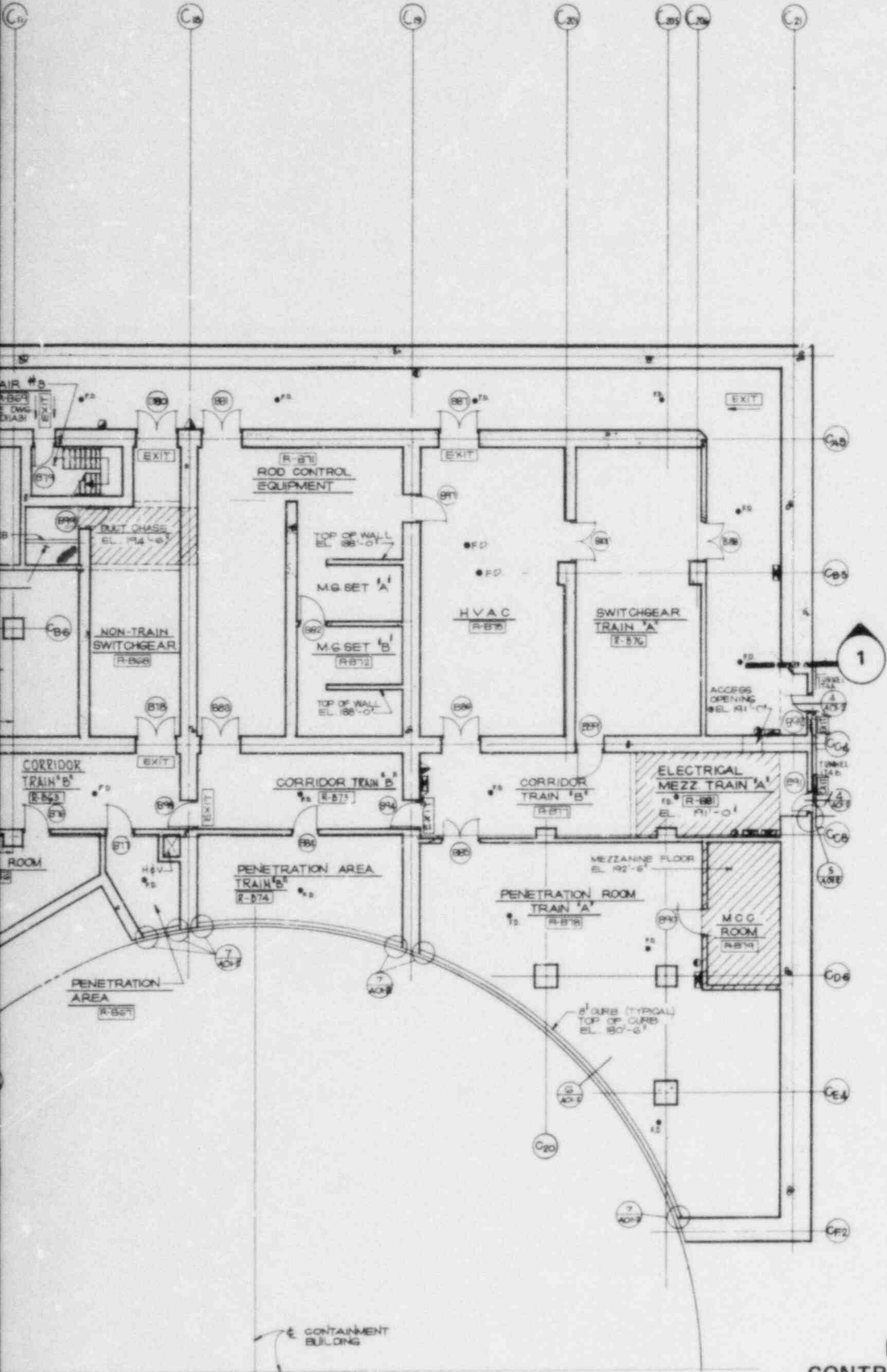


Figure 1
LOCATION OF
CONTROL BUILDING



ELECTRICAL SHAFT
R-851
ELECTRICAL TUNNEL
R-841
ELECTRICAL NORMAL
R-858
ELECTRICAL 'A'
R-859
CORRIDOR TRAIN 'A'
R-810
HVAC
EXIT
HVAC TRAIN 'A'
R-860
TRAIN 'C' CHANNEL 3
TRAIN 'C' D.C. ROOM
R-884
TRAIN 'C' CHANNEL 3
R-855
SWITCHGEAR TRAIN 'B'
R-861
HVAC TRAIN 'B'
R-862
EYE WASH & SHOWER
TRAIN 'A' CHANNEL 1
R-854
TRAIN 'A' CHANNEL 1
R-852
NON - TRAIN D.C. ROOM
R-853
CONTINUED ON FIGURE 3
CORRIDOR TRAIN 'B'
R-863
NON-TRAIN SWITCHGEAR
R-850
TRAIN 'B' CHANNEL 2
R-849
TRAIN 'D' CHANNEL 4
R-848
TRAIN 'B' CHANNEL 2
R-847
PENETRATION ROOM TRAIN 'B'
R-865
TRAIN 'D' CHANNEL 4
R-844
ELECTRICAL 'B'
R-846
ELECTRICAL NORMAL
R-845
ELEVATOR #2
HVAC
R-840
LOBBY
R-842
STAIR #2
R-843
MECH 'A'
R-824
CORRIDOR
R-839
FUEL HANDLING BUILDING
2



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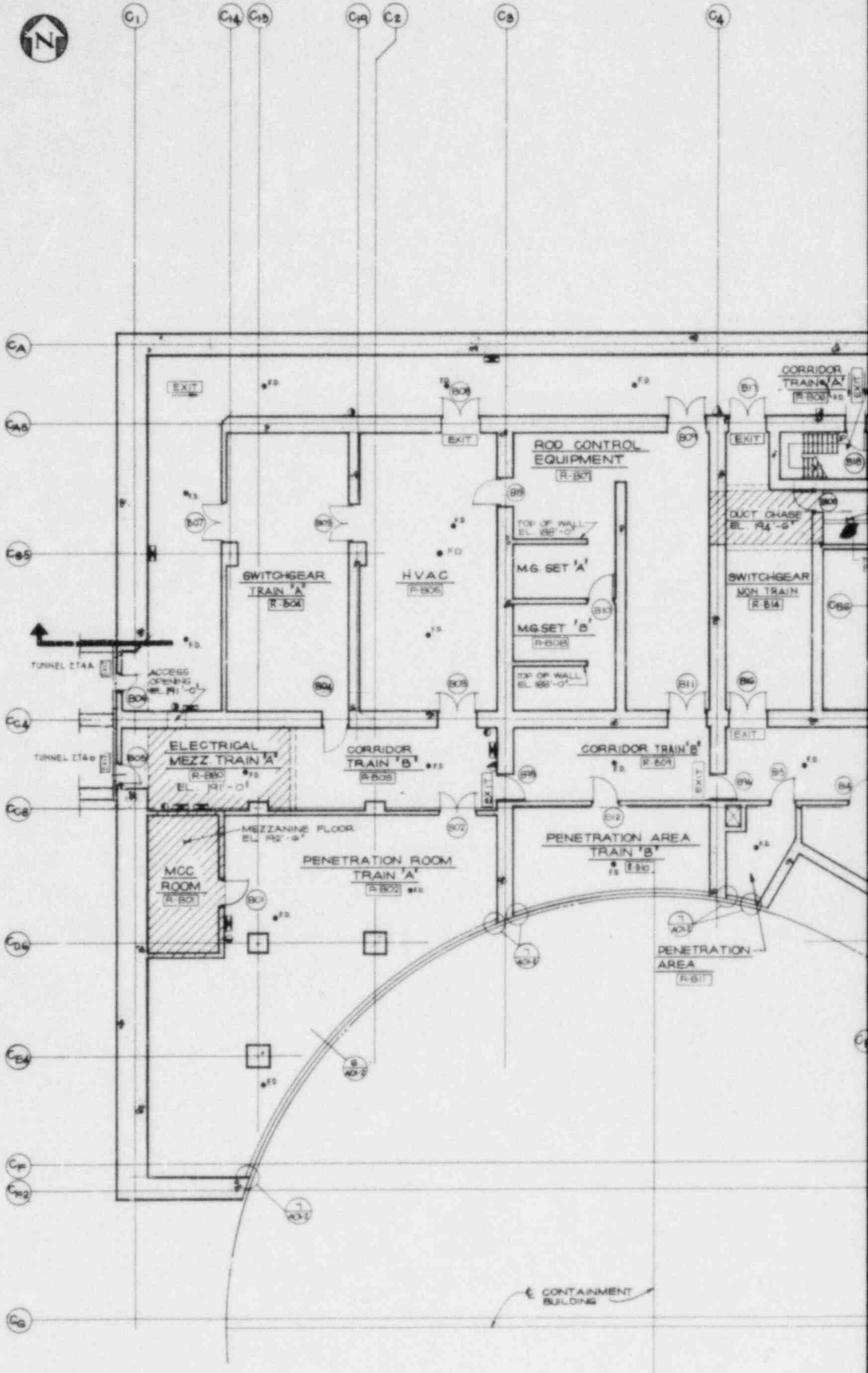
APERTURE
CARD

Also Available On
Aperture Card

Figure 2
CONTROL BUILDING
FLOOR PLAN EL. 180'-0"
LEVEL B, UNIT 1

8411050179 -02



N
C1
C14
C15
C19
C2
C3
C4
CA
CA5
CB5
CC4
CC8
CD6
CE4
CF
CF2
CG
EXIT
CORRIDOR TRAIN 'A'
ROD CONTROL EQUIPMENT
R-B07
TOP OF WALL EL. 188'-0"
M.G. SET 'A'
M.G. SET 'B'
R-B08
DUCT CHASE EL. 194'-6"
SWITCHGEAR TRAIN 'A'
R-B06
HVAC
R-B05
SWITCHGEAR NON TRAIN
R-B14
TUNNEL ET4A
ACCESS OPENING EL. 191'-0"
TUNNEL ET4B
ELECTRICAL MEZZ TRAIN 'A'
EL. 191'-0"
CORRIDOR TRAIN 'B'
R-B03
CORRIDOR TRAIN 'B'
R-B09
MEZZANINE FLOOR EL. 192'-0"
MCC ROOM
R-B01
PENETRATION ROOM TRAIN 'A'
R-B02
PENETRATION AREA TRAIN 'B'
R-B10
PENETRATION AREA
R-B11
℄ CONTAINMENT BUILDING

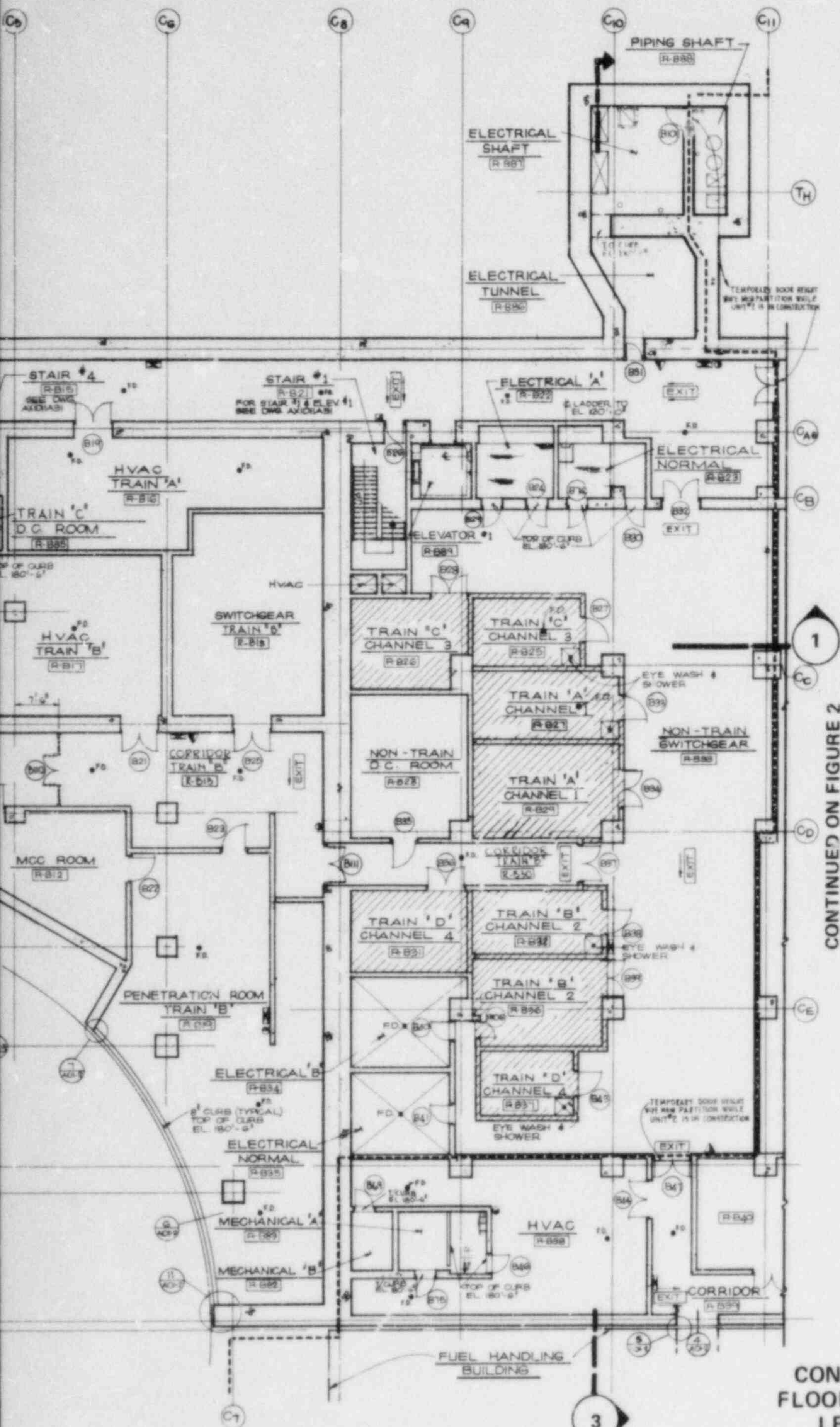


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Figure 3
CONTROL BUILDING
FLOOR PLAN EL. 180'-0"
LEVEL B, UNIT 2

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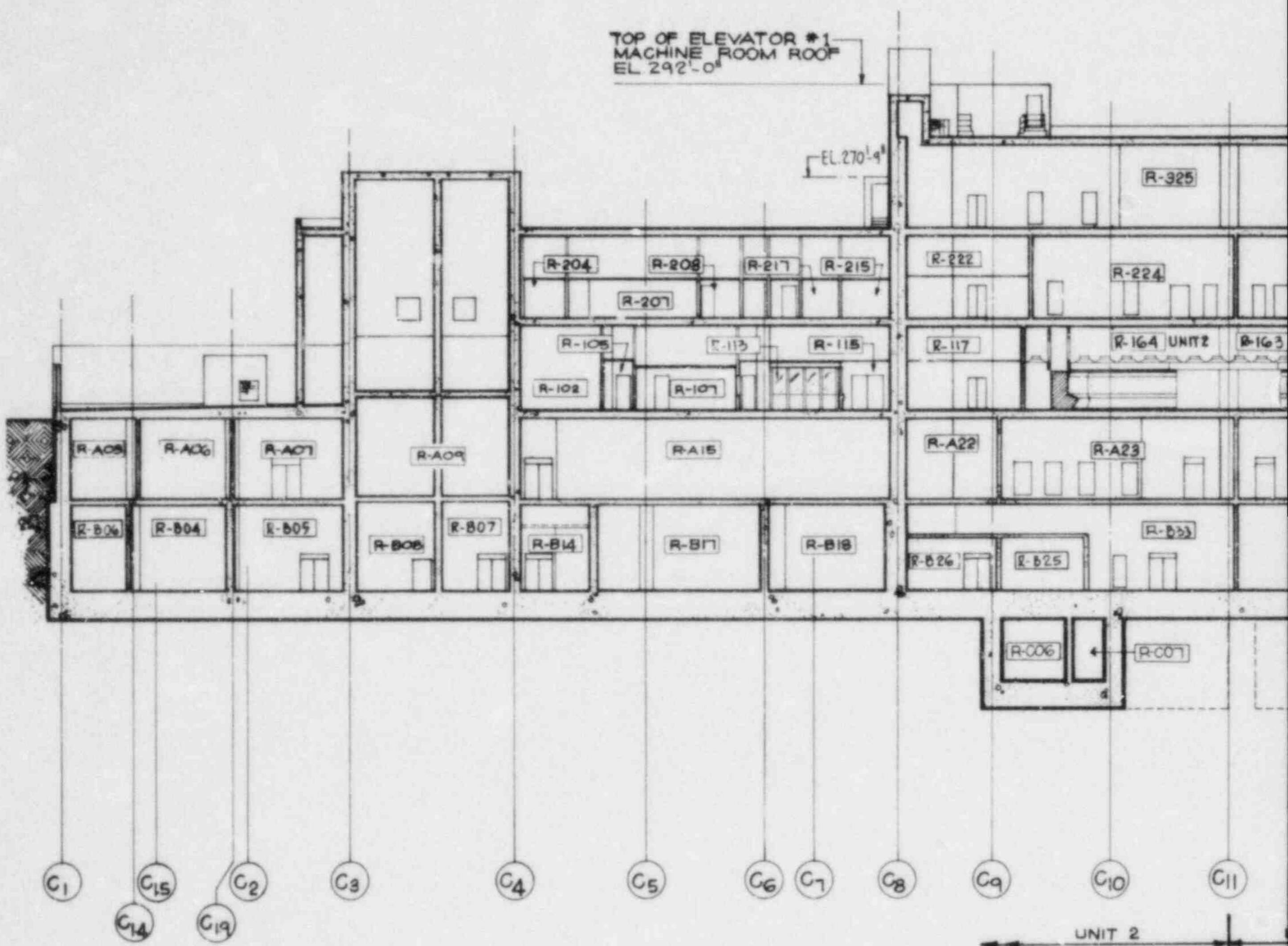
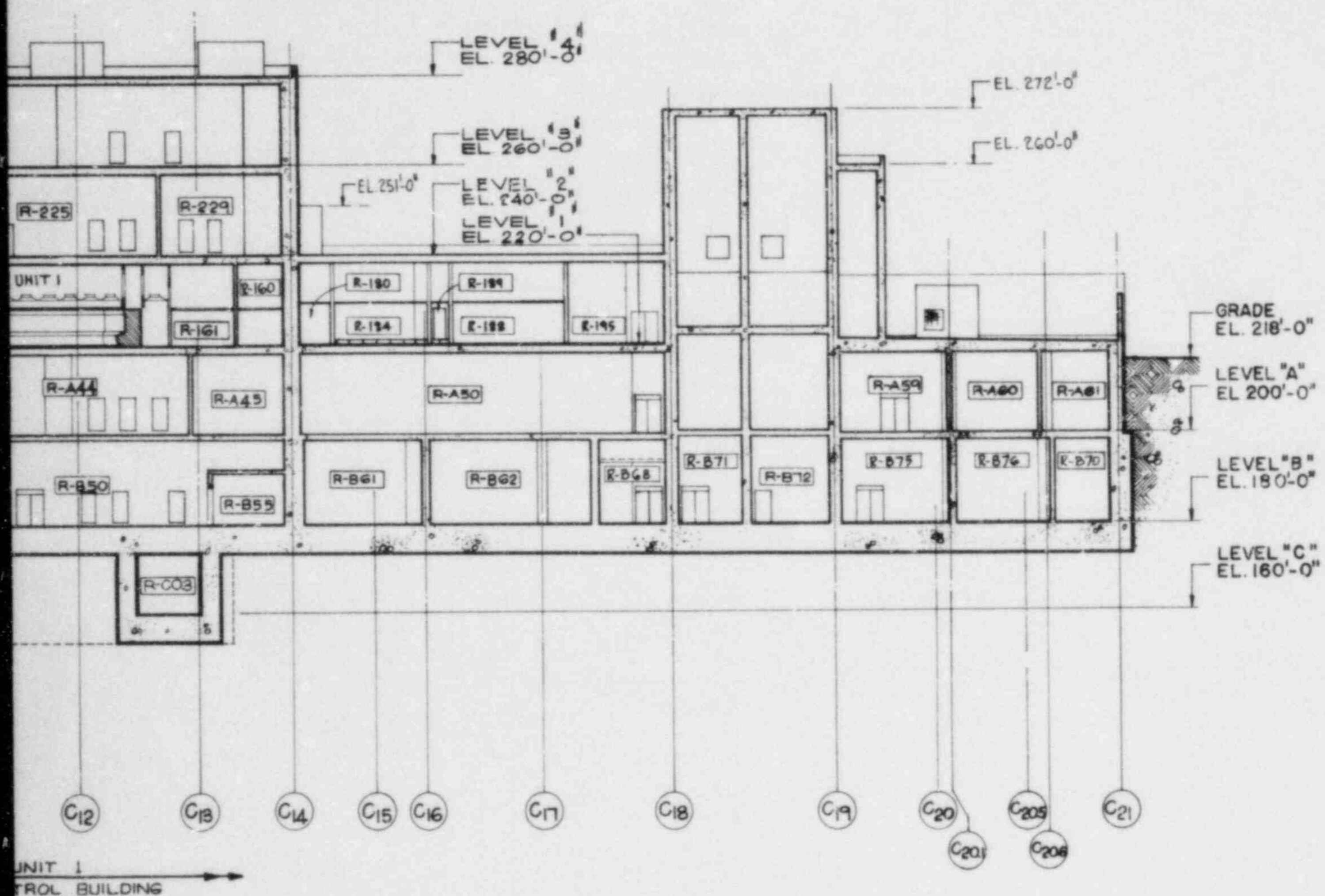
CARD

8411050179-03



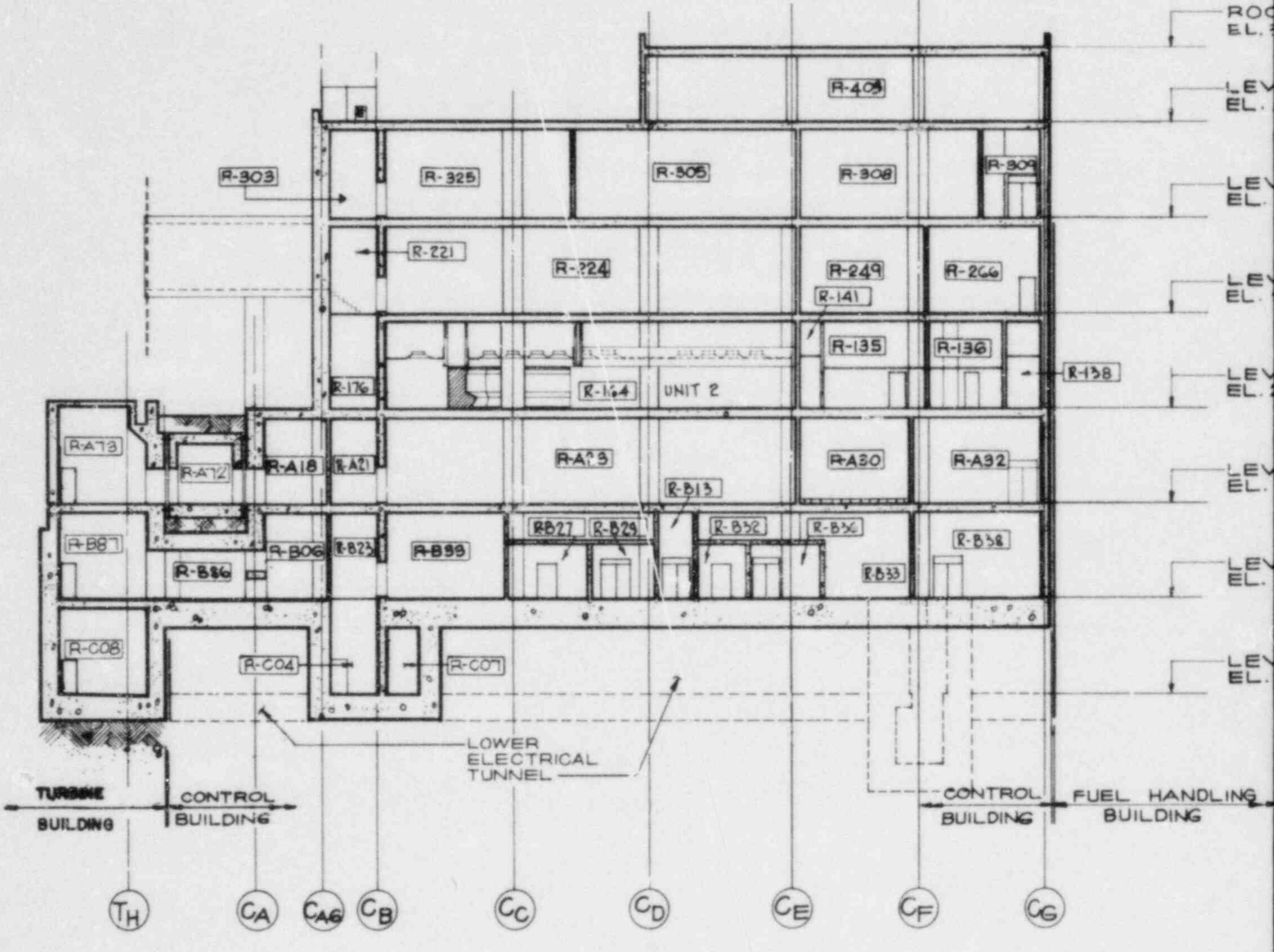
TOP OF ELEVATOR #1
MACHINE ROOM ROOF
EL 292'-0"
EL 270'-9"
R-325
R-204
R-208
R-217
R-215
R-207
R-222
R-224
R-105
R-113
R-115
R-102
R-107
R-117
R-164 UNIT 2
R-163
R-A05
R-A06
R-A07
R-A09
R-A15
R-A22
R-A23
R-B06
R-B04
R-B05
R-B08
R-B07
R-B14
R-B17
R-B18
R-B26
R-B25
R-B33
R-C06
R-C07
C1
C15
C14
C2
C19
C3
C4
C5
C6
C7
C8
C9
C10
C11
UNIT 2
CONTROL BUILDING
CON

SECTION



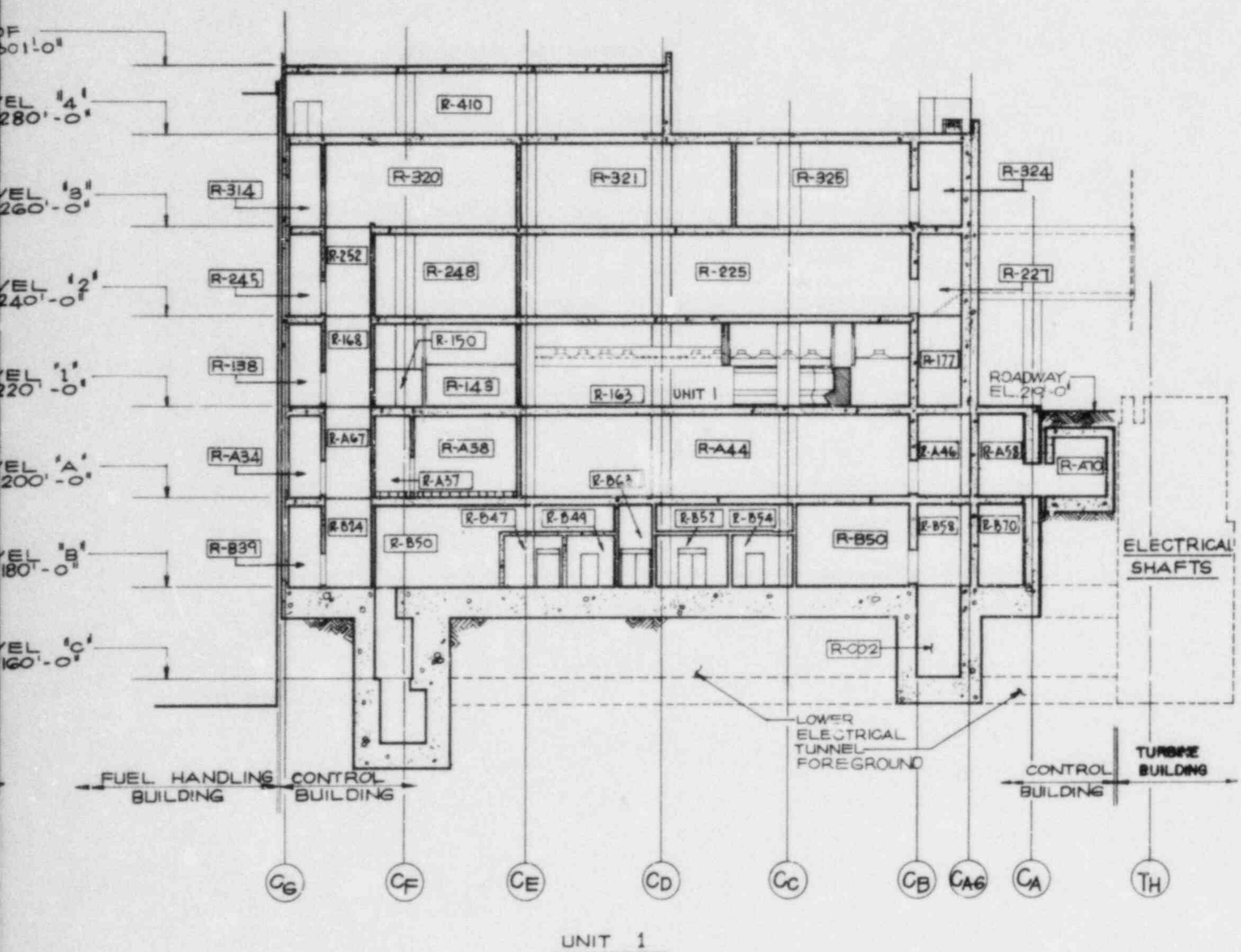
Also Available On
Aperture Card

Figure 4
CONTROL BUILDING
SECTION LOOKING NORTH



UNIT 2

SECTION 3



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Aperture Card

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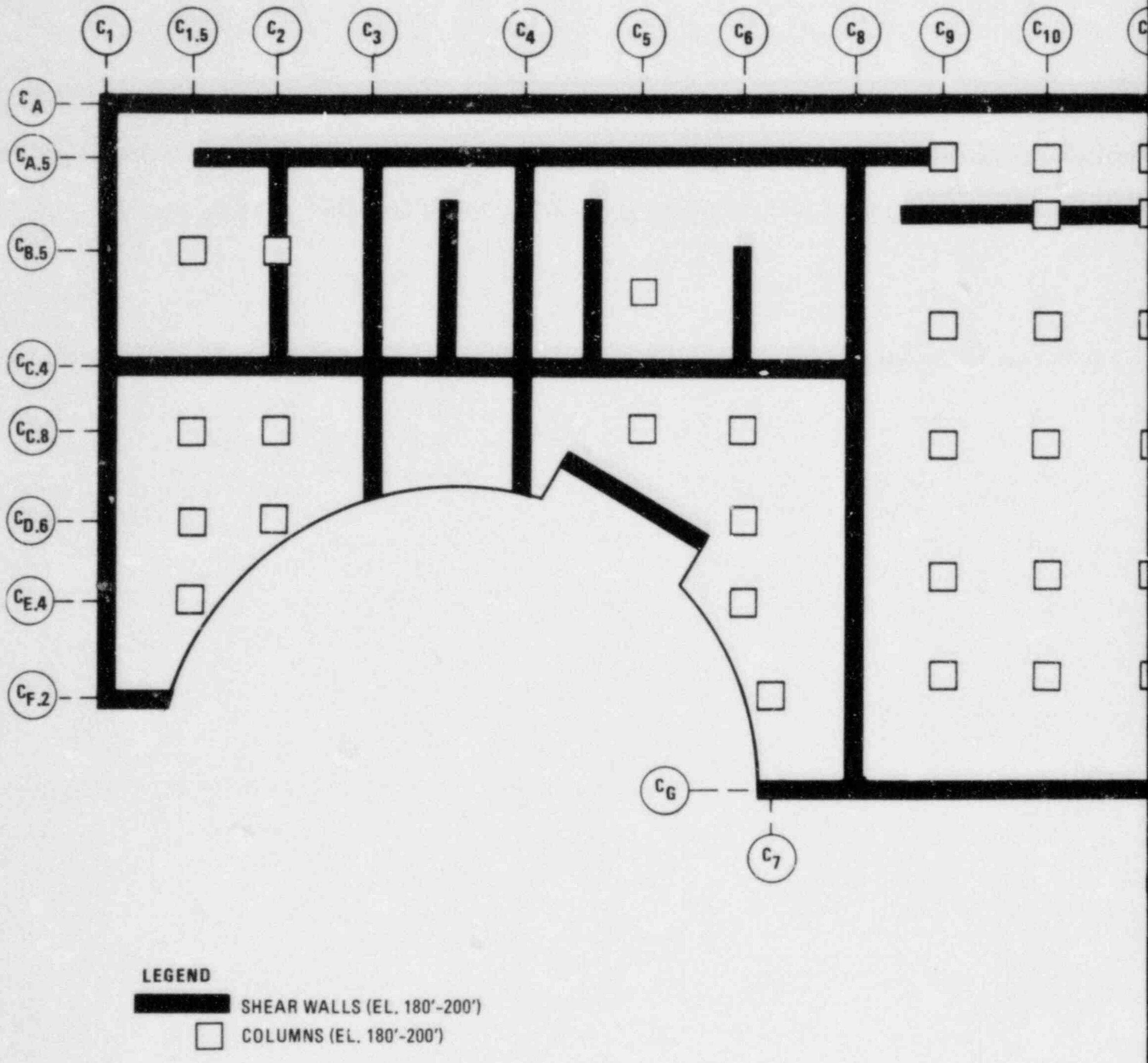
APERTURE
CARD

Figure 5
CONTROL BUILDING
SECTIONS LOOKING EAST AND WEST

8411050179-05



C1
C1.5
C2
C3
C4
C5
C6
C8
C9
C10
CA
CA.5
CB.5
CC.4
CC.8
CD.6
CE.4
CF.2
CG
C7
LEGEND
SHEAR WALLS (EL. 180'-200')
COLUMNS (EL. 180'-200')

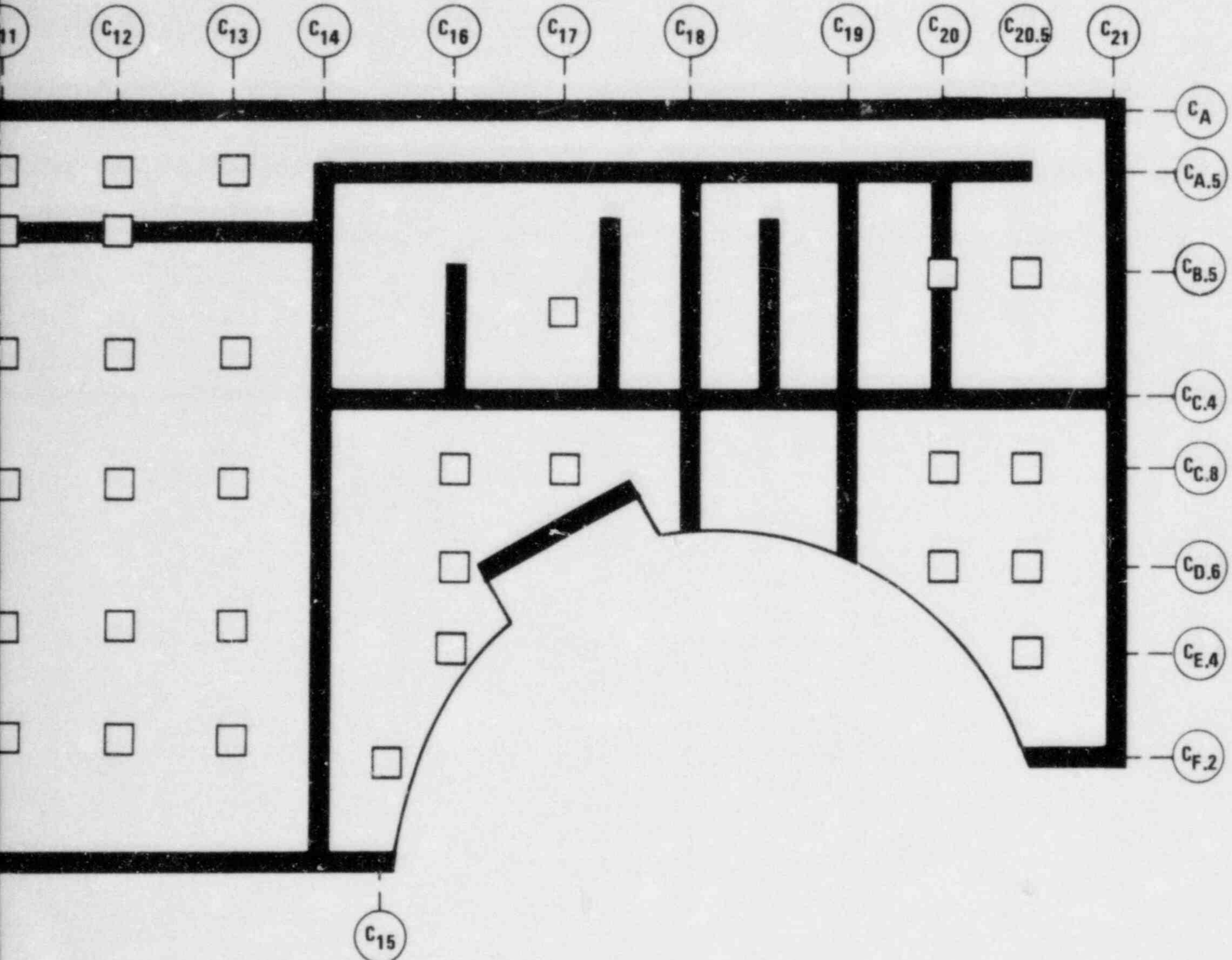
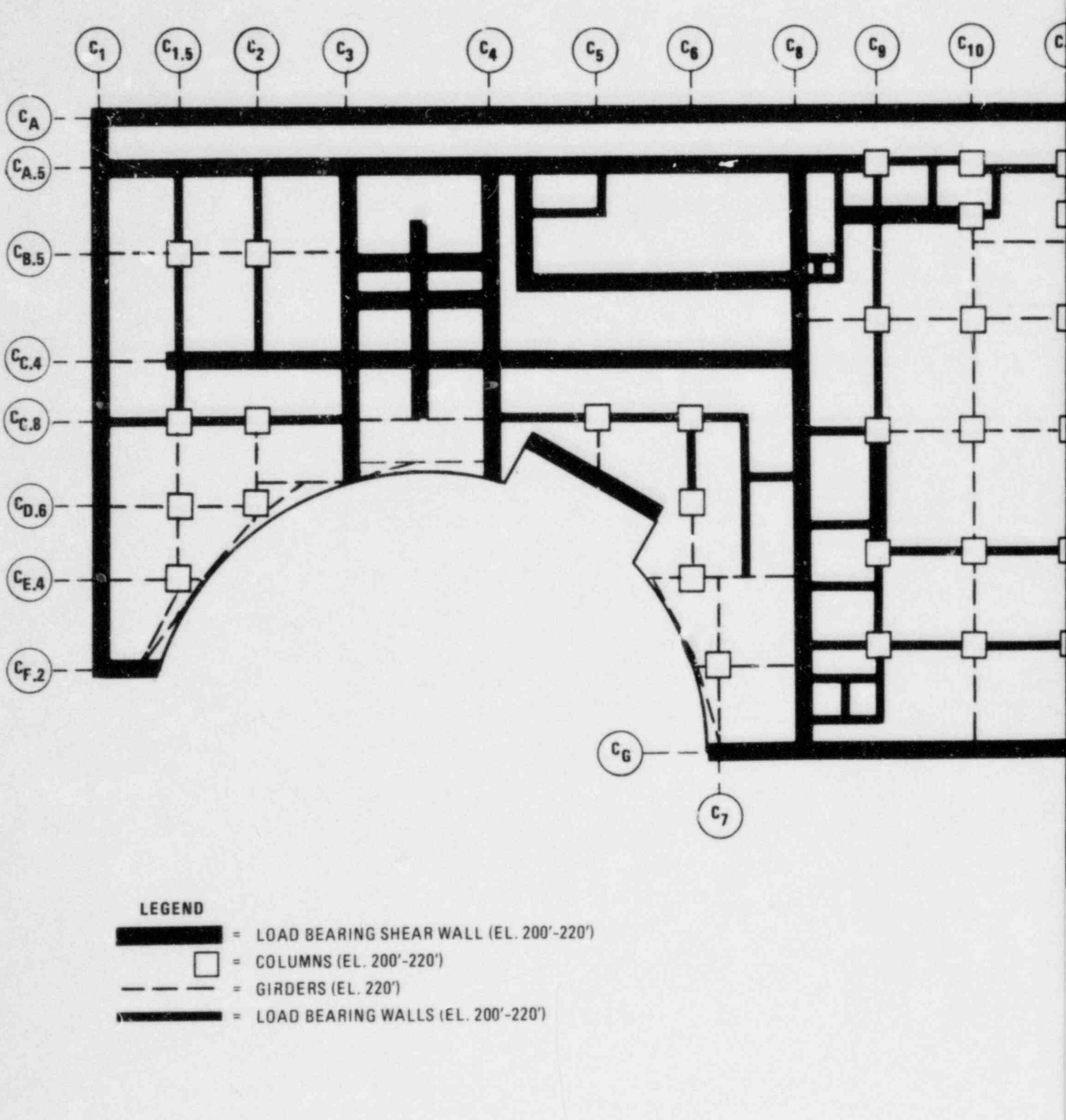


Figure 6
EL. 180'-0", LEVEL B LOCATION OF
KEY STRUCTURAL ELEMENTS



C1
C1.5
C2
C3
C4
C5
C6
C8
C9
C10
CA
CA.5
CB.5
CC.4
CC.8
CD.6
CE.4
CF.2
CG
C7
LEGEND
= LOAD BEARING SHEAR WALL (EL. 200'-220')
= COLUMNS (EL. 200'-220')
= GIRDERS (EL. 220')
= LOAD BEARING WALLS (EL. 200'-220')

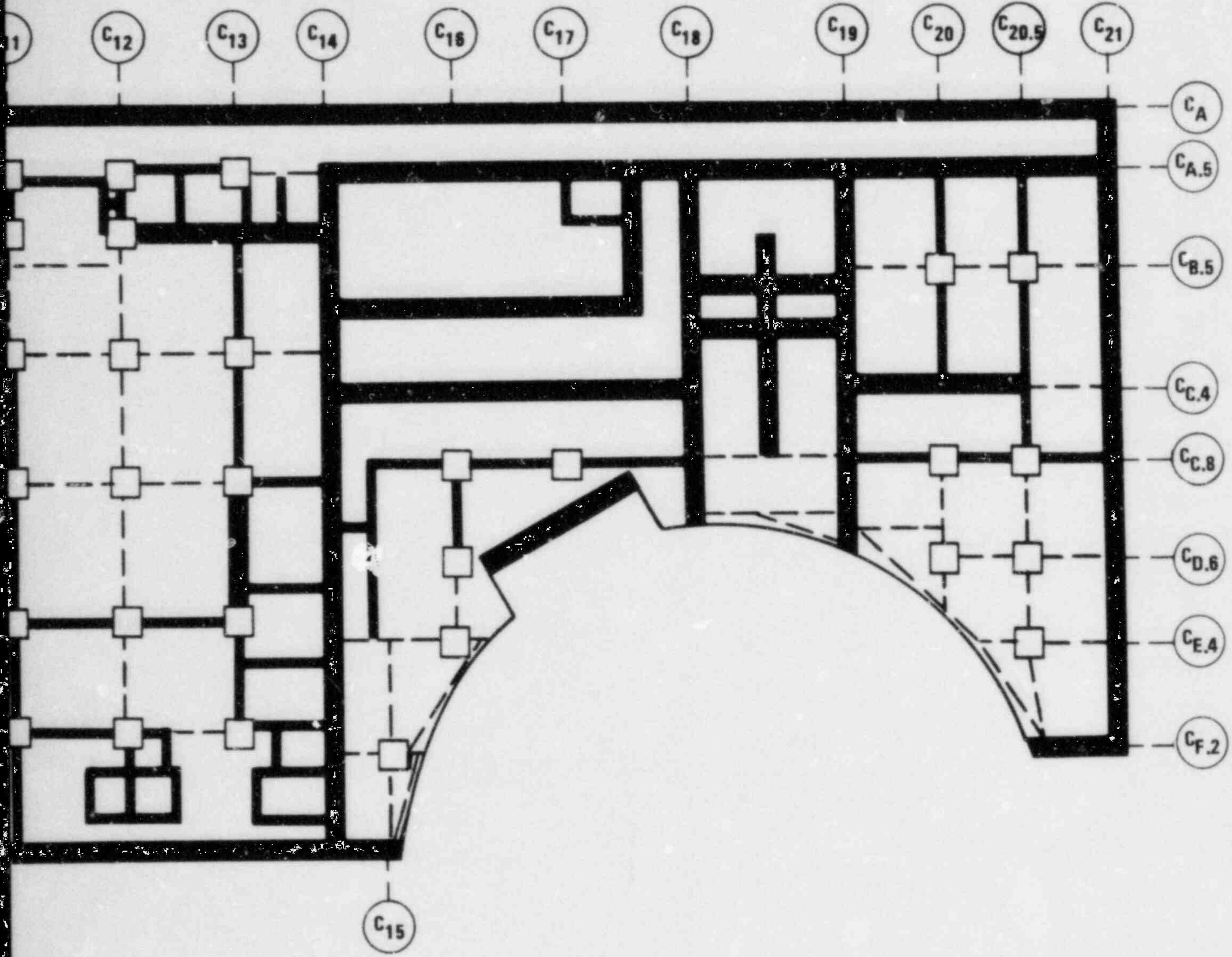


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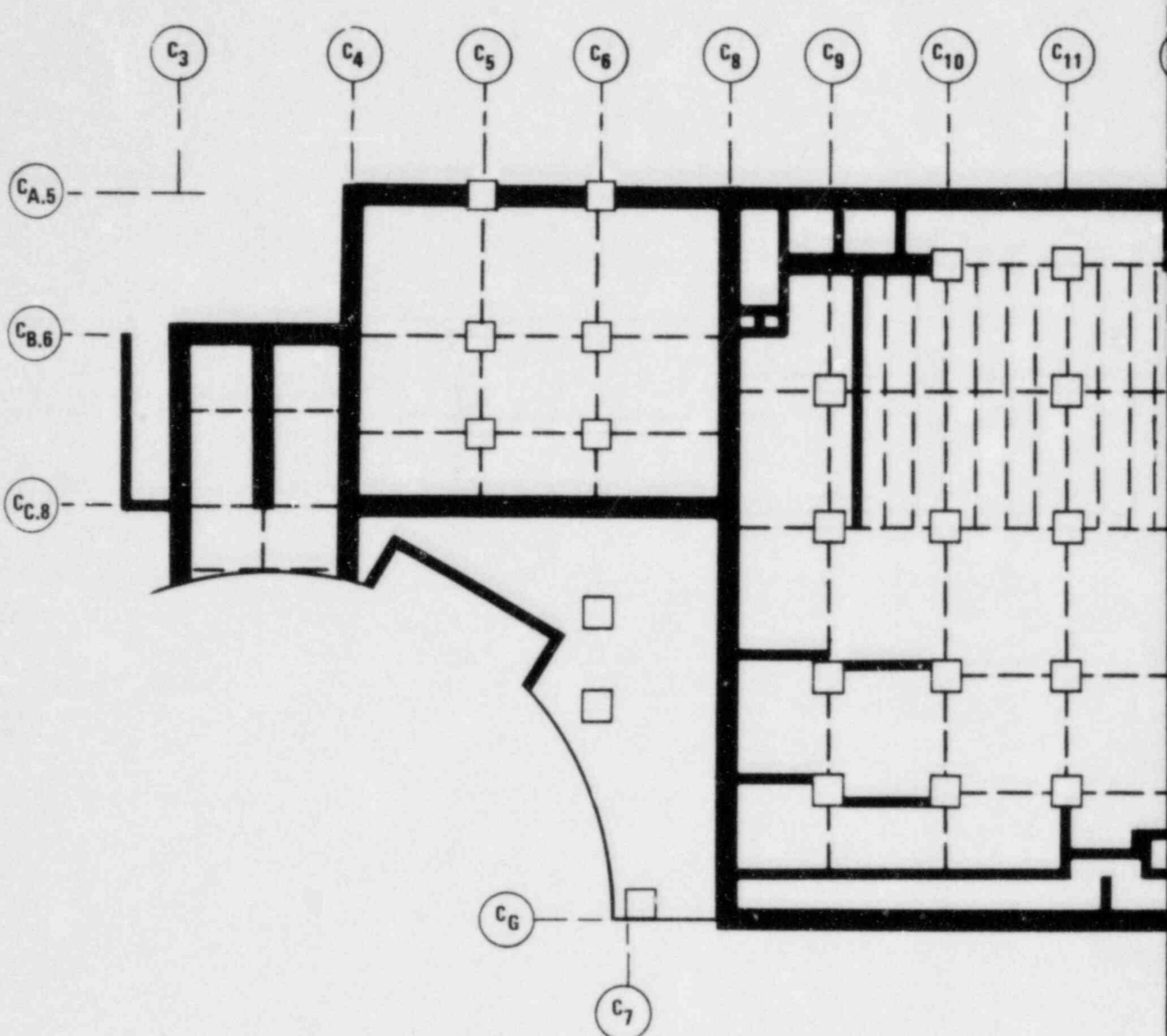
Figure 7
EL. 220'-0", LEVEL 1 LOCATION OF
KEY STRUCTURAL ELEMENTS

8411050179 -07



LEGEND

= LOAD BEARING SHEAR WALL (EL. 240'-260')

= COLUMNS (EL. 240'-260')

= GIRDERS (EL. 260')

= LOAD BEARING WALLS (EL. 240'-260')

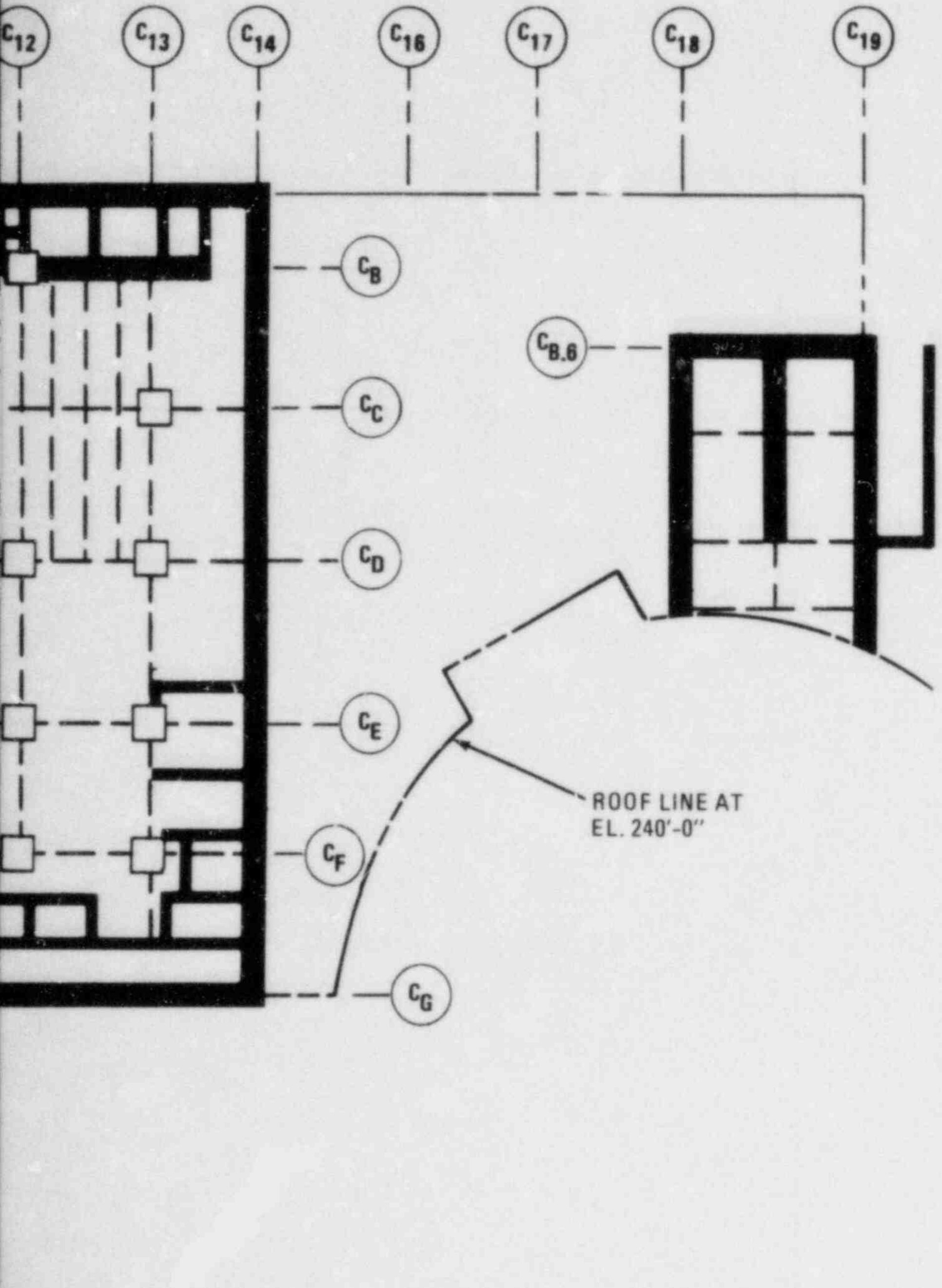
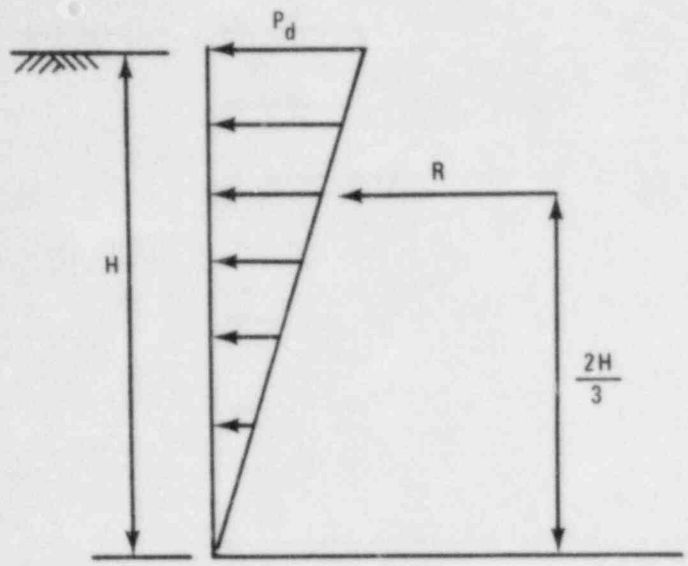


Figure 8
EL. 260'-0", LEVEL 3 LOCATION OF
KEY STRUCTURAL ELEMENTS



H: HEIGHT FROM BASE OF THE STRUCTURE TO THE SOIL SURFACE

P_d = DYNAMIC INCREMENTAL SOIL PRESSURE

R = RESULTANT FORCE

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

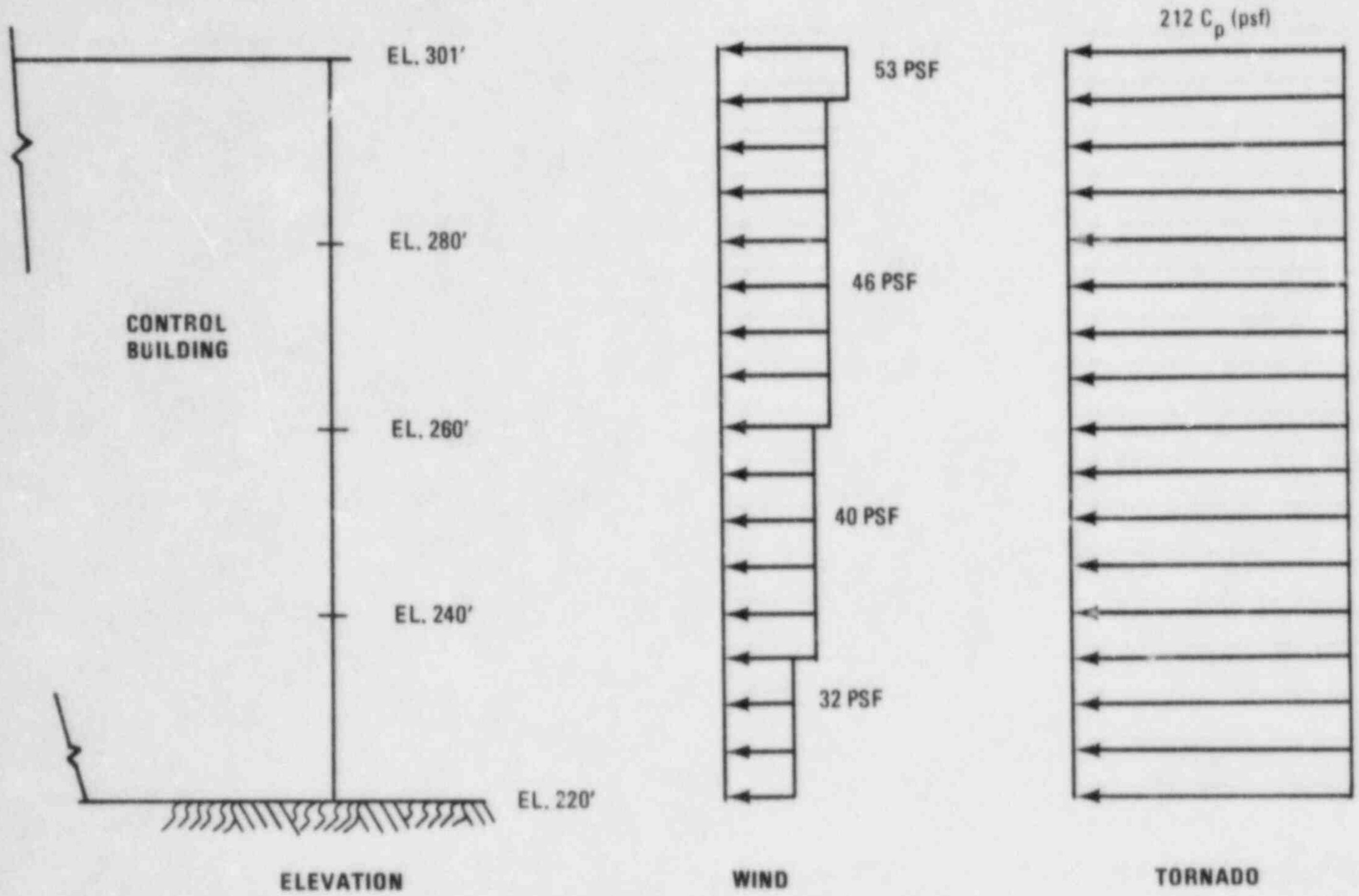
$= .045\,\gamma_M H^2$ (OBE)*

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

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



$P = C_s P_{max} C_p$

WHERE:

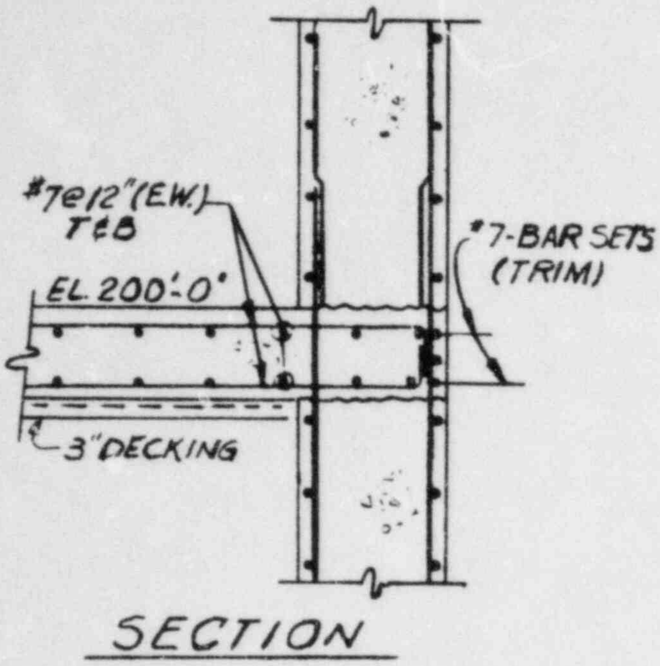
C_s = SIZE COEFFICIENT
= .64

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

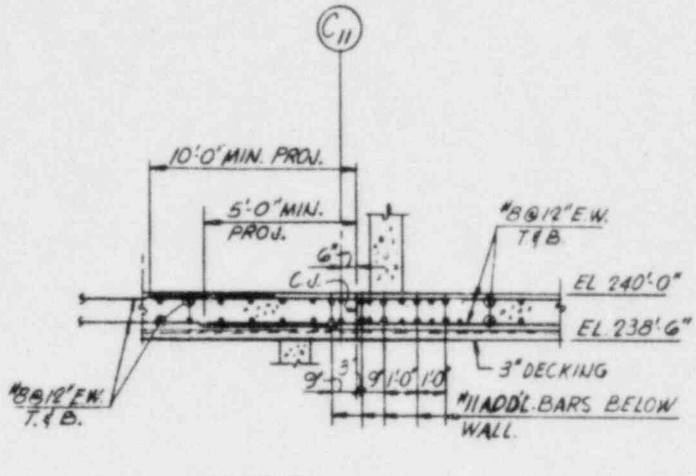
C_p = EFFECTIVE EXTERNAL PRESSURE COEFFICIENT

$P = (.64)\ (332\ \text{psf})\ C_p$
$= 212\ C_p$ (psf)

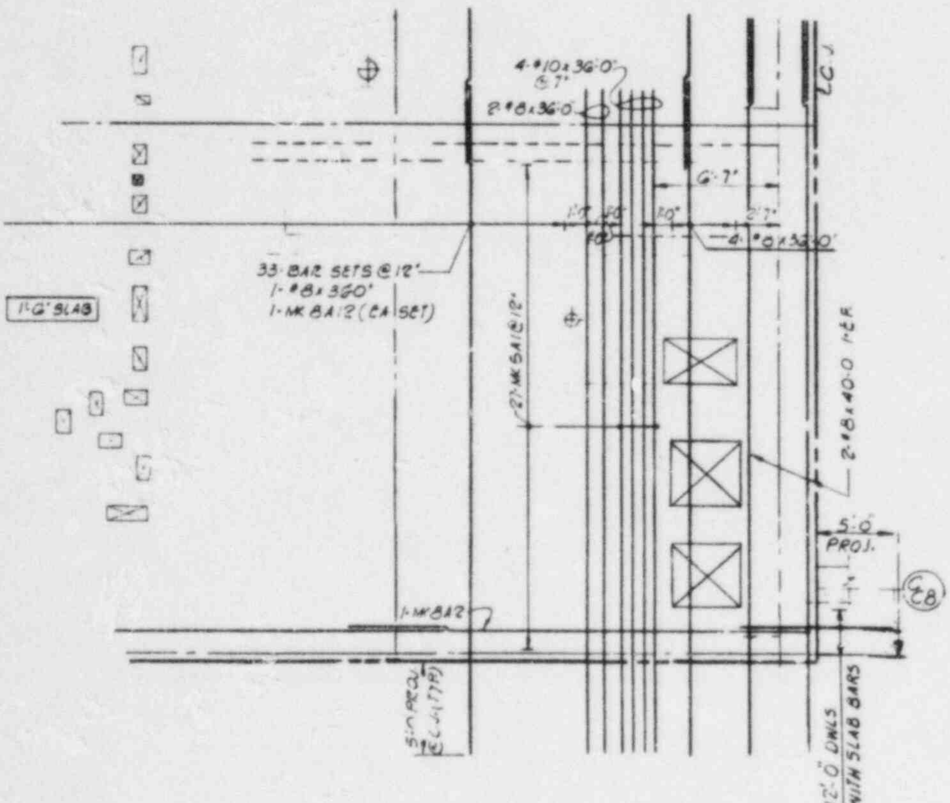
Figure 10
WIND AND TORNADO EFFECTIVE VELOCITY PRESSURE PROFILES



SLAB TO WALL

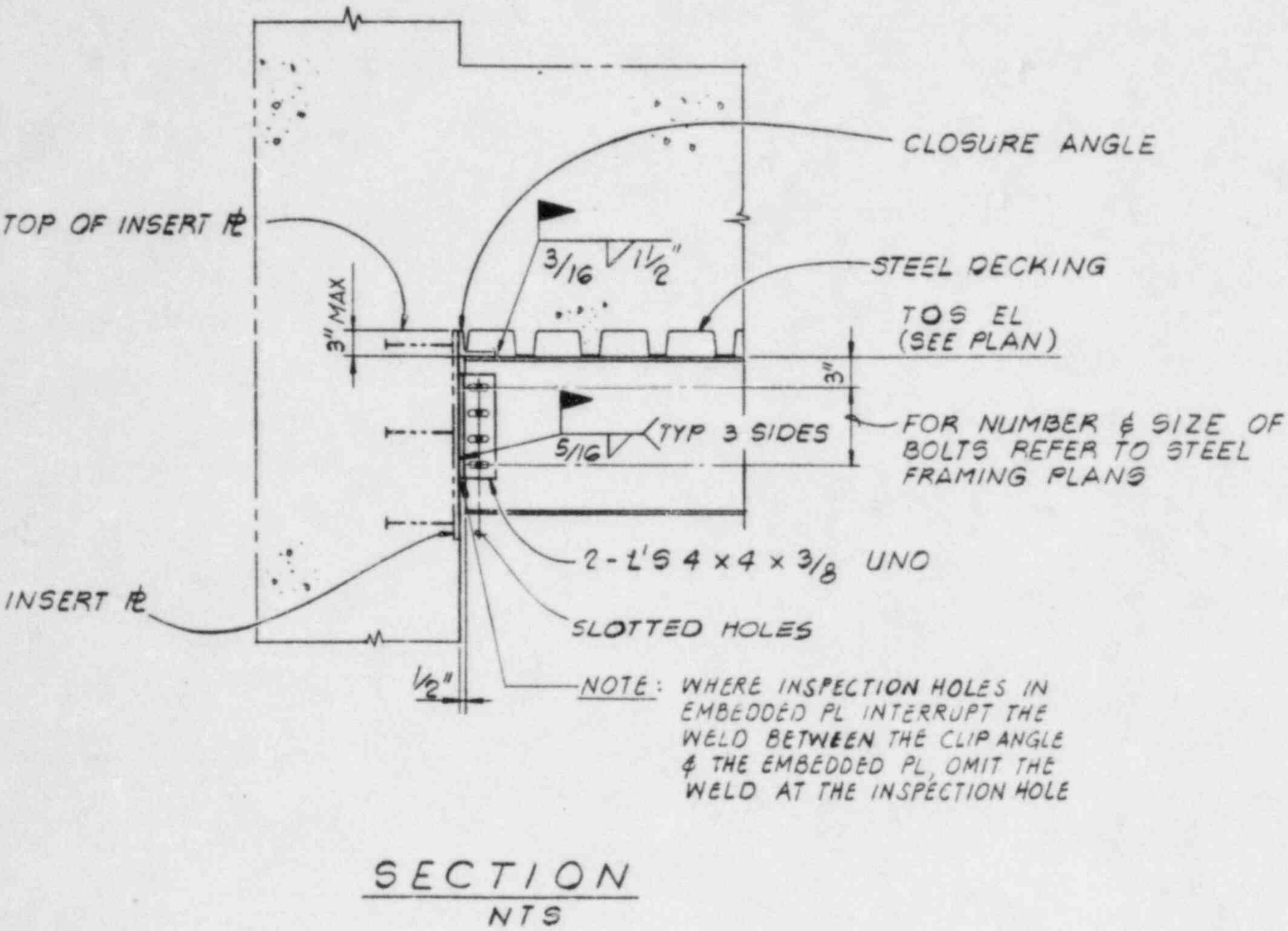


DISCONTINUOUS VERTICAL WALL



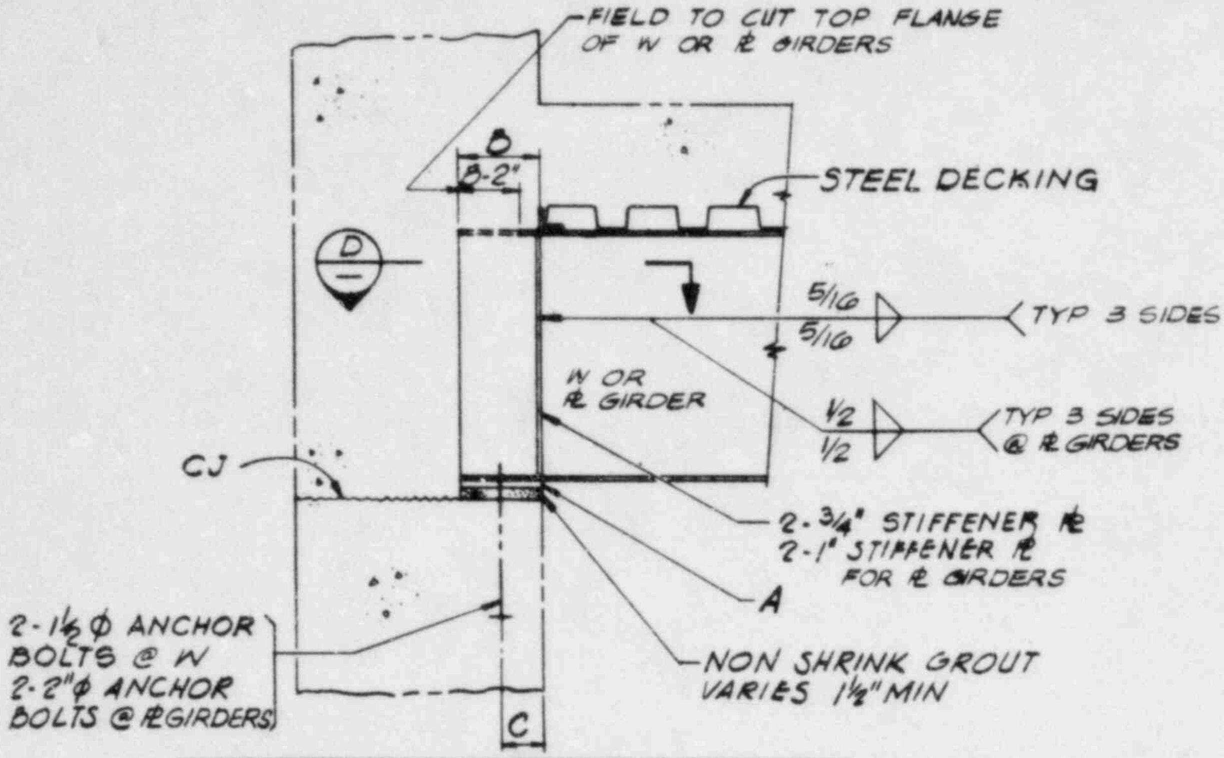
PARTIAL PLAN – 240' SLAB

Figure 11
REPRESENTATIVE ROOF AND FLOOR SLAB DETAILS



WEB CLIP ANGLE PINNED CONNECTION

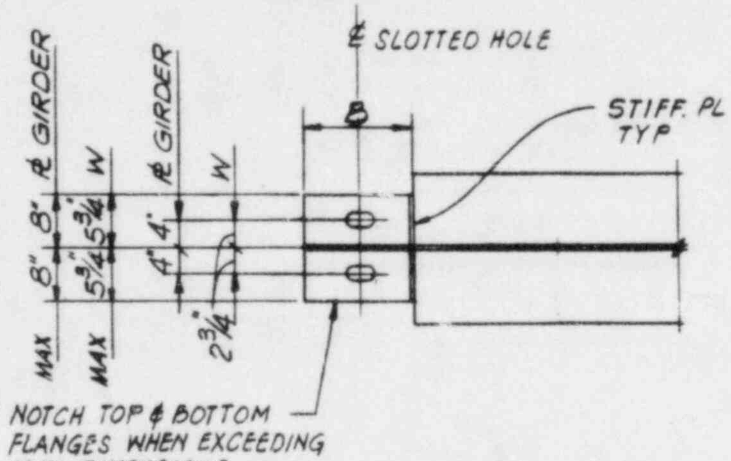
Figure 12
REPRESENTATIVE STRUCTURAL STEEL
GIRDER DESIGN DETAILS
(Sheet 1 of 3)



COLUMN	BEAM	A	B	C	REMARKS
48x48	W	1"x12"x1'-5	1'-0"	6"	℄ WALLS
48x48	PL GIRDER	[illegible]x1'-5"	1'-0"	6"	℄ WALLS
42x42	W	1"x[illegible]x1'-0	1'-0"	6"	
36x48	W	1"x12"x1'-5	1'-0"	6"	
48x48	PL GIRDER	2"x17"x1'-6	1'-6"	9"	PL GIRDER #5, #6 & #7
36x36	W	1"x10"x1'-4"	10"	5"	
36x36	W36x280	1"x12"x1'-4"	1'-0"	8"	
36x36	PL GIRDER	2"x12"x1'-4"	1'-0"	6"	
36x48	PL GIRDER	2"x12"x1'-5"	1'-0"	6"	

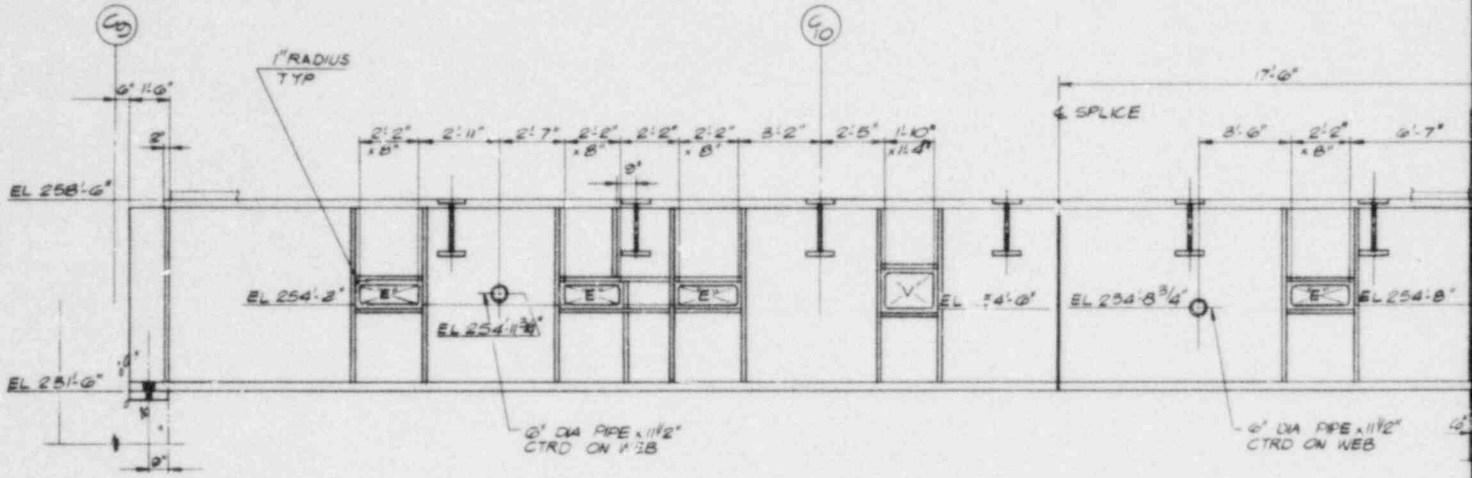
SECTION
NTS

BOTTOM FLANGE BEARING PLATE
CONNECTION



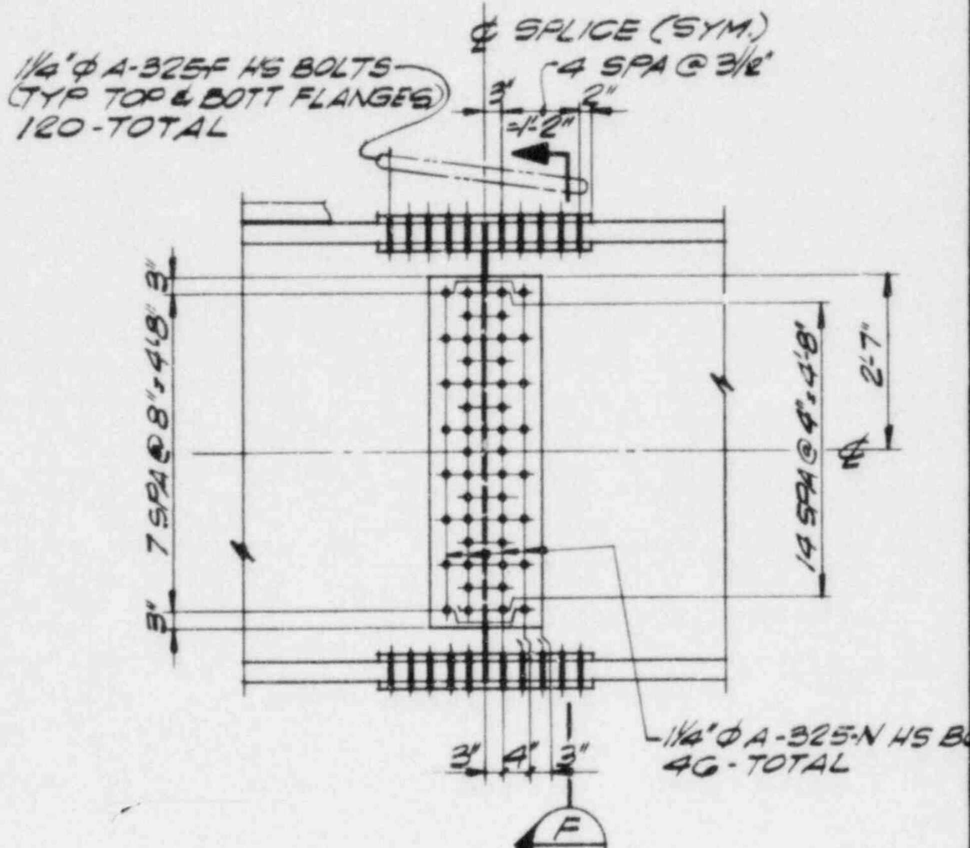
PLAN
NTS
D

Figure 12
REPRESENTATIVE STRUCTURAL STEEL
GIRDER DESIGN DETAILS
(Sheet 2 of 3)



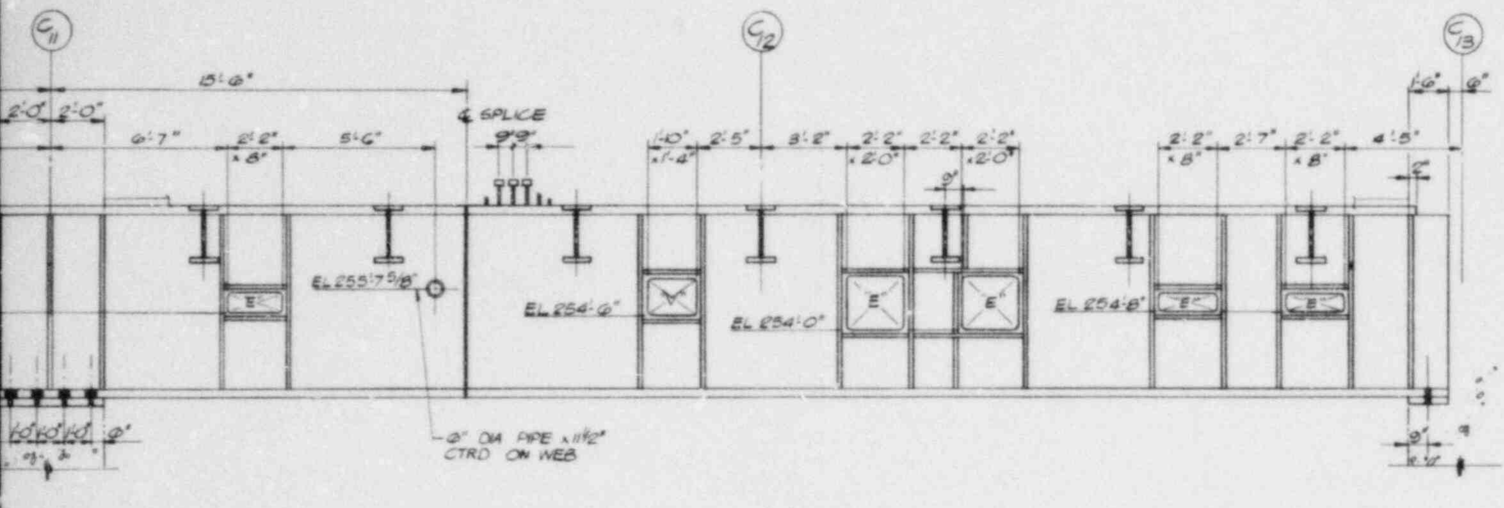
EL

PLATE



DETAIL

FULL SECTIO



EVATION

GIRDER NO. 6

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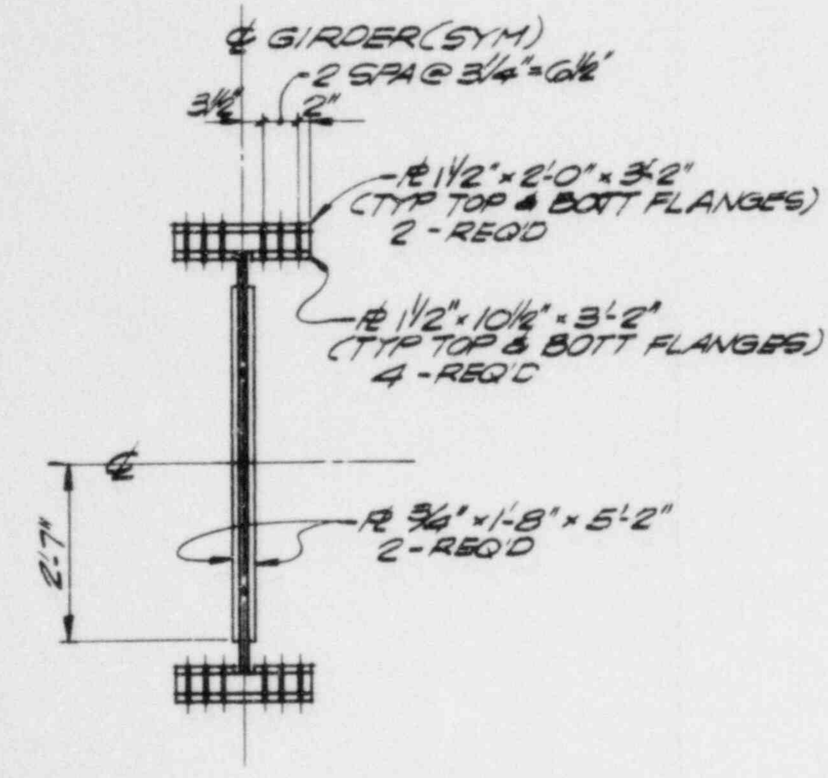


Also Available On
Aperture Card

OLTS

SECTION F

N SPLICE

Figure 12
REPRESENTATIVE STRUCTURAL STEEL
GIRDER DESIGN DETAILS
(Sheet 3 of 3)

8411050179-09

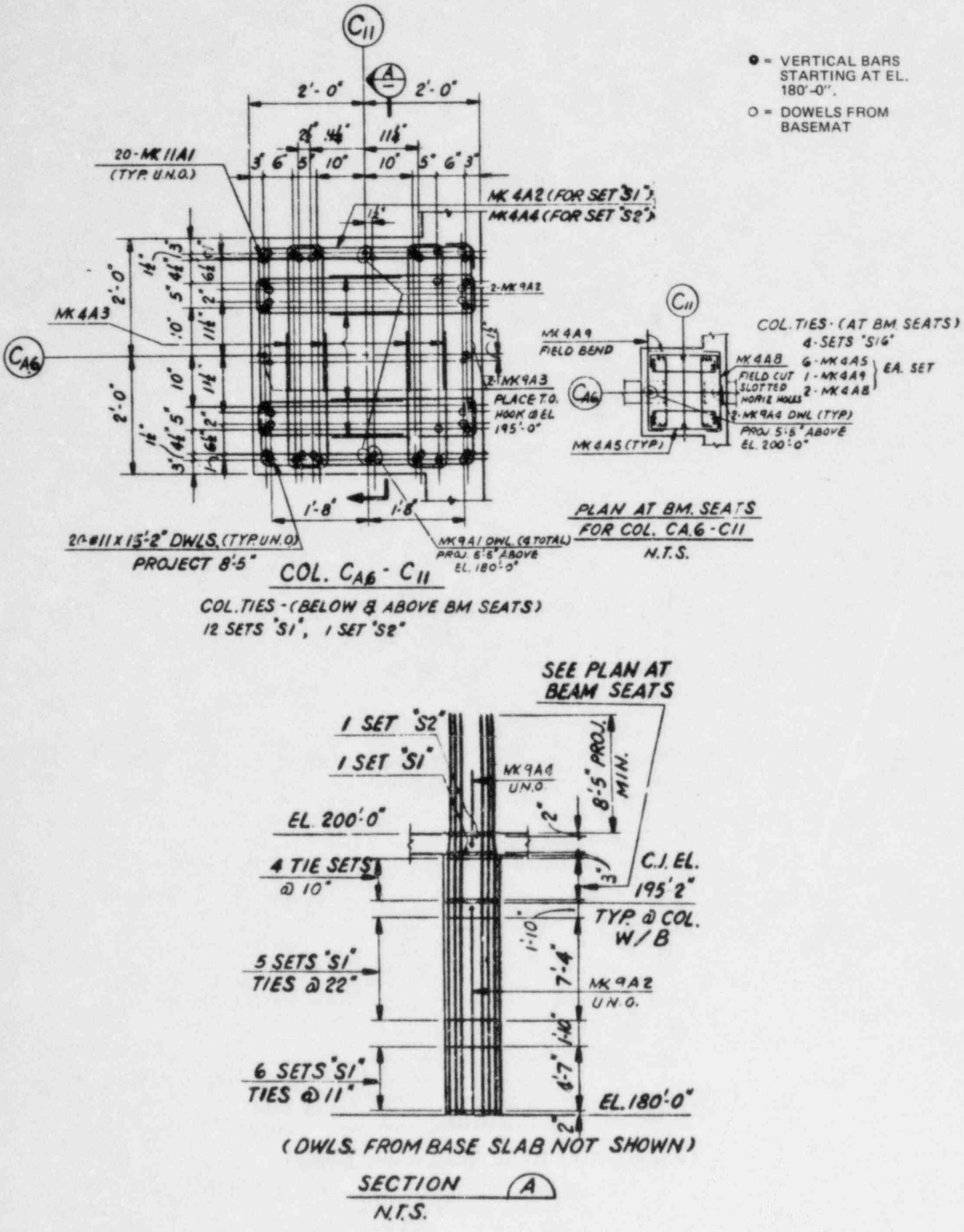
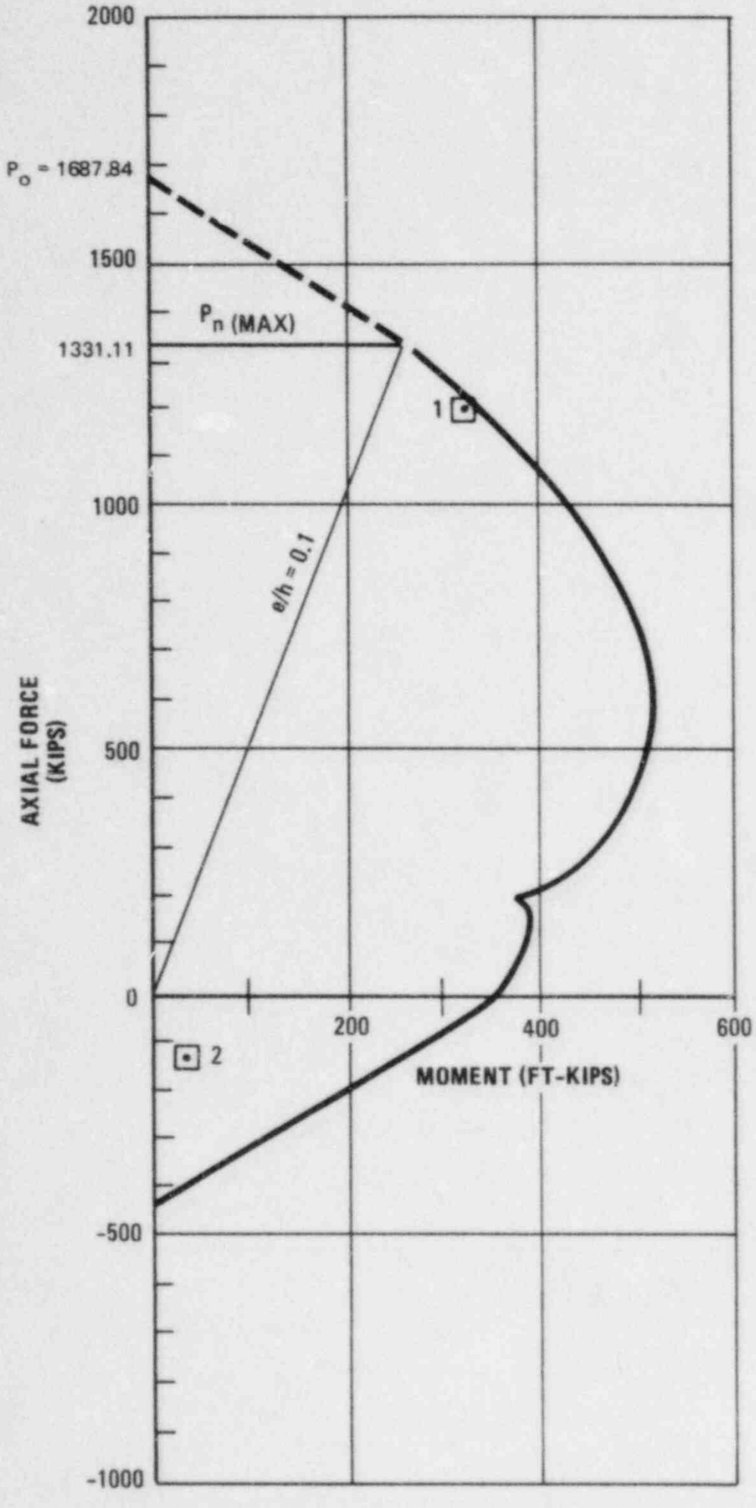
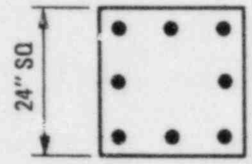


Figure 13
REPRESENTATIVE REINFORCED
CONCRETE COLUMN DESIGN DETAILS



COLUMN: $C_{9.0}$ - $C_{F.0}$
ELEV: 280′-0″
REBAR: 8–#9

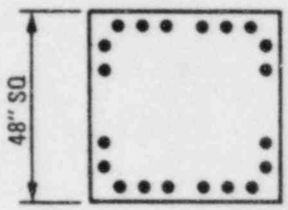


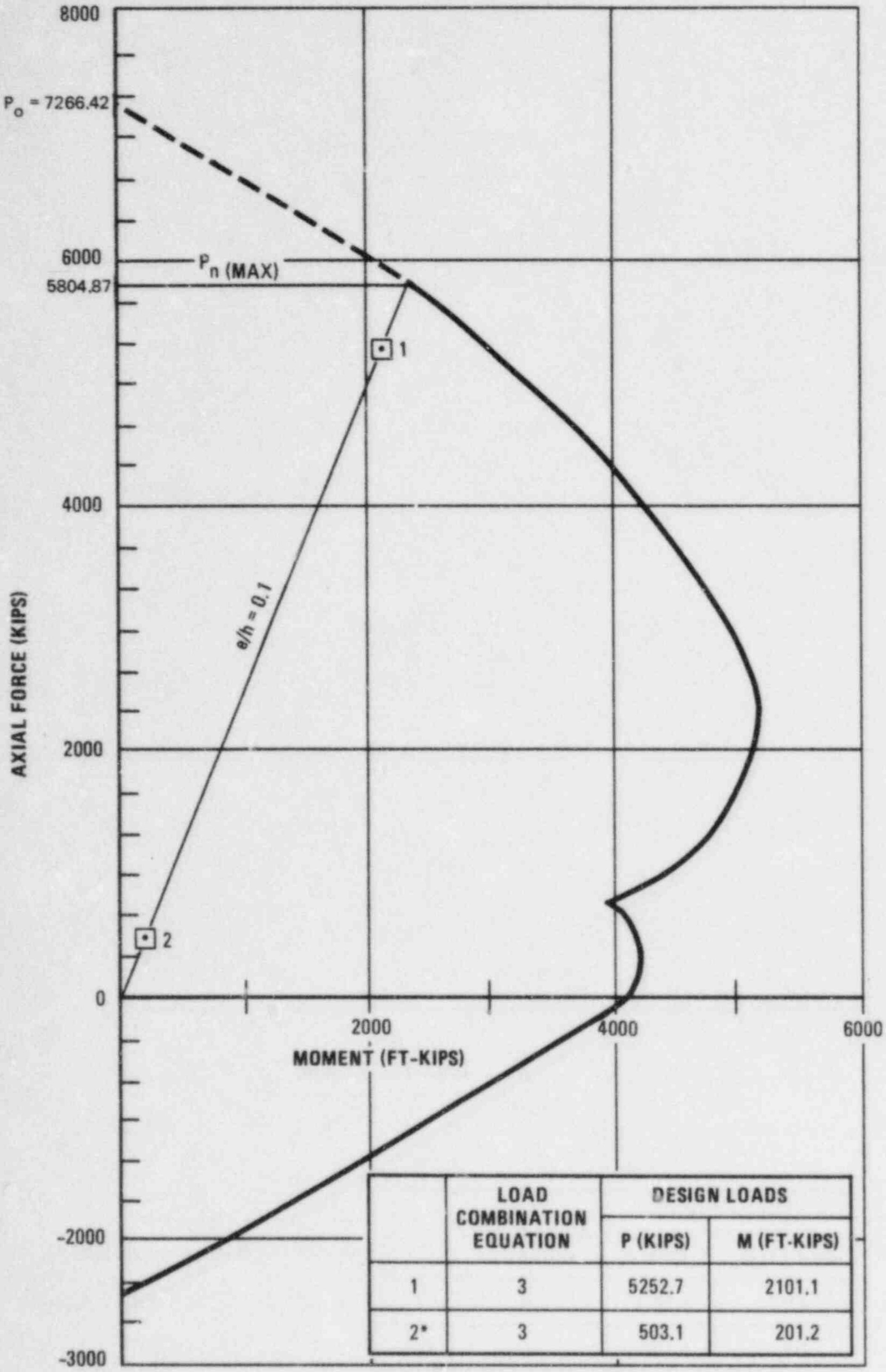
	LOAD COMBINATION EQUATION	DESIGN LOADS	
		P (KIPS)	M (FT-KIPS)
1	3	1210.5	324.9
2*	3	−137.5	36.9

*PER FOOTNOTE C, TABLE 3.2, APPENDIX B.

Figure 14
INTERACTION DIAGRAM FOR
COLUMN $C_{9.0}$ – $C_{F.0}$

COLUMN: $C_{12.0} - C_{D.0}$
ELEV: 180'-0"
REBAR: 20 #14

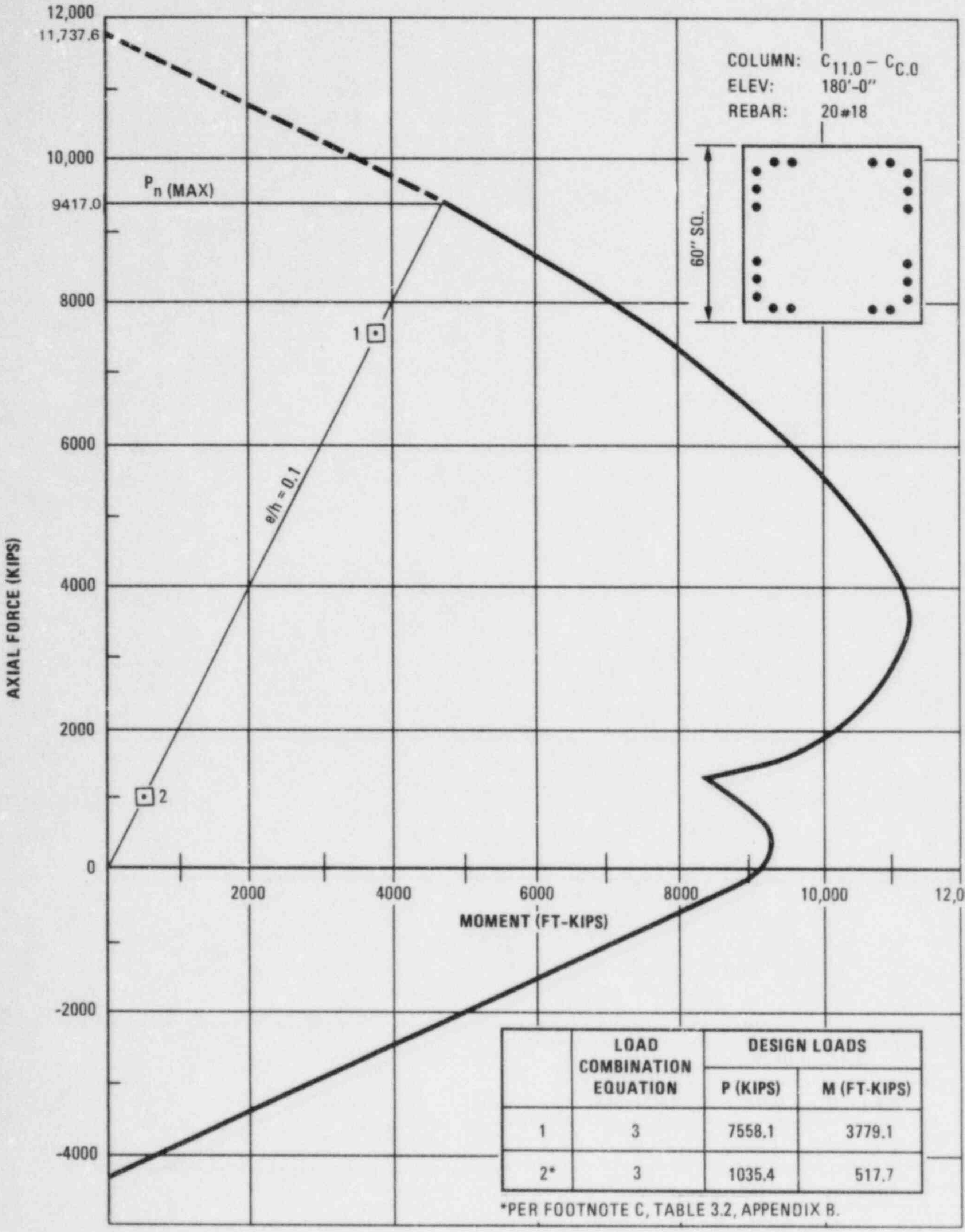




	LOAD COMBINATION EQUATION	DESIGN LOADS	
		P (KIPS)	M (FT-KIPS)
1	3	5252.7	2101.1
2*	3	503.1	201.2

*PER FOOTNOTE C, TABLE 3.2, APPENDIX B.

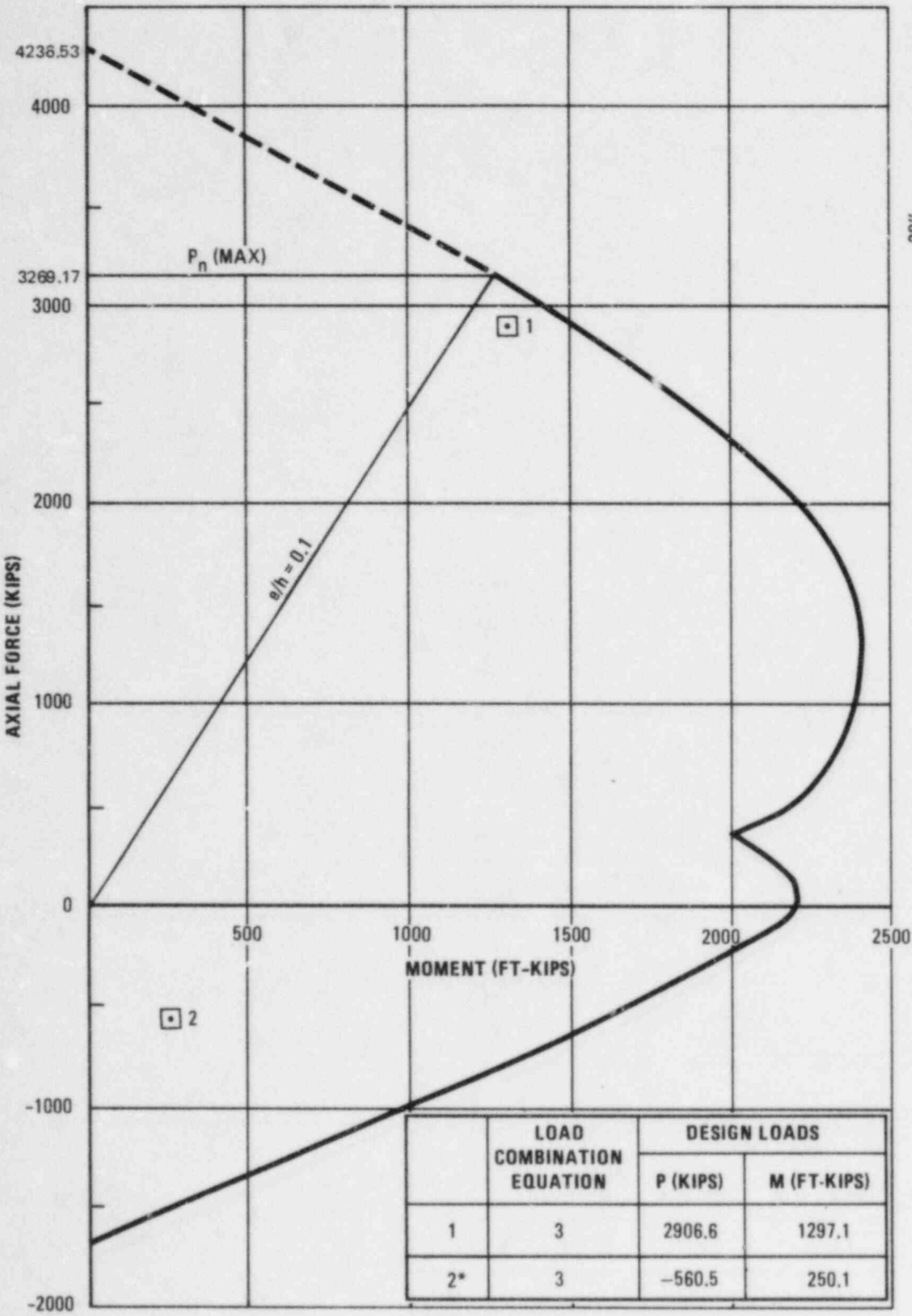
Figure 15
INTERACTION DIAGRAM FOR
COLUMN $C_{12.0} - C_{D.0}$



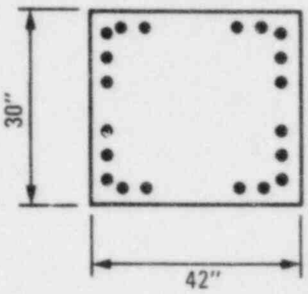
	LOAD COMBINATION EQUATION	DESIGN LOADS	
		P (KIPS)	M (FT-KIPS)
1	3	7558.1	3779.1
2*	3	1035.4	517.7

*PER FOOTNOTE C, TABLE 3.2, APPENDIX B.

Figure 16
INTERACTION DIAGRAM FOR
COLUMN $C_{11.0} - C_{C.0}$



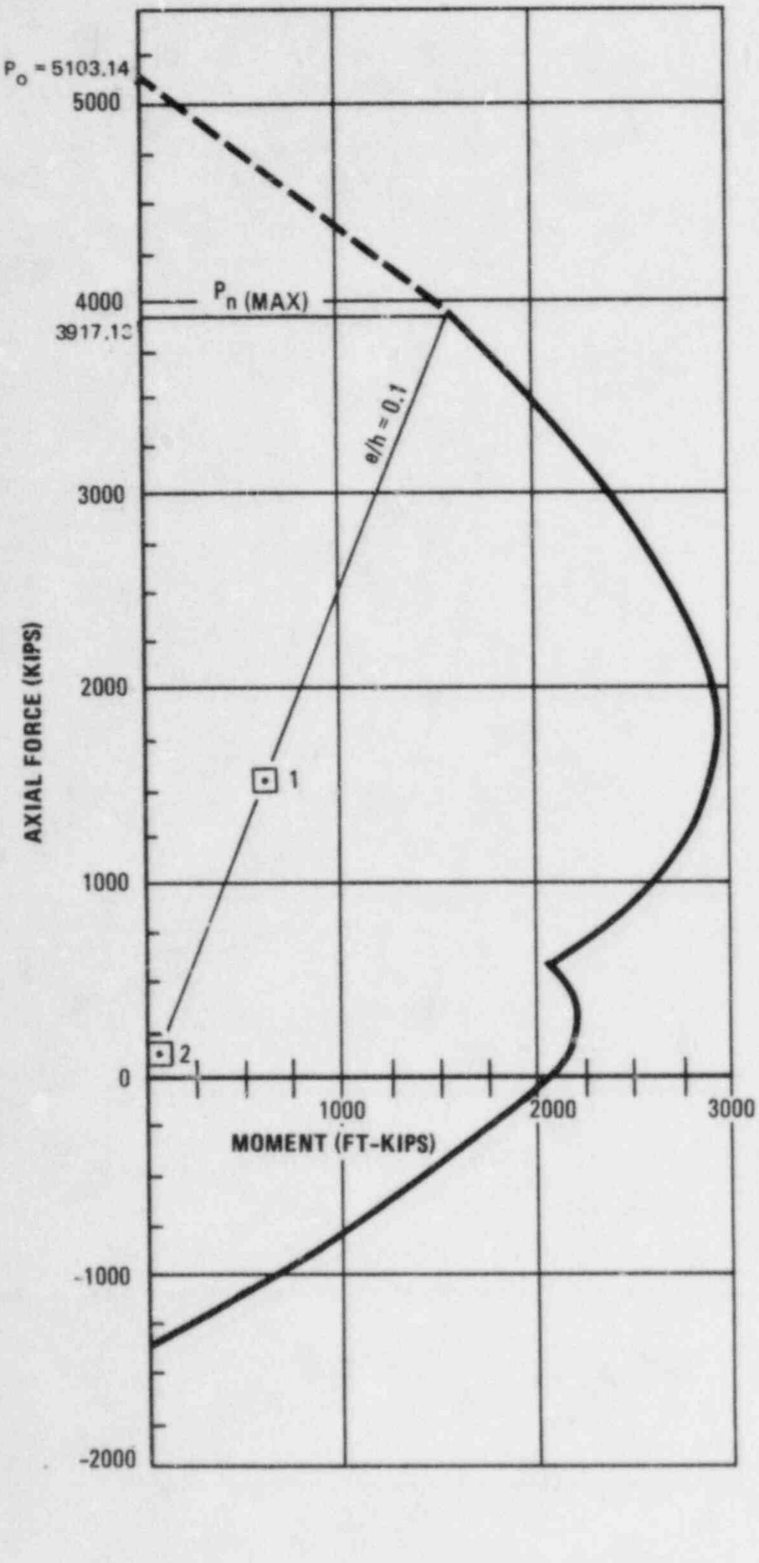
COLUMN: $C_{5.0} - C_{C.8}$
ELEV: 180'-0"
REBAR: 20#11



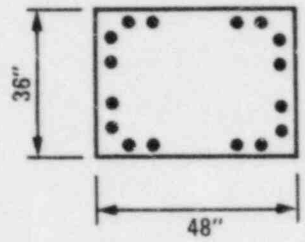
	LOAD COMBINATION EQUATION	DESIGN LOADS	
		P (KIPS)	M (FT-KIPS)
1	3	2906.6	1297.1
2*	3	–560.5	250.1

*PER FOOTNOTE C, TABLE 3.2, APPENDIX B.

Figure 17
INTERACTION DIAGRAM FOR
COLUMN $C_{5.0} - C_{C.8}$



COLUMN: $C_{6.0} - C_{B.6}$
ELEV: 220'-0"
REBAR: 16#11



	LOAD COMBINATION EQUATION	DESIGN LOADS	
		P (KIPS)	M (FT-KIPS)
1	3	1539.8	615.9
2*	3	157.2	62.9

*PER FOOTNOTE C, TABLE 3.2, APPENDIX B.

Figure 18
INTERACTION DIAGRAM FOR
COLUMN $C_{6.0} - C_{B.6}$

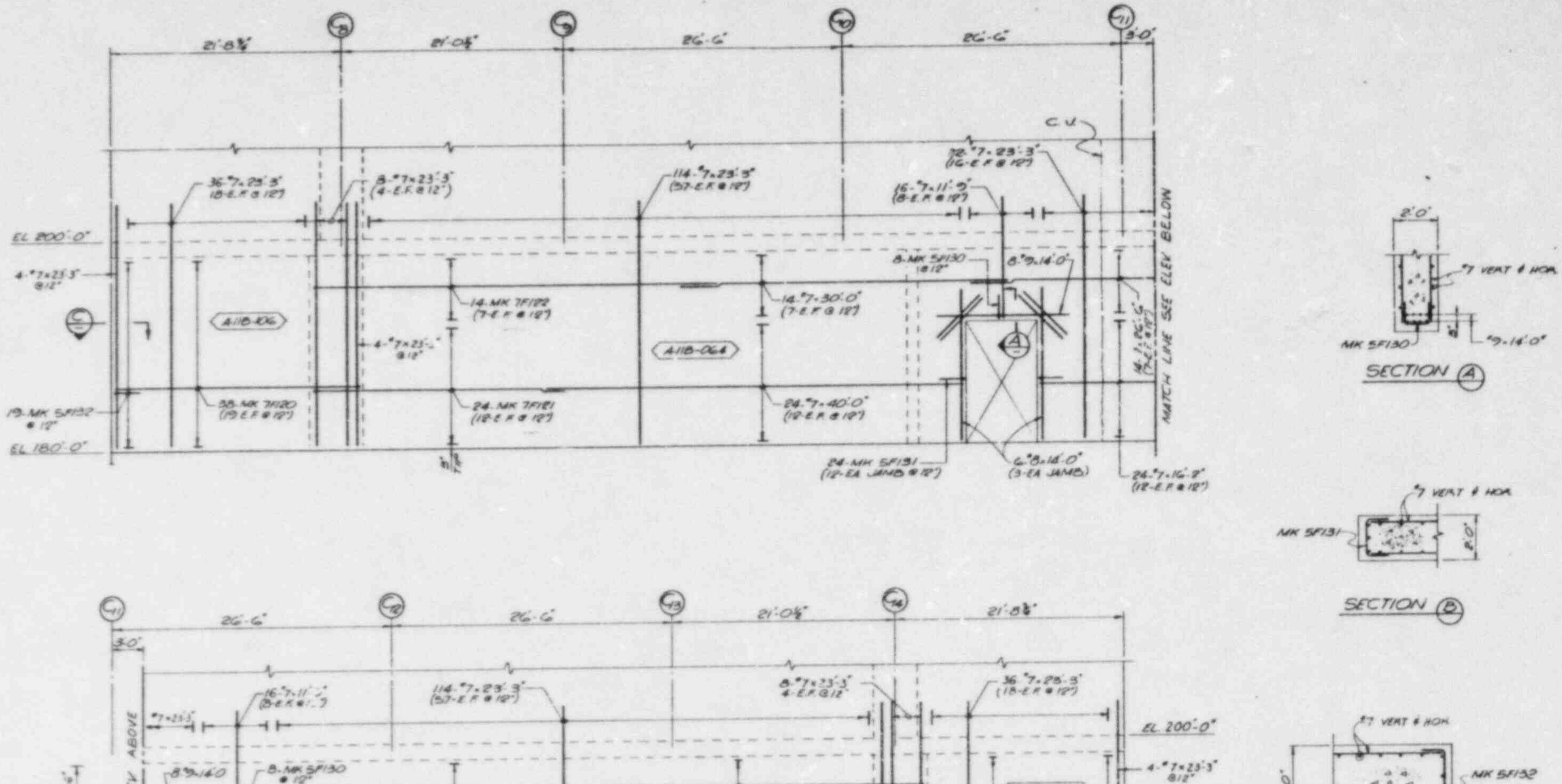
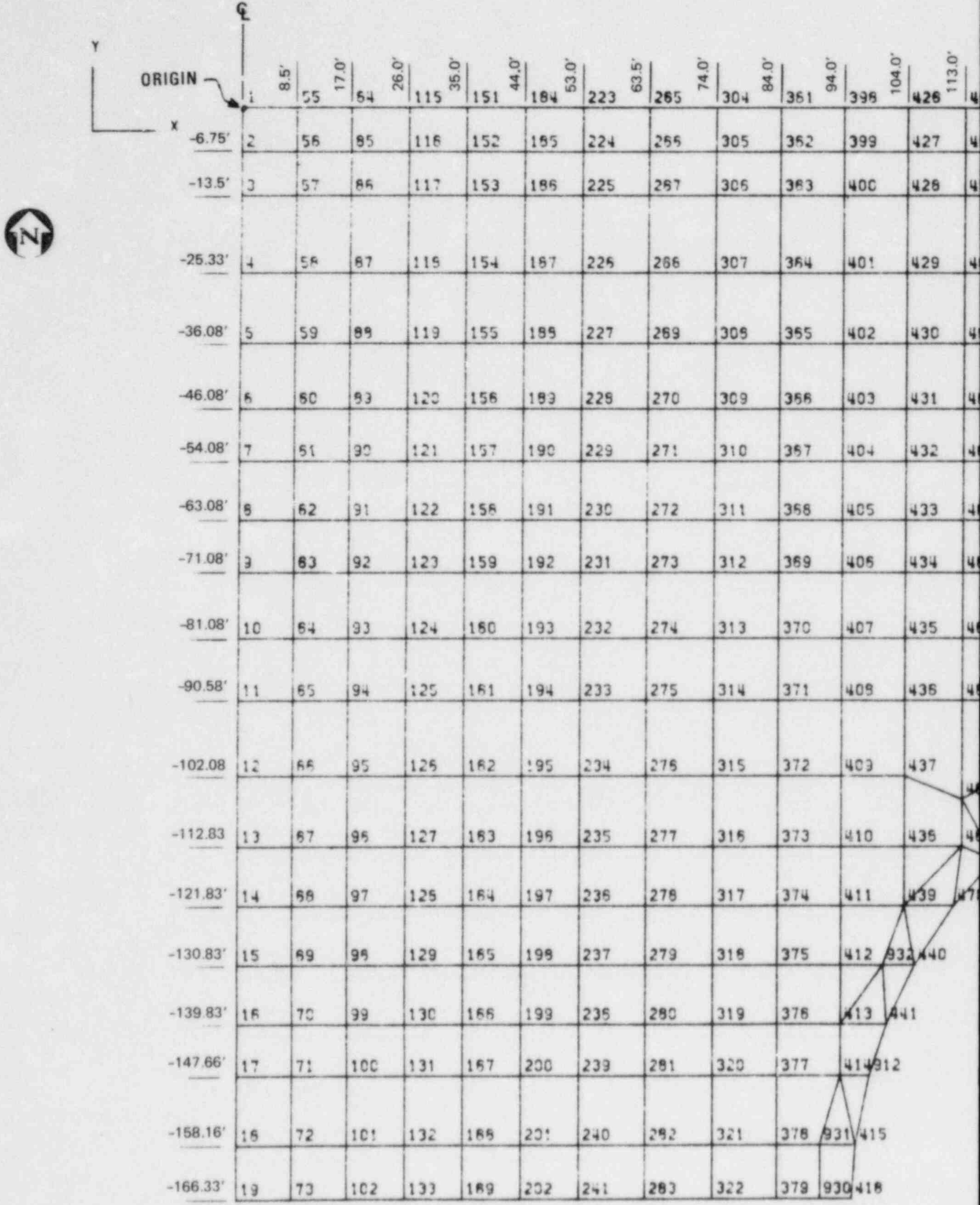
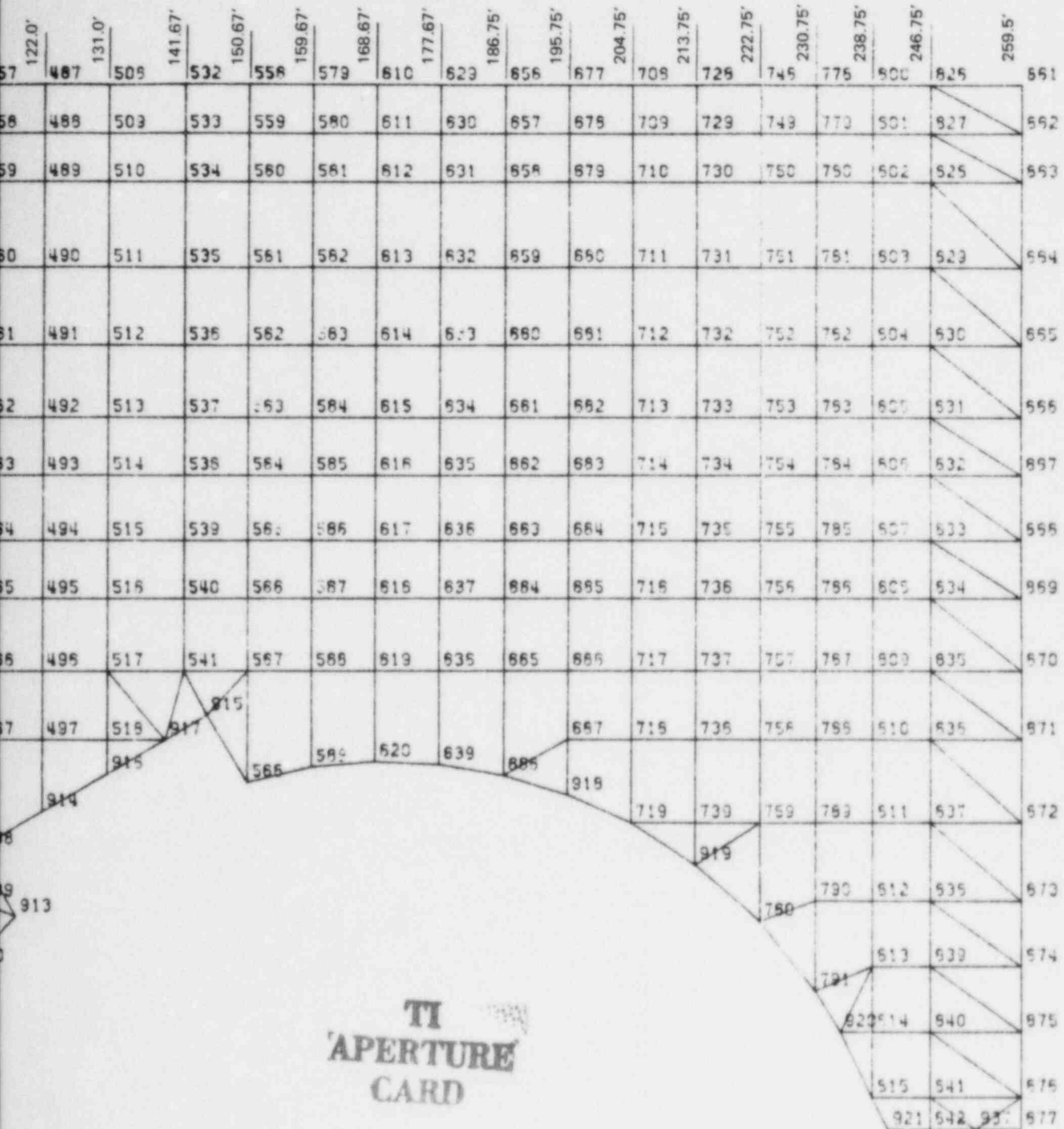


Figure 19
REPRESENTATIVE SHEAR WALL DESIGN DETAILS



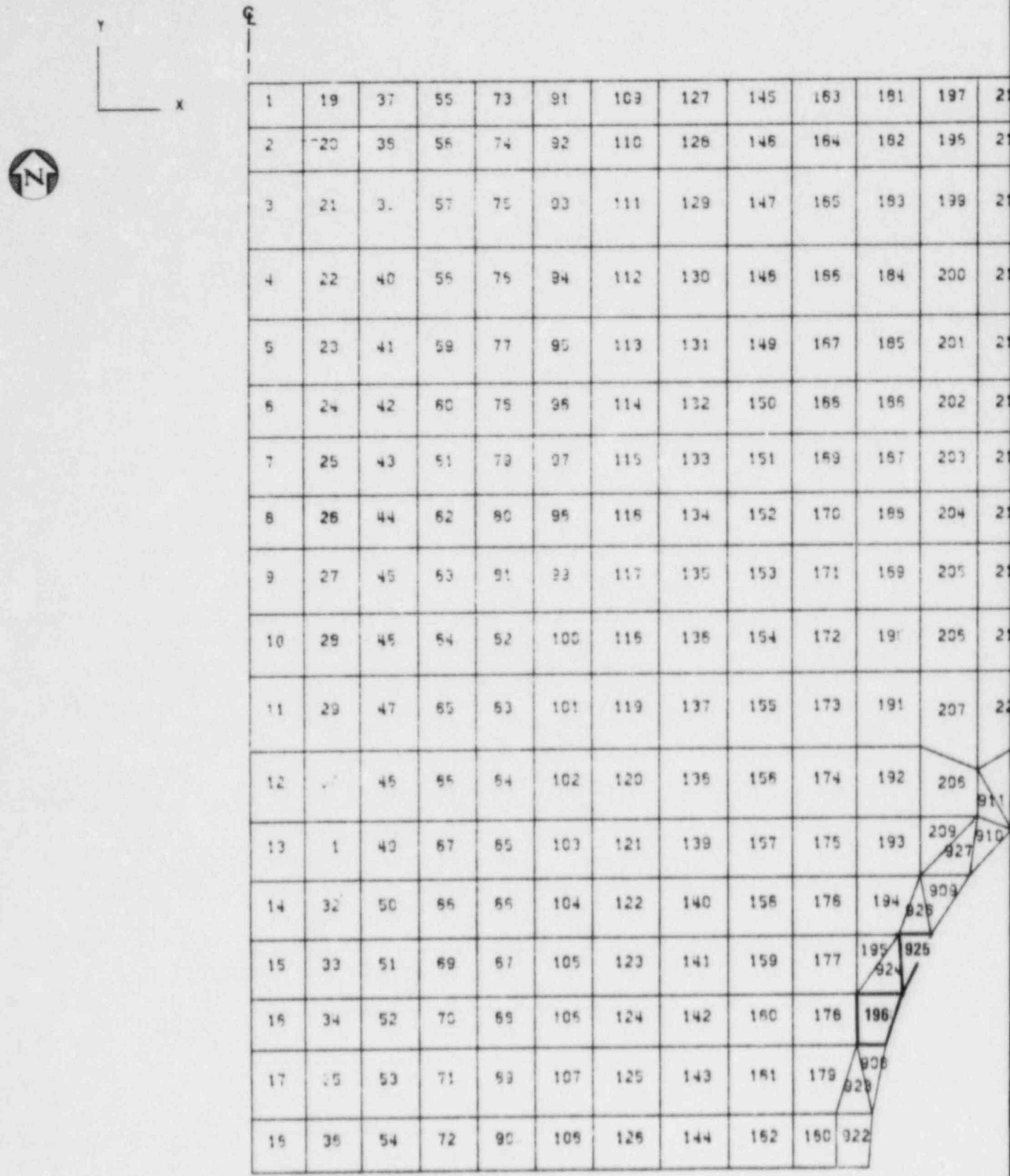
℄
Y
X
ORIGIN
8.5'
17.0'
26.0'
35.0'
44.0'
53.0'
63.5'
74.0'
84.0'
94.0'
104.0'
113.0'
1 55 84 115 151 184 223 265 304 361 398 426
-6.75' 2 56 85 116 152 185 224 266 305 362 399 427
-13.5' 3 57 86 117 153 186 225 267 306 363 400 428
-25.33' 4 58 87 118 154 187 226 268 307 364 401 429
-36.08' 5 59 88 119 155 188 227 269 308 365 402 430
-46.08' 6 60 89 120 156 189 228 270 309 366 403 431
-54.08' 7 61 90 121 157 190 229 271 310 367 404 432
-63.08' 8 62 91 122 158 191 230 272 311 368 405 433
-71.08' 9 63 92 123 159 192 231 273 312 369 406 434
-81.08' 10 64 93 124 160 193 232 274 313 370 407 435
-90.58' 11 65 94 125 161 194 233 275 314 371 408 436
-102.08 12 66 95 126 162 195 234 276 315 372 409 437
-112.83 13 67 96 127 163 196 235 277 316 373 410 438
-121.83' 14 68 97 128 164 197 236 278 317 374 411 439
-130.83' 15 69 98 129 165 198 237 279 318 375 412 932 440
-139.83' 16 70 99 130 166 199 238 280 319 376 413 441
-147.66' 17 71 100 131 167 200 239 281 320 377 414 912
-158.16' 18 72 101 132 168 201 240 282 321 378 931 415
-166.33' 19 73 102 133 169 202 241 283 322 379 930 416

NODE N

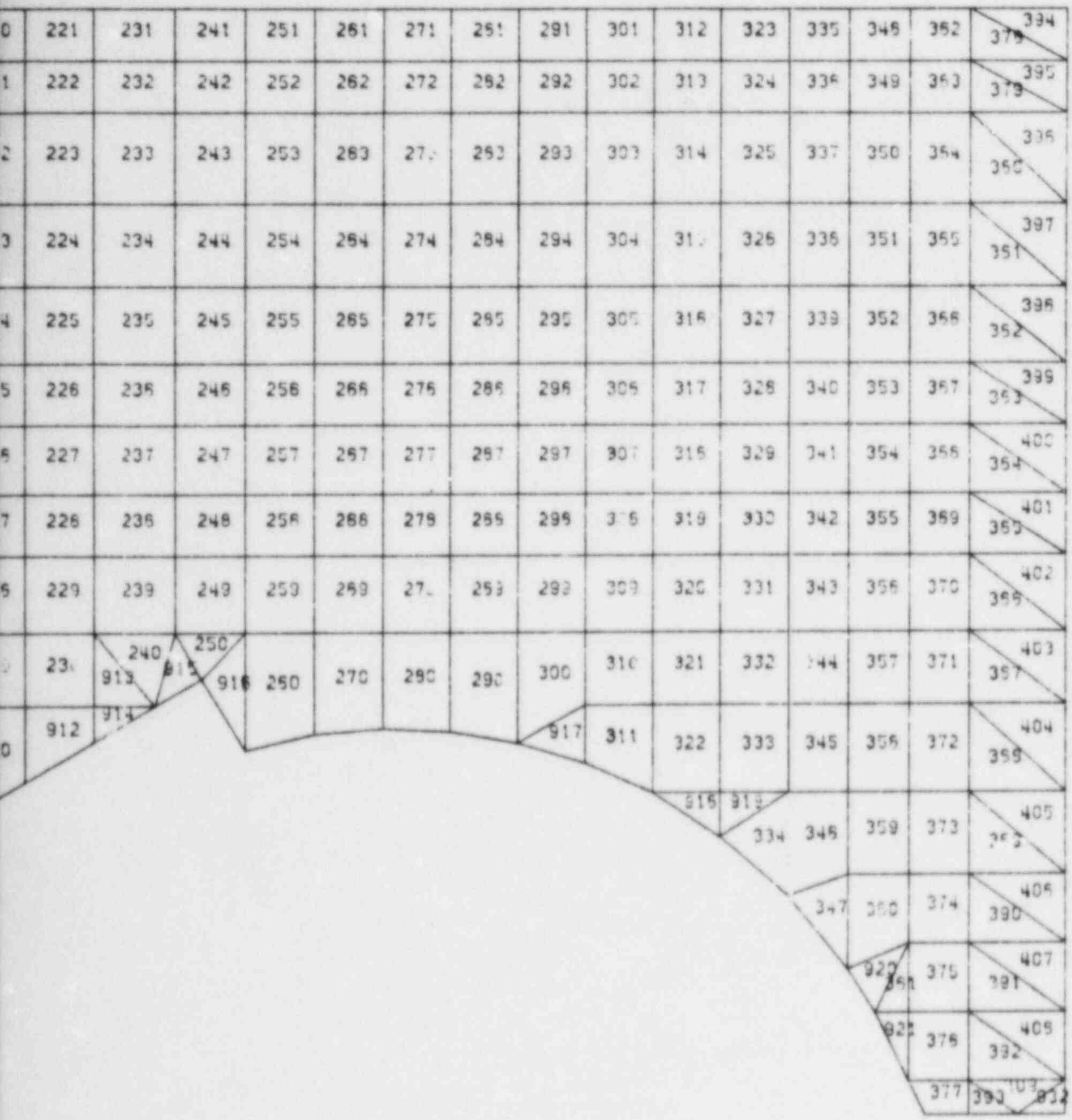


UMBERING

Figure 20
BASEMAT FINITE ELEMENT MODEL
(Sheet 1 of 3)

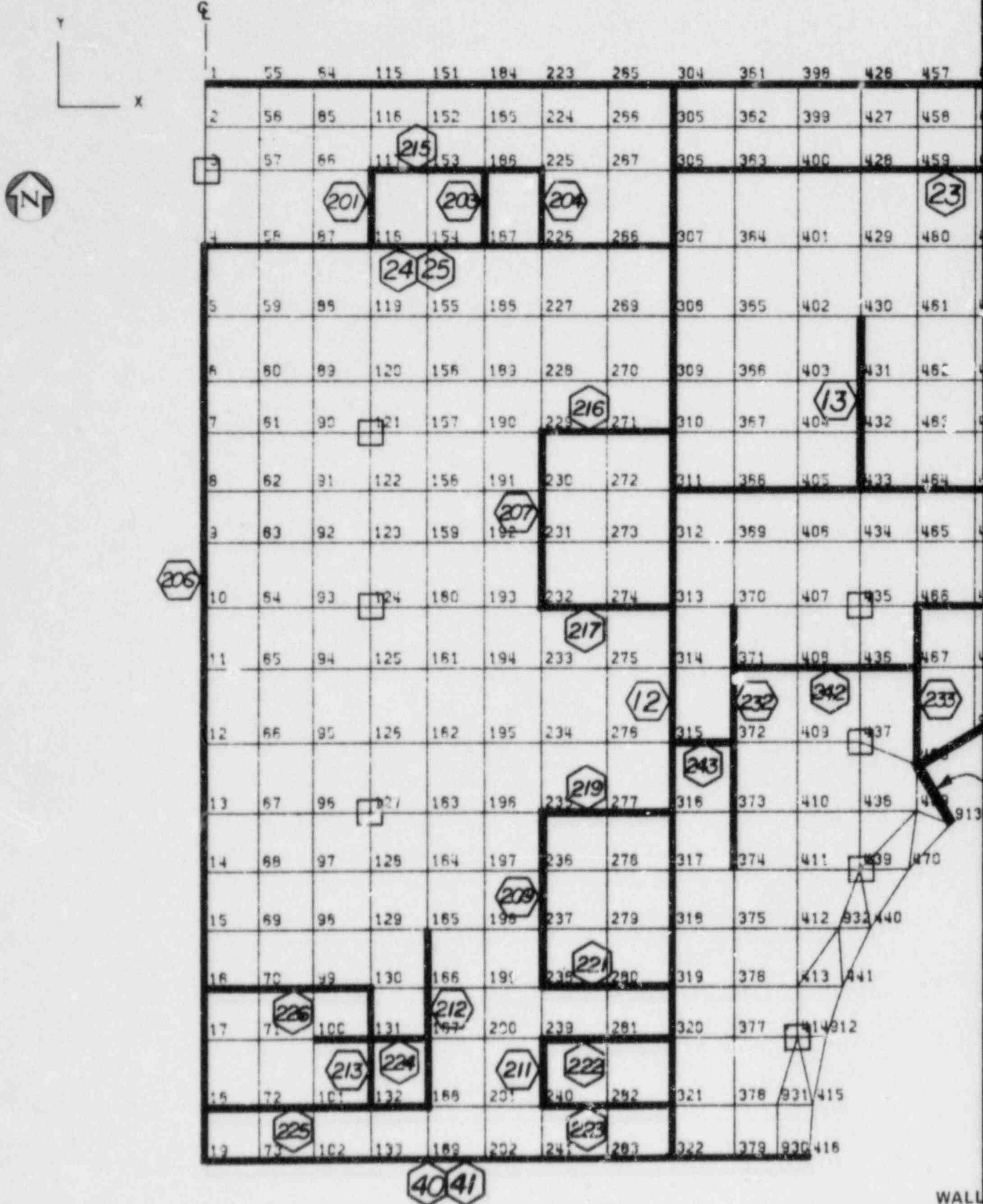


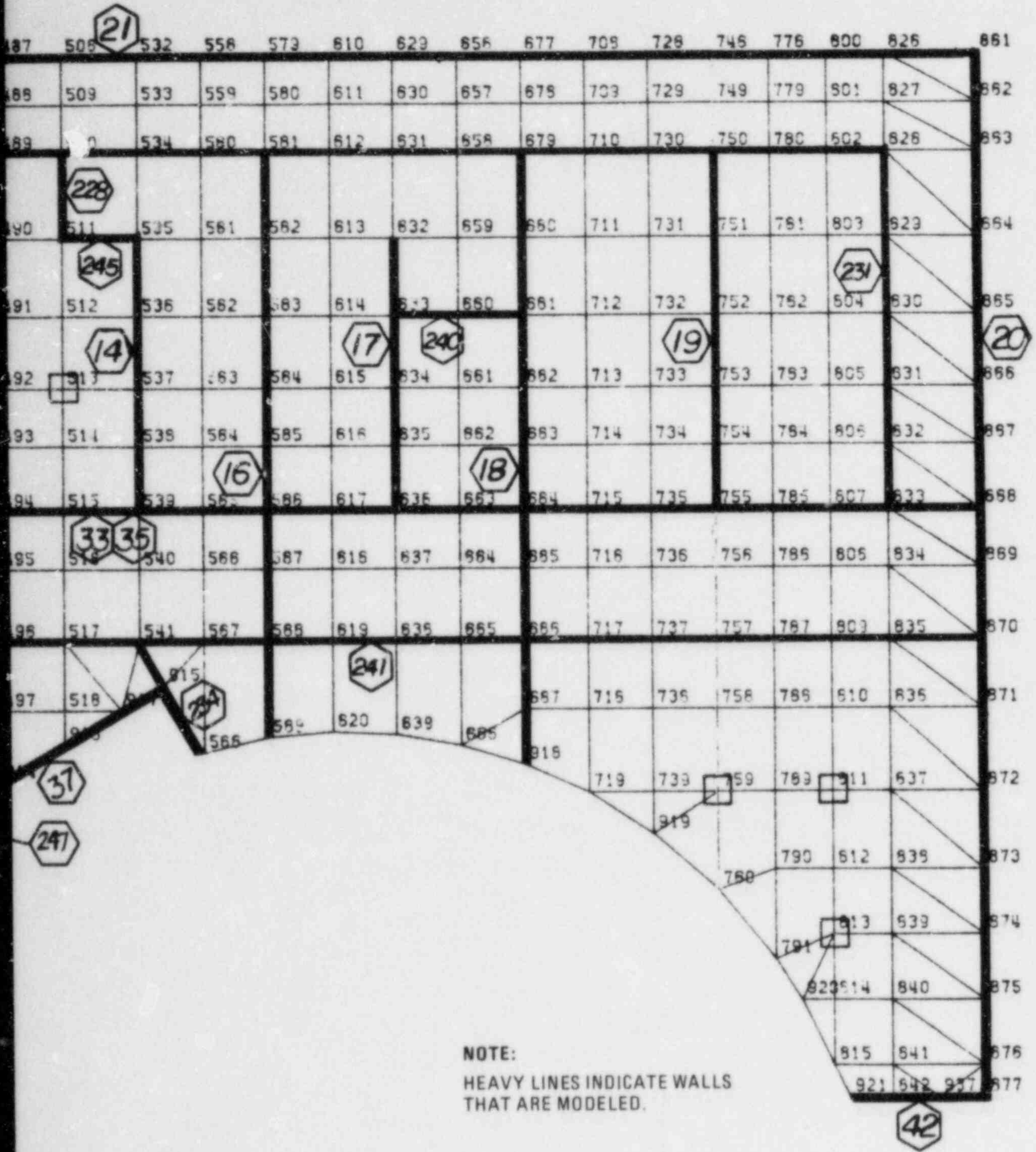
℄
Y
X
N
1 19 37 55 73 91 109 127 145 163 181 197
2 20 38 56 74 92 110 128 146 164 182 198
3 21 57 75 93 111 129 147 165 183 199
4 22 40 58 76 94 112 130 148 166 184 200
5 23 41 59 77 95 113 131 149 167 185 201
6 24 42 60 78 96 114 132 150 168 186 202
7 25 43 61 79 97 115 133 151 169 187 203
8 26 44 62 80 98 116 134 152 170 188 204
9 27 45 63 81 99 117 135 153 171 189 205
10 28 46 64 82 100 118 136 154 172 206
11 29 47 65 83 101 119 137 155 173 191 207
12 48 66 84 102 120 138 156 174 192 208 911
13 49 67 85 103 121 139 157 175 193 209 927 910
14 32 50 68 86 104 122 140 158 176 194 926 909
15 33 51 69 87 105 123 141 159 177 195 925 924
16 34 52 70 88 106 124 142 160 178 196
17 53 71 89 107 125 143 161 179 923 908
18 36 54 72 90 108 126 144 162 180 922



ELEMENT NUMBERING

Figure 20
BASEMAT FINITE ELEMENT MODEL
(Sheet 2 of 3)


Y
X
N
WALL



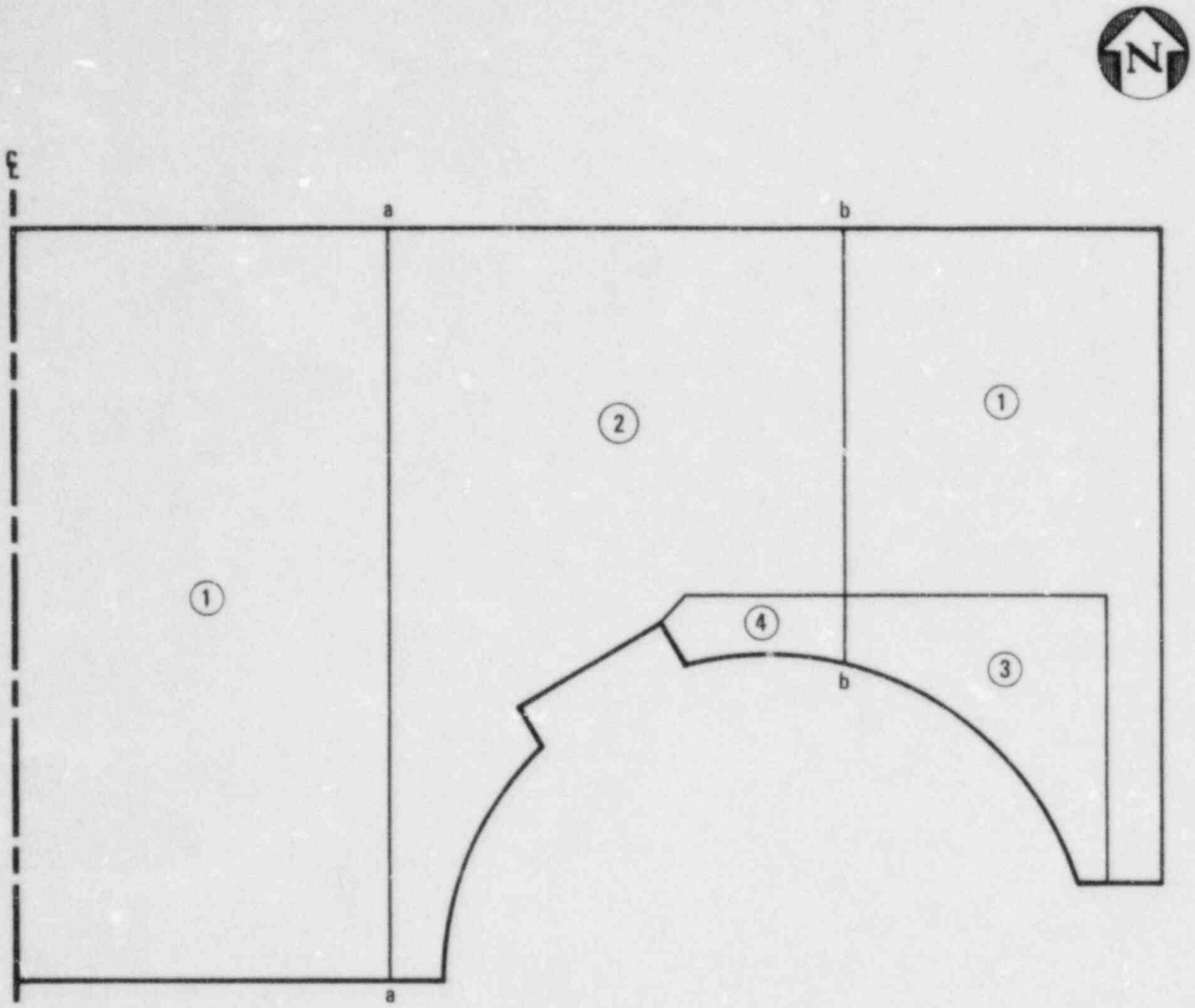
Also Available On
Aperture Card

APERTURE
CARD

ELEMENT NUMBERING

Figure 20
BASEMAT FINITE ELEMENT MODEL
(Sheet 3 of 3)

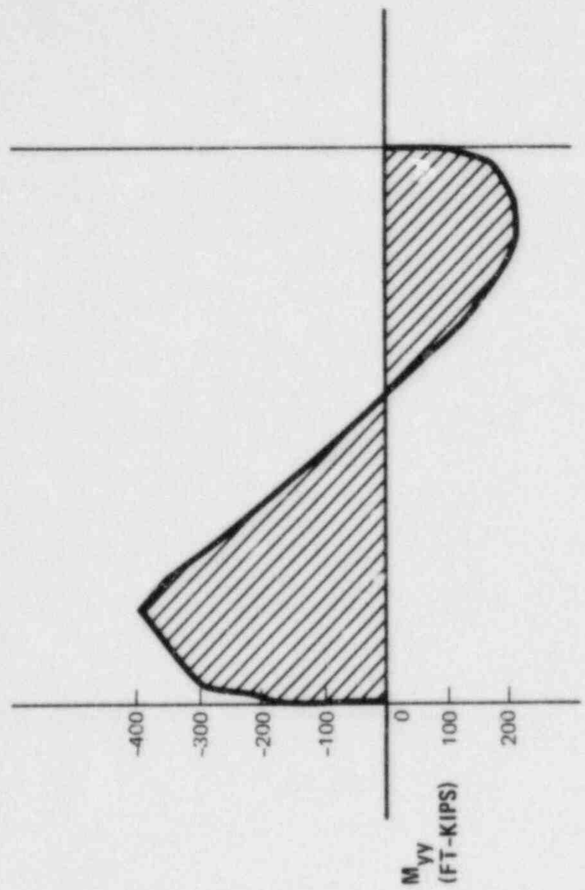
8411050179 -/2



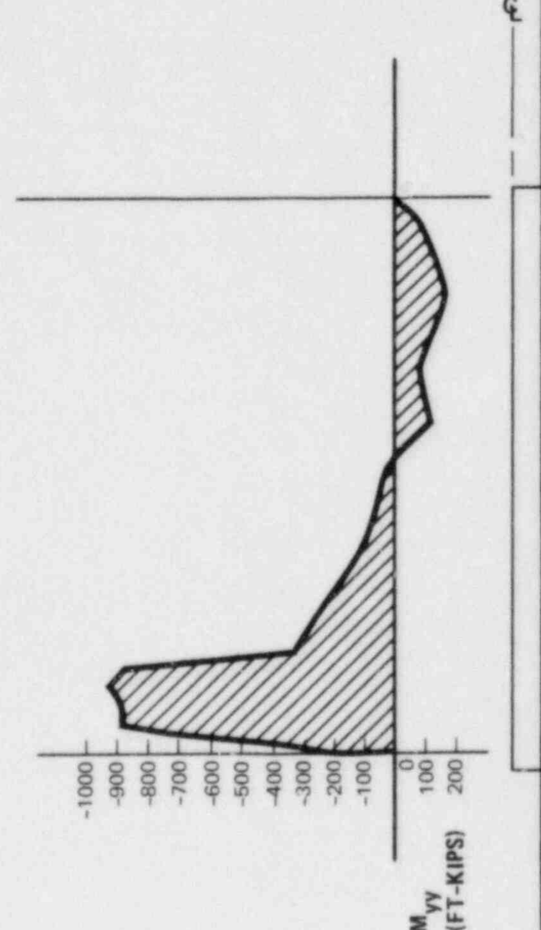
ZONE	MAIN REINFORCING STEEL FURNISHED (EACH WAY, TOP AND BOTTOM)	BASEMAT THICKNESS
1	2 LAYERS #18 @ 12	7 FEET
2	1 LAYER #18 @ 12 1 LAYER #18 @ 36	
3	2 LAYERS # 18 @ 12	10 FEET
4	1 LAYER #18 @ 12 1 LAYER #18 @ 36	

NOTE: THE EAST/WEST MAIN REINFORCING STEEL FOR EACH ZONE CROSSES THE ZONE BOUNDARIES a-a AND b-b AND IS FULLY DEVELOPED BEYOND THESE BOUNDARIES.

Figure 21
BASEMAT MAIN REINFORCING
STEEL BY ZONES



MOMENT PROFILE
AT c-c



MOMENT PROFILE
AT b-b

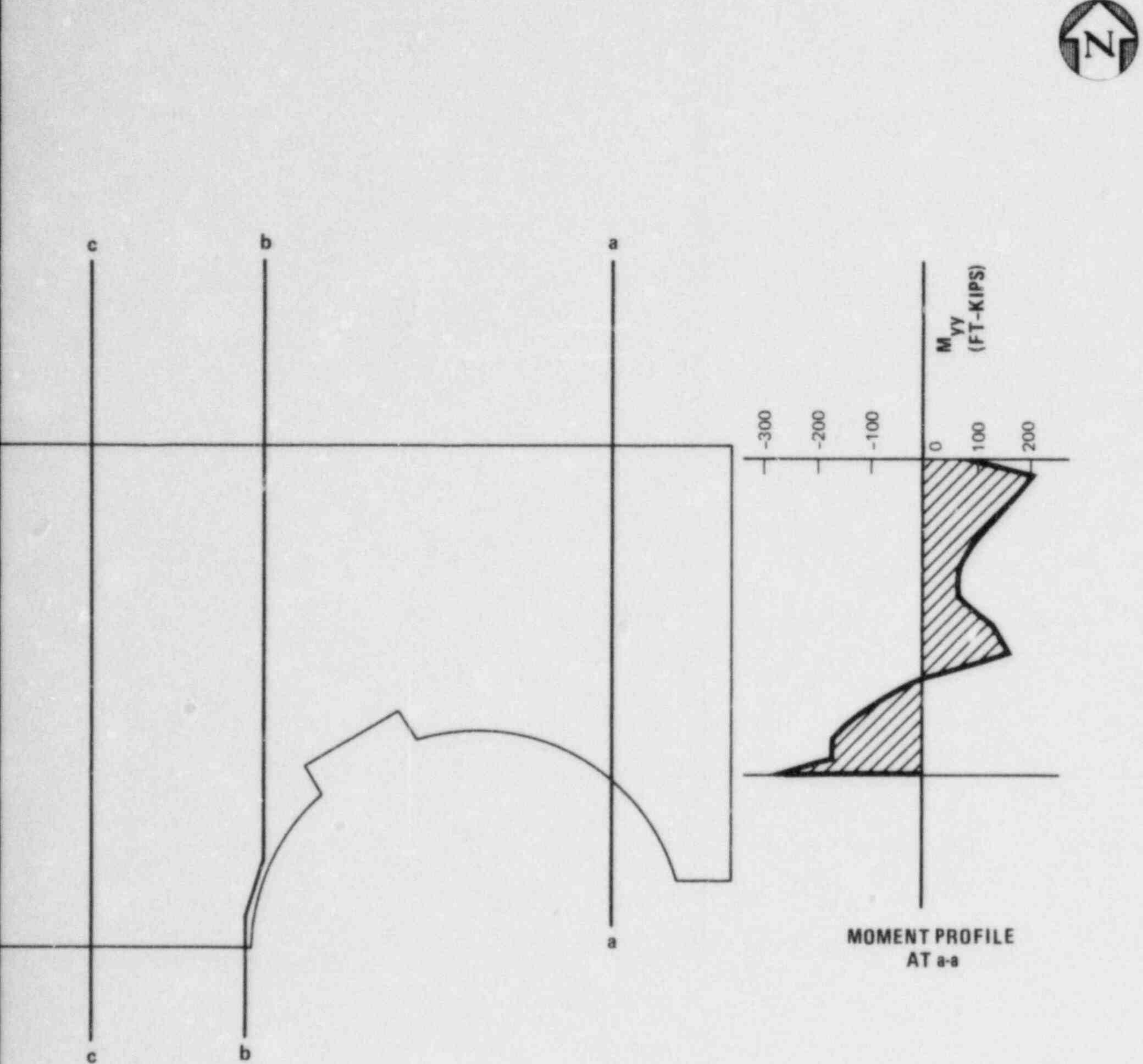


Figure 22
REPRESENTATIVE BASEMAT ANALYSIS RESULTS;
MOMENT DUE TO N-S SEISMIC RESPONSE

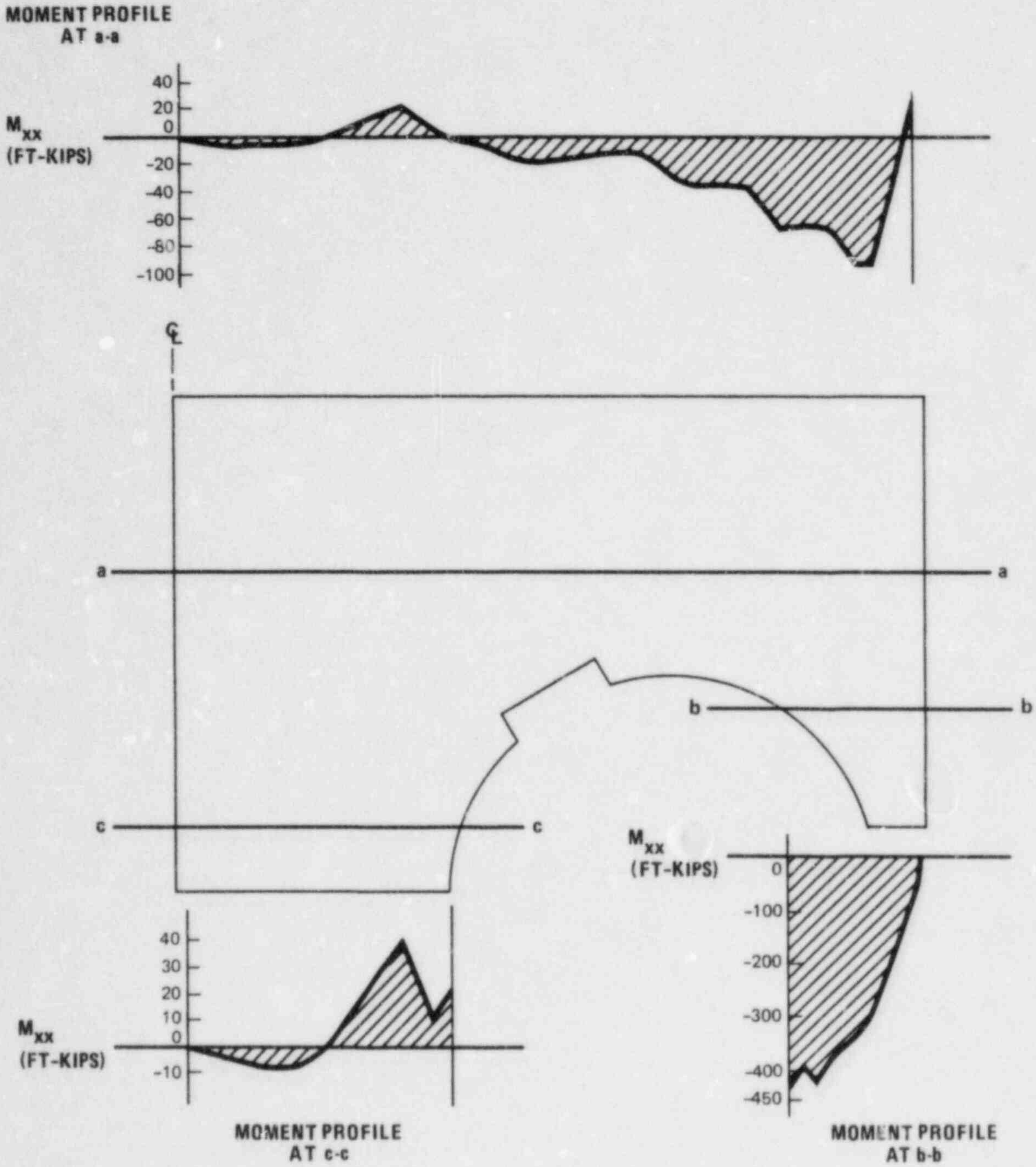
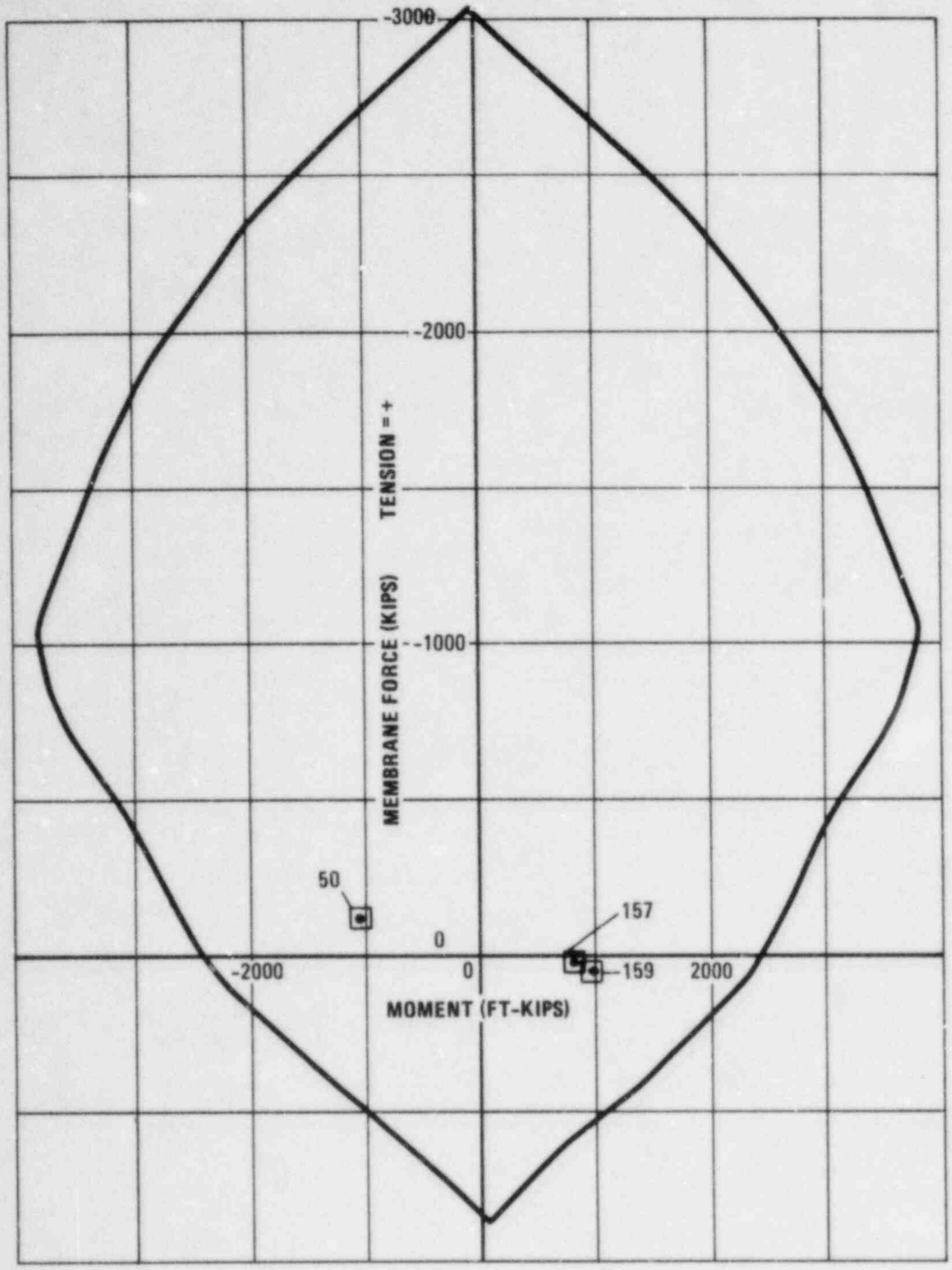
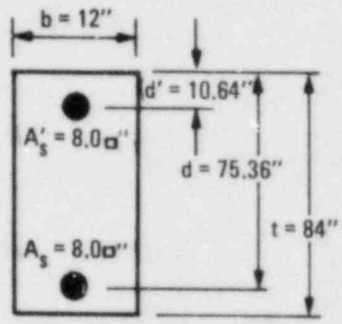


Figure 23
REPRESENTATIVE BASEMAT ANALYSIS RESULTS;
MOMENT DUE TO E-W SEISMIC RESPONSE

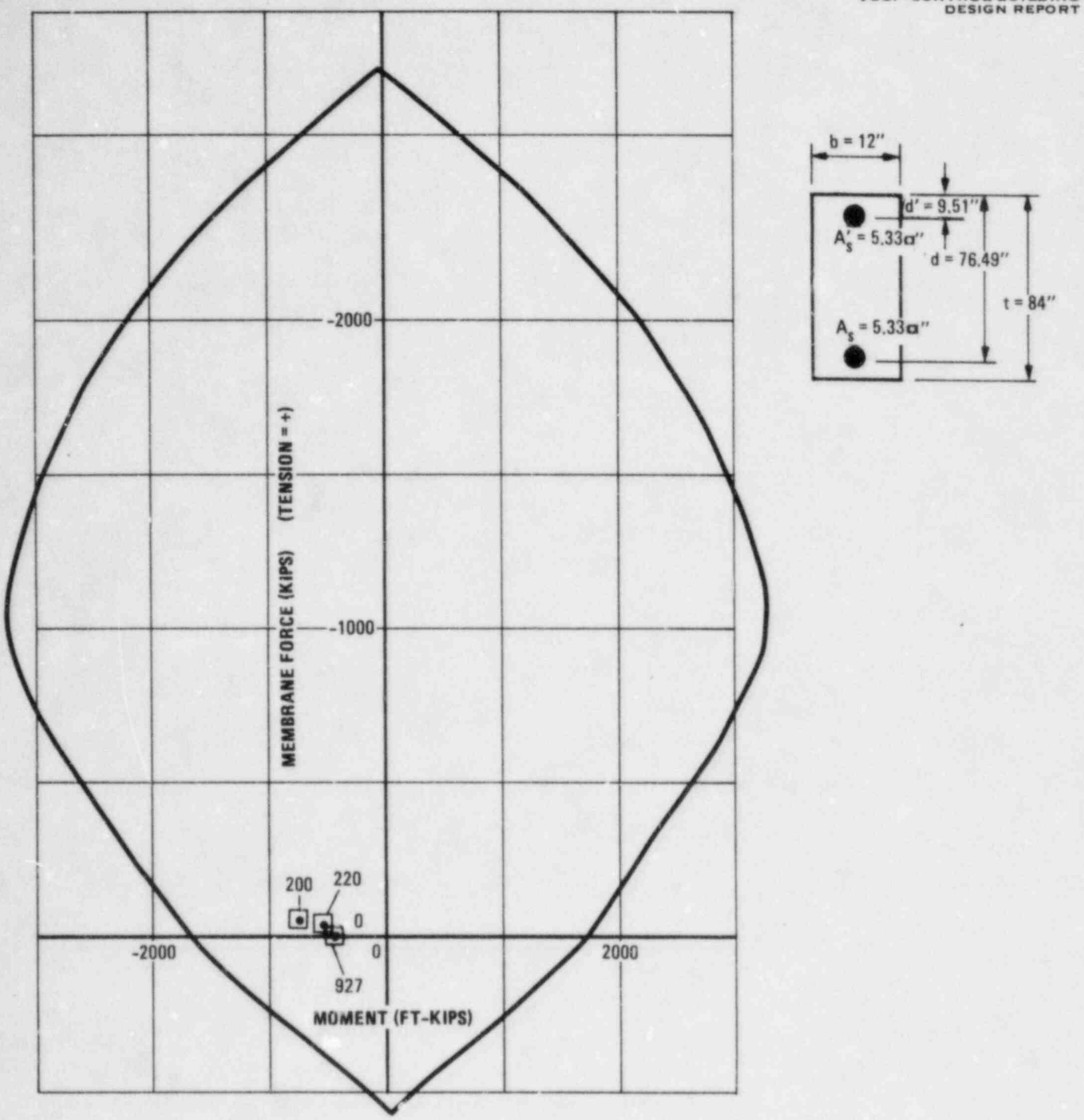




NOTE: NUMBERS INSIDE THE INTERACTION DIAGRAM REFER TO BASEMAT ELEMENT NUMBERS

ELEMENT NO.	LOAD COMBINATION EQUATION	DESIGN LOAD	
		P (KIPS)	M (FT-KIPS)
50	3	−103	−1095
157	3	28	896
159	3	40	959

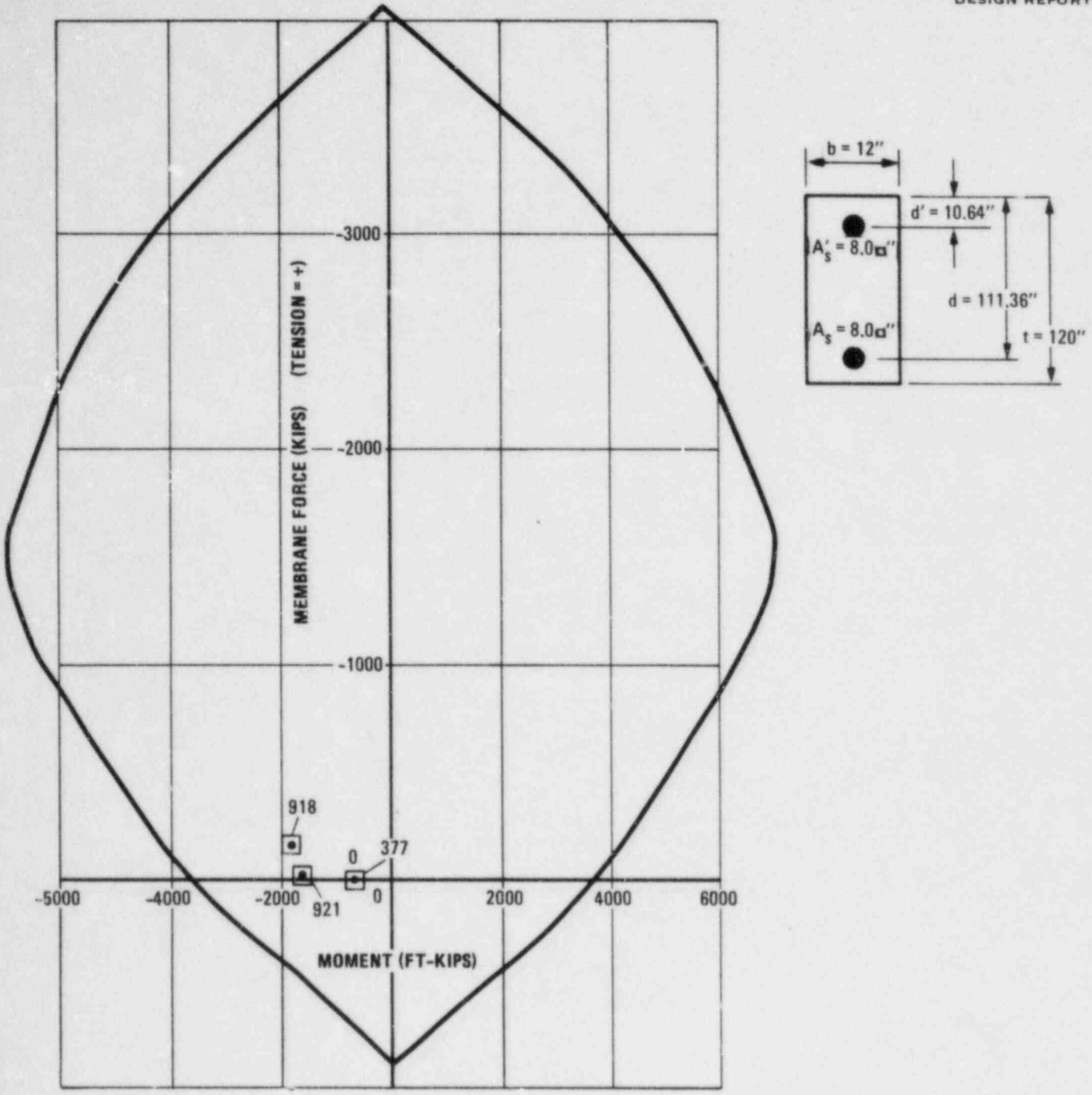
Figure 24
INTERACTION DIAGRAM FOR BASEMAT ZONE 1



NOTE: NUMBERS INSIDE THE INTERACTION DIAGRAM REFER TO BASEMAT ELEMENT NUMBERS.

ELEMENT NO.	LOAD COMBINATION EQUATION	DESIGN LOAD	
		P (KIPS)	M (FT-KIPS)
200	3	−48	−710
220	3	−31	−527
927	3	− 1	−480

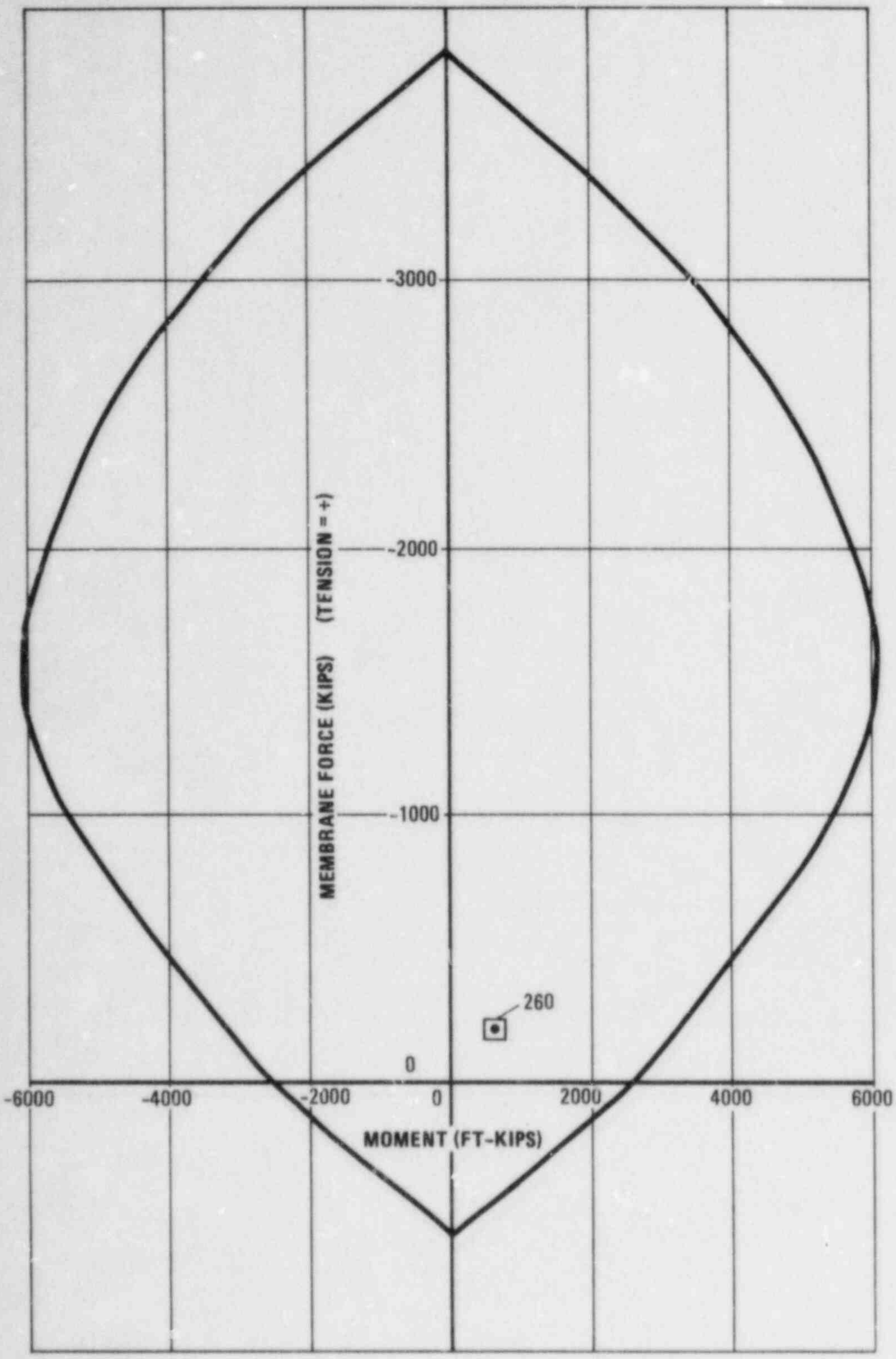
Figure 25
INTERACTION DIAGRAM FOR BASEMAT ZONE 2

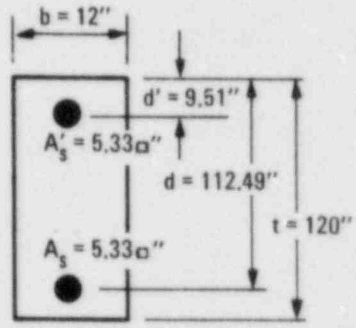


NOTE: NUMBERS INSIDE THE INTERACTION DIAGRAM REFER TO BASEMAT ELEMENT NUMBERS.

ELEMENT NO.	LOAD COMBINATION EQUATION	DESIGN LOAD	
		P (KIPS)	M (FT-KIPS)
377	3	4	−597
918	3	−161	−1792
921	3	−9	−1690

Figure 26
INTERACTION DIAGRAM FOR BASEMAT ZONE 3





NOTE: NUMBERS INSIDE THE INTERACTION DIAGRAM REFER TO BASEMAT ELEMENT NUMBERS.

ELEMENT NO.	LOAD COMBINATION EQUATION	DESIGN LOAD	
		P (KIPS)	M (FT-KIPS)
260	3	−195	585

Figure 27
INTERACTION DIAGRAM FOR BASEMAT ZONE 4

APPENDIX A

DEFINITION OF LOADS

APPENDIX A

DEFINITION OF LOADS

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

A.1 NORMAL LOADS

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

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

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

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

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

A.2 SEVERE ENVIRONMENTAL LOADS

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

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

A.3 EXTREME ENVIRONMENTAL LOADS

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

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

A.4 ABNORMAL LOADS

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

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

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

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

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

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

APPENDIX B

LOAD COMBINATIONS

APPENDIX B

LOAD COMBINATIONS

B.1 STEEL STRUCTURES

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

B.2 CONCRETE STRUCTURES

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

TABLE B.1[a]

STEEL DESIGN LOAD COMBINATIONS
ELASTIC METHOD

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

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

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

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

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

TABLE B.2[(a)(f)]

CONCRETE DESIGN LOAD COMBINATIONS STRENGTH METHOD

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

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

APPENDIX C

DESIGN OF STRUCTURES FOR TORNADO MISSILE IMPACT

APPENDIX C

DESIGN OF STRUCTURES FOR TORNADO MISSILE IMPACT

C.1 INTRODUCTION

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

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

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

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

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

C.1.1 Procedures

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

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

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

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

d. Determining structural response considering other concurrent loading conditions.

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

C.2 LOCAL EFFECTS

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

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

C.2.1 Reinforced Concrete Elements

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

C.2.2 Steel Elements

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

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

where:

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

E_k = missile kinetic energy (ft-lb).

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

V_s = missile striking velocity (ft/s).

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

---

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

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

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

where:

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

C.3 STRUCTURAL RESPONSE DUE TO MISSILE IMPACT LOADING

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

C.3.1 General

In an elastic impact, the missile and the structure deform elastically, remain in contact for a short period of time (duration of impact), and subsequently disengage due to the action of elastic interface restoring forces.

In a plastic impact, the missile or the structure or both may deform plastically or sustain permanent deformation or damage (local damage). Elastic restoring forces are small, and the missile and the structure tend to remain in contact after impact. Plastic impact is much more common in nuclear plant design than elastic impact, which is rarely encountered. For example, test data indicate that the impact from all postulated tornado-generated missiles can be characterized as a plastic collision.

If the interface forcing function can be defined or conservatively idealized (from empirical relationships or from theoretical considerations), the structure can be modeled mathematically, and conventional analytical or numerical techniques can be used to predict structural response. If the interface forcing function cannot be defined, the same mathematical model of the structure can be used to determine structural response by application of conservation of momentum and energy balance techniques with due consideration for type of impact (elastic or plastic).

In either case, in lieu of a more rigorous analysis, a conservative estimate of structural response can be obtained by first determining the response of the impacted structural element and then applying its reaction forces to the supporting structure. The predicted structural response enables assessment of structural design adequacy in terms of strain energy capacity, deformation limits, stability, and structural integrity.

Three different procedures are given for determining structural response: the force-time solution, the response chart solution, and the energy balance solution. The force-time solution involves numerical integration of the equation(s) of motion and is the most general method applicable for any pulse shape and resistance function. The response chart solution can be used with comparable results, provided the idealized pulse shape (interface forcing function) and the resistance function are compatible with the response chart. The energy balance solution is used in cases where the interface forcing function cannot be defined or where an upper limit check on structural response is desired. This method will consistently overestimate structural response, since the resisting spring forces during impact are neglected.

In defining the mass-spring model, consideration is given to local damage that could affect the response of the element. For concrete slab elements, the beneficial effect of formation of a fracture plane which propagates from the impact zone to the back of the slab (back face fracture plane) just prior to scabbing (reference 2) is neglected. The formation of this fracture plane limits the forces transferred to the surrounding slab and significantly reduces overall structural response. Since scabbing is to be precluded in the design, the structural response check is made assuming the fracture plane is not formed. It is recognized, however, that should the missile velocity exceed that for threshold of scabbing, structural response would be limited by this mechanism.

Therefore, the structural response is conservatively evaluated ignoring formation of the fracture plane and any reduction in response.

C.3.2 Structural Assessment

The predicted structural response enables assessment of design adequacy in terms of strain energy capacity, deformation limits, stability, and structural integrity.

For structures allowed to displace beyond yield (elasto-plastic response), a check is made to ensure that deformation limits would not be exceeded, by comparing calculated displacements or required ductility ratios with allowable values (such as those contained in table C-1).

C.4 REFERENCES

1. Gwaltney, R. C., "Missile Generation and Protection in Light-Water-Cooled Power Reactor Plants," ORNL NSIC-22, Oak Ridge National Laboratory, Oak Ridge, Tennessee, for the USAEC, September 1968.
2. Rotz, J. V., "Results of Missile Impact Tests on Reinforced Concrete Panels," Vol 1A, pp 720-738, Second Specialty Conference on Structural Design of Nuclear Power Plant Facilities, New Orleans, Louisiana, December 1975.

TABLE C-1

DUCTILITY RATIOS (Sheet 1 of 2)

Member Type and Load Condition	Maximum Allowable Value of Ductility Ratio (μ)
Reinforced Concrete	
Flexure(1):	
Beams and one-way slabs(2)	$\frac{0.10}{p-p'} \leq 10$
Slabs with two-way reinforcing(2)	$\frac{0.10}{p-p'} \leq 10$ or 30 (See 3 and 4)
Axial compression(1):	
Walls and columns	1.3
Shear, concrete beams and slabs in region controlled by shear:	
Shear carried by concrete only	1.3
Shear carried by concrete and stirrups	1.6
Shear carried completely by stirrups	2.0
Shear carried by bent-up bars	3.0
Structural Steel	
Columns(5) $\ell/r \leq 20$	1.3
$\ell/r > 20$	1.0
Tension due to flexure	10
Shear	10
Axial tension and steel plates in membrane tension(6)	$0.5 \frac{e_u}{e_y}$
Compression members not required for stability of building structures	10

TABLE C-1

DUCTILITY RATIOS (Sheet 2 of 2)

Notes:

(1) The interaction diagram used to determine the allowable ductility ratio for elements subject to combined flexure and axial compression is provided in figure C-1.

(2) p and p' are the positive and negative reinforcing steel ratios, respectively.

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

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

(5) ℓ/r is the member slenderness ratio. The value specified is for axial compression. For columns and beams with uniform moment the following value is used:

$$\frac{14 \times 10^4}{F_y \left(\frac{k\ell}{r}\right)^2} + \frac{1}{2} \leq 10$$

(6) e_u and e_y are the ultimate and yield strains. e_u shall be taken as the ASTM-specified minimum.

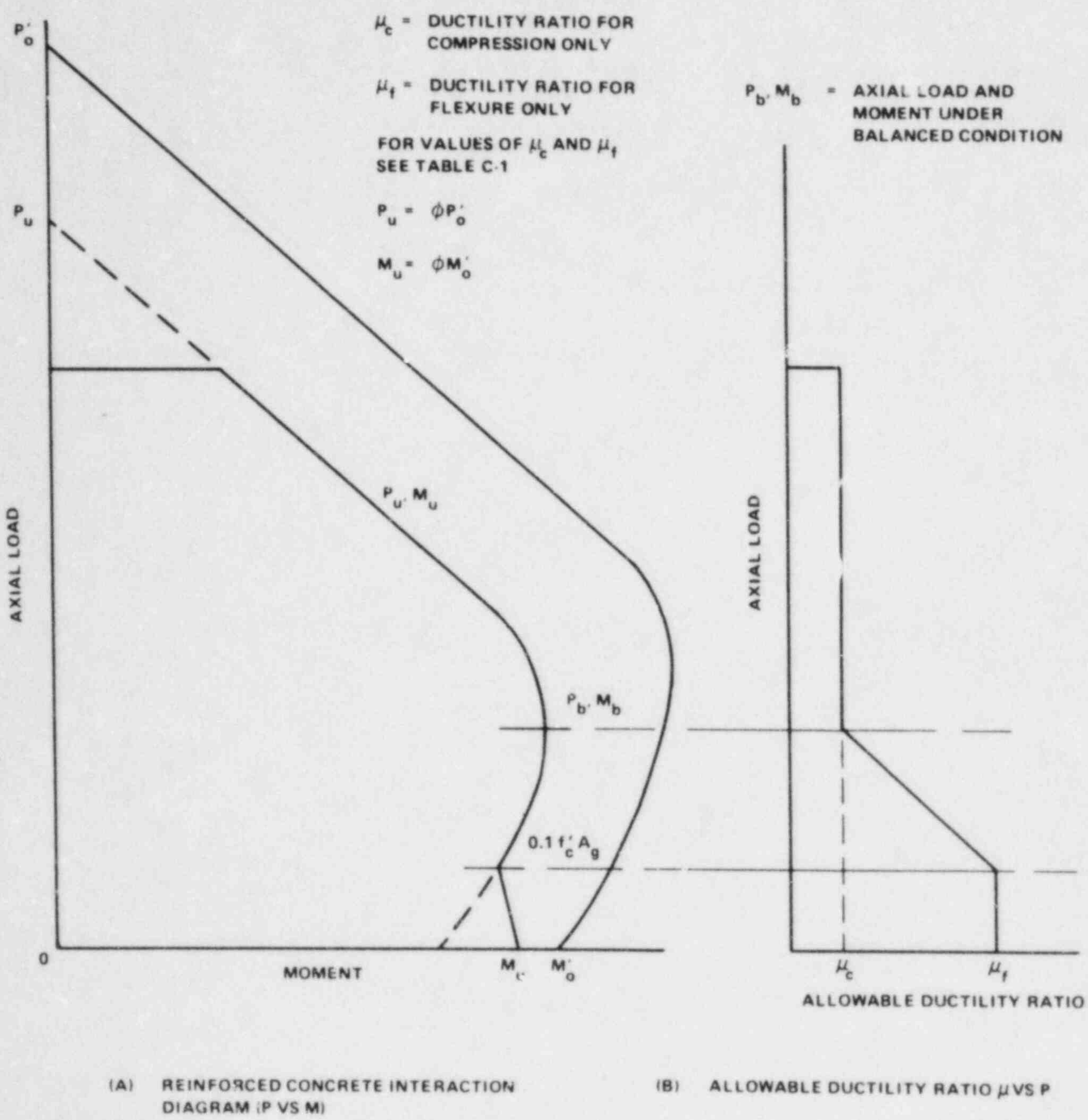


Figure C–1
MAXIMUM ALLOWABLE DUCTILITY RATIO
FOR REINFORCED CONCRETE SECTION
WITH BEAM–COLUMN ACTION