VOGTLE ELECTRIC GENERATING PLANT

GEORGIA POWER COMPANY

AUXILIARY BUILDING

DESIGN REPORT

Prepared

by

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1.0 INTRODUCTION

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

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

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

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

The purpose of this design report is to provide the Nuclear Regulatory Commission (NRC) with specific design and construction information for the auxiliary building, in order to assist in planning, and conducting a structural audit. Quantitative information is provided regarding the scope of the actual design computations and the final design results.

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

2.0 DESCRIPTION OF STRUCTURE

2.1 GENERAL DESCRIPTION

The auxiliary building is a seven-story, reinforced concrete, building common to the two-unit plant which houses the radioactive waste treatment facilities, heat exchangers, pumps, hot machine shop, cask handling crane, and heating, ventilating, and air-conditioning facilities, and other associated equipment. The building is a shear wall box type structure with floor and roof slabs acting as rigid diaphragms spanning between the walls. There are three stories above grade designated as levels 2 and 3 with level 1 being the grade level, and four subterranean stories designated as levels A, B, and C, with level D being the basemat level. The interior walls contain openings for doorways, piping, electrical cable trays, and heating, ventilating, and air conditioning (HVAC) duct systems. The exterior walls are solid except for openings at grade level for the railroad door, cask handling crane access to the fuel handling building, heat exchanger doors and openings at levels 1 and 2, and other small openings. There are openings in the roof slab at elevation 260'-0" for the HVAC air intake.

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 auxiliary building is located south of the fuel handling and containment buildings and north of the radwaste transfer building (see figure 1). A 5½-inch seismic gap is provided to separate the auxiliary building from these adjacent structures. The basemat is founded and placed directly on the marl bearing stratum at elevation 109'-3". The top of marl elevation is ±130'-0" and level D is keyed 20'-9" down into the marl. Category 1 backfill is placed against the south, east, and west walls of the building from the top of the marl up to the finished grade elevation. In addition, this Category 1 backfill is placed against the north wall from the top of the marl up to the bottom of the adjacent fuel handling and containment buildings basemats at elevations 154'-0" and 158'-6" respectively.

2.3 GEOMETRY AND DIMENSIONS

The auxiliary building plan dimensions are approximately 129 feet wide by 440 feet long. The level 4 roof slab in the center section of the building is at elevation 288'-2" and the level 3 roof slabs in the east and west wings of the building are at elevation 260'-0". Building plan and section drawings are shown in figures 2 through 6.

2.4 KEY STRUCTURAL ELEMENTS

The key structural elements in the auxiliary building include the roof and floor slabs, shear walls, and basemat.

The following is a brief description of the function and design considerations for these elements.

2.4.1 Roof and Floor Slabs

The auxiliary building has three main roof slabs, level 4 elevation 288'-2" at the center section cask handling crane bay, and level 3 elevation 260'-0" at the two wings. The roof slabs are 2 feet thick and the roof is flat. The slabs are structurally supported by walls and concrete beams. Part of the auxiliary building roof slab is used to form a portion of the containment access shafts. Several missile barriers are provided on the roof to protect openings provided for main steam and feedwater accident exhaust venting, HVAC air intake, and Category 1 piping.

The main floor slabs are at the following elevations; level C - 143'-6", level B - 170'-6", level A - 195'-0", level 1 - 220'-0", and level 2 - 240'-0". They vary from 2 feet to 3 feet 8 inches thick. The floor slabs are structurally supported by walls, concrete beams, and columns.

The level 1 floor slab of the center section cask handling crane bay has a railroad access for shipping and receiving new and spent fuel casks.

2.4.2 Shear Walls

All walls in the auxiliary building are designed as shear walls contributing to the lateral load carrying capacity of the structure, unless the contribution of a given wall to story rigidity is small.

The interior shear walls vary from 2 feet to 4 feet thick, with the thicker walls located on the subterranean levels.

The exterior shear walls are 6 feet thick at level D, 5 feet at C, 4 feet at levels B and A, 3 feet at levels 1 and 2, and 2 feet minimum at level 3.

2.4.3 Basemat

The auxiliary building basemat is approximately 129 feet wide by 440 feet long, and has a uniform thickness of 10 feet. Top of the basemat is elevation 119'-3". The basemat contains several sumps that are lined with 1/4-inch-thick stainless steel plate and unlined concrete pits. Top of the sumps and pits are of various elevations approximately 5 feet lower than the top of basemat. The basemat is stiffened by the level D shear walls that divide this level into many room compartments. These rooms house systems composed of mechanical and electrical equipment that include tanks, pumps, and electrical switchgear.

2.5 MAJOR EQUIPMENT

Major systems with equipment housed in the auxiliary building include the chemical and volume control system (CVCS), emergency core cooling system (ECCS), residual heat removal (RHR) system, heating, HVAC systems, and other associated equipment. Much of the equipment associated with these systems is serviced by an overhead monorail system. Structural steel platforms are provided to access valves in the associated piping systems. Electrical cable tray, HVAC duct, and piping systems are supported from structural steel platforms, or structural steel used to support metal decking for shoring purposes during construction, or have an independent structural steel support from embeded plates on walls and slabs.

2.6 SPECIAL FEATURES

2.6.1 Tornado Missile Barriers

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

2.6.2 Walls and Corbels Supporting the Cask Handling Crane

New and spent fuel casks are transported to and from the fuel handling building by the cask handling crane. The cask handling crane bay is located at the centerline of the auxiliary building. For wall and slab layout in this area refer to figure 6.
The cask handling crane is supported at elevation 264'-7" by a continuous concrete corbel and beam 5 feet wide by 6 feet 7 inches deep that is monolithic with the 2-feet-thick crane supporting concrete walls. The supporting walls are laterally stiffened by floor and roof slabs at levels 1, 2, and 3.

Additionally the walls are buttressed with 4-foot-square concrete pilasters that are located approximately 20 feet on center. The pilasters are monolithic with the crane supporting walls and extend from the level 1 floor slab up to the corbel and beam to provide column-like support.

2.6.3 Main Steam Isolation Valve (MSIV) Room Walls and Slabs

The MSIV room is located south of each containment building at level 1. The main steam pipe lines enter the auxiliary building from the tunnels at the east and west sides of the building for units 1 and 2 respectively, and they exit the building at the north exterior wall. The main steam lines are routed through and restrained by the 4 foot and 2 foot-thick five-way restraint walls. Several pipe whip restraints are provided to prevent the pipes from whipping against the walls and slabs during a postulated pipe break accident. Walls and slabs in this area vary from 2 feet to 3 feet thick. Structural steel platforms and monorails are provided to service the main steam isolation valves. Refer to figure 6.

3.0 DESIGN BASES

3.1 CRITERIA

The following documents are applicable to the design of the auxiliary building.

3.1.1 Codes and Standards

A. American Concrete Institute (ACI), Building Code Requirements for Reinforced Concrete, ACI 318-71, including 1974 Supplement.

B. American Institute of Steel Construction (AISC), Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings, adopted February 12, 1969, and Supplements No. 1, 2, and 3.

3.1.2 Regulations

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

3.1.3 General Design Criteria (GDC)

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

3.1.4 Industry Standards

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

3.2 LOADS

The auxiliary building is designed for all credible loading conditions. The loads are listed and defined in Appendix A.

3.2.1 Normal Loads

3.2.1.1 Dead Loads (D)

These loads include the weight of concrete walls, roof and floor slabs, structural steel platforms, beams and columns, piping, conduits, cable tray, HVAC ducts, supports, mechanical and electrical equipment. The dead loads used to account for equipment, mechanical, electrical, and piping loads are listed below by level:

Level	Load (psf)
3	100
2	300
1	300

Level	Load (psf)
1	Actual cask carrier loading at the railroad entrance
1	Drum storage area-1000
A	300
B	300
C	300
D	200

3.2.1.2 Live Loads (L)

These loads include occupancy loads, soil pressures, hydrostatic pressures due to groundwater, movable equipment loads, and precipitation loads. The live loads used for design are listed below. Live loads due to soil pressure distribution acting on the exterior walls are shown pictorially in figure 7.

The minimum roof live load of 30 psf envelops the effects of occupancy, snow, and 100-year rainwater ponding loads.

• Roof live load	30 psf
• Floor live load in areas not occupied by equipment	100 psf
• Level 1 railroad entrance (cask, skid and rail car)	256 kips total
• Monorails	Lift capacity of the hoist plus impact

3.2.1.3 Operating Thermal Loads (T_o)

The operating temperature inside the auxiliary building ranges from 40°F to 100°F.

3.2.1.4 Operating Pipe and Equipment Loads (R_o)

The pipe and equipment reactions during normal or shutdown condition are included in the 100 psf to 300 psf of the design dead loads (D).

3.2.2 Severe Environmental Loads

3.2.2.1 Operating Basis Earthquake, OBE (E)

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

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

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

The dynamic lateral earth pressures acting on the auxiliary building due to the OBE are computed by the Mononobe-Okabe method of analysis for dynamic earth pressures in dry cohesionless materials. Figure 7 shows the dynamic lateral earth pressure distribution acting on the exterior walls of the building.

3.2.2.2 Design Wind (W)

The auxiliary building is designed for a wind velocity of 110 mph which is based on a wind speed 30 feet above ground (reference 1). Exposure C, applicable for flat open country, is used. The effective velocity pressure profile for the 110-mph wind is shown in figure 8.

3.2.3 Extreme Environmental Loads

3.2.3.1 Safe Shutdown Earthquake, SSE (E')

Based on the plant site geologic and seismologic investigations, the peak ground acceleration for SSE is established as 0.20g. The free-field response spectra and the development of horizontal and vertical floor accelerations and in-structure response spectra of the basemat, floor and roof slab elevations are discussed in the Seismic Analysis Report. Table 1 shows the horizontal and vertical floor accelerations.

The SSE damping values, as percentages of critical, applicable to the auxiliary building design are as follows.

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

The dynamic lateral earth pressures acting on the auxiliary building due to the SSE are computed by the Mononobe-Okabe method of analysis for dynamic earth pressures in dry cohesionless materials. Figure 7 shows the dynamic lateral earth pressure distribution acting on the exterior walls of the building.

3.2.3.2 Tornado Loads (W_t)

Loads due to the design tornado include wind pressures, atmospheric pressure differentials, and tornado missile strikes. The design tornado parameters, which are in conformance with the Region I parameters defined in Regulatory Guide 1.76, are as follows:

- Rotational tornado speed — 290 mph
- Translational tornado speed — 70 mph maximum, 5 mph minimum
- Maximum wind speed — 360 mph
- Radius of tornado at maximum rotational speed — 150 feet

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

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

The auxiliary building is also 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 degrees from the horizontal use the listed horizontal velocities. Those trajectories greater than 45 degrees use the listed vertical velocities.

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

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

The tornado effective velocity pressure profile used in the design (see figure 8) 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.

3.2.3.3 Probable Maximum Precipitation, PMP (N)

The load due to probable maximum precipitation is applied to auxiliary building roof areas.

Special roof scuppers are provided with sufficient capacity to ensure that the depth of ponding water due to the PMP rainfall does not exceed 18 inches. This results in an applied PMP load of 94 psf.

3.2.3.4 Blast Load (B)

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

3.2.4 <u>Abnormal Loads</u>

3.2.4.1 Pressure Load (P_a)

The differential uniform pressure load due to a postulated pipe break accident in the main steam and feedwater line areas vary in magnitude, with a maximum differential pressure load of 9.7 psi.

The walls and slabs that are designed for a main steam and feedwater line break are indicated in figures 5 and 6.

The differential uniform pressure loads due to a postulated pipe break accident outside the main steam and feedwater line areas vary in magnitude, with a maximum differential pressure load of 8 psi.

3.2.4.2 Thermal Loads under Accident Conditions (T_a)

The main steam and feedwater line area walls and slabs are designed for the thermal effects due to a maximum room temperature of 320°F. Walls and slabs that are designed for main steam and feedwater line thermal accident conditions are indicated in figures 5 and 6

3.2.4.3 Pipe Reactions under Thermal Conditions (R_a)

Pipe reactions under thermal conditions generated by the postulated pipe break accident are considered for major supports in the main steam and feedwater line areas. The reactions vary in magnitude with a maximum pipe support reaction of 315.5 kips. Walls and slabs that are designed for main steam and feedwater line pipe reactions under thermal conditions are indicated in figures 5 and 6.

3.2.4.4 Pipe Rupture Loads (Y_r, Y_j, Y_m)

Loads on walls and slabs generated by the reaction of a ruptured high-energy pipe are most significant in the main steam and feedwater line areas. The reactions vary in magnitude with the maximum loads occurring in the five-way restraint walls. In addition to the five-way restraint reactions, pipe whip restraint reactions due to ruptured high-energy pipes are considered for wall and slab design. The main steam and feedwater line five way restraint walls are shown in figures 5 and 6.

Jet impingement loads are considered for the design of walls and slabs. The loads vary in magnitude, with a maximum jet impingement load of 1,134 kips occurring in the main steam and feedwater line area. The main steam and feedwater line area, where jet impingement loads occur, are indicated in figures 5 and 6.

3.3 LOAD COMBINATIONS AND STRESS/STRENGTH LIMITS

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

3.4 MATERIALS

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

3.4.1 <u>Concrete</u>

• Compressive strength	f'_c = 5 ksi
• Modulus of elasticity	E_c = 4,285 ksi
• Shear modulus	G = 1,780 ksi
• Poisson's ratio	ν = 0.17 - 0.25

High strength concrete used in the main steam and feedwater areas:

- Compressive strength — f'_c = 6 ksi
- Modulus of elasticity — E_c = 4,696 ksi
- Shear modulus — G = 1,955 ksi
- Poisson's ratio — ν = 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

3.4.3.1 ASTM A36

- Minimum yield stress — F_y = 36 ksi
- Minimum tensile strength — F_{ult} = 58 ksi
- Modulus of elasticity — E_s = 29,000 ksi

3.4.3.2 ASTM A500, Grade B: Structural Tubing

- Minimum yield stress — F_y = 46 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 Anchor Bolts and Headed Anchor Studs

3.4.5.1 ASTM A36

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

3.4.5.2 ASTM A108

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

3.4.5.3 ASTM A307

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

3.4.6 Foundation Media

3.4.6.1 General Description

See section 2.2

3.4.6.2 Category 1 Backfill

- Moist unit weight — γ_m = 126 pcf
- Saturated unit weight — γ_t = 132 pcf
- Shear modulus

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

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

3.4.6.3 Modulus of Subgrade Reaction

- Static 60 kcf
- Dynamic 85 kcf

3.4.6.4 Net Bearing Capacities

- Ultimate 63.7 ksf
- Allowable static 21.2 ksf
- Allowable dynamic 31.9 ksf

4.0 STRUCTURAL ANALYSIS AND DESIGN

This section provides the methodologies employed to analyze the auxiliary building and to design its key structural elements, using the applicable loads and load combination 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.

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

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

For manual analyses, the analysis techniques, boundary conditions, and application of loads are described 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.

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 auxiliary building.

All other load combinations, including the effects of abnormal loads and tornado loads, are evaluated where applicable on a local area basis (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 auxiliary building consist of concrete slabs and beams that support the applied vertical loads, walls and columns that support the slabs and beams, and the basemat which transmits the loads from the walls and columns to the foundation medium. Representative vertical load carrying elements are identified in figure 6.

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 beams 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.

4.3 LATERAL LOAD ANALYSIS

The lateral load carrying elements of the auxiliary 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 figure 2.

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 lcads are applied at each slab level, as appropriate. The design horizontal earthquake load at each level of the building is obtained by mulitiplying 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 is obtained from the lateral earth pressure with due consideration to the seismic effects and the surcharge effects from the adjacent structures (i.e., fuel handling, control, radwaste transfer, and containment buildings). 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 rigidities of the walls.

For a given shear wall the shear due to story shear (direct shear) and shear due to torsional moment (torsional shear) are combined at a given level to obtain the total design shear load.

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 forming plan (elevation 260'-0") of the auxiliary building is presented in figure 6, showing the structural elements provided for vertical and lateral support of the slab panels, which consist of load bearing shear walls.

Based on the panel configuration, the relative stiffness of the supporting members and the type of fixity provided, slab panels are analyzed for one-way or two-way slab action using appropriate boundary conditions and standard beam and plate formulas.

Equivalent uniformly distributed loads are applied to slab panels. The design vertical earthquake loads for slab panels at a given level are obtained by multiplying the effective mass from the applied loading (including its 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 auxiliary building floor accelerations and response spectra are insignificant, as long as the fundamental floor system frequency is equal to or higher than 8 cps. The evaluation of the floor systems in the auxiliary 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 consideration 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. See figure 9.

As appropriate, additional reinforcement is provided in the slab adjacent to large floor openings.

4.5.2 Design Results

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

4.6 SHEAR WALLS

4.6.1 Analysis and Design Methodology

The location of shear walls is identified in figure 2 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 under lateral load analysis in sections 4.2 and 4.3. The in-plane design loads include axial loads resulting from the overturning moment.

The out-of-plane design loads are considered using the soil pressure distribution 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 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 out-of-plane shear forces resulting from the out-of-plane design loads. At governing sections, the combined effects of in-plane overturning moment and axial loads, and the out-of-plane loads are evaluated.

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

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

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

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

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

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

D. The reinforcement provided for the in-plane loads is evaluated for the combined effects of in-plane and out-of-plane loads, and additional reinforcing is added if necessary.

4.6.2 Design Results

The design results for governing load combinations are presented in table 4 for representative shear walls, and typical design details are shown in figure 10.

4.7 BASEMAT

4.7.1 Analysis Methodology and Computer Model

The auxiliary building basemat is analyized using a finite-element model with the structural design language computer program (STRUDL), which is a general purpose computer program for finite-element analysis. 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 is modeled using plate and membrane elements, and spring-type boundary conditions are used to characterize the stiffness effect of the soil. The vertical stiffness of each soil spring is determined by multiplying the nodal tributary area by the modulus of subgrade reaction.

Plate bending elements (pure bending only) are superimposed on membrane elements to model the structural shear walls in the first story above the basemat (to represent the stiffness interaction effects at the wall/basemat junction). The superimposed bending and membrane elements simulate in-plane and out-of-plane wall stiffness properties. Plate bending elements are used to model the basemat. There are a total of 2,934 plate bending elements, 1,073 membrane elements, and 1,932 spring-type boundary conditions used to model the basemat.

Figure 11 shows the computer plots of the basemat model indicating node number and element number for the portion of the basemat modeled. Only one half of the basemat is modeled taking advantage of the symmetry of the auxiliary building in the east-west direction about the building centerline at column line A_{10}.

The boundary conditions for the basemat are modeled as follows: spring type boundary conditions, representing the vertical translational soil stiffness are attached to each basemat node; and plate bending elements used to model the basemat floor have both in-plane east-west and north-south horizontal translational degrees of freedom fixed at each node, and the remaining degrees of freedom are released. Along the axis of building symmetry (nodes 1 through 31), symmetrical boundary conditions are used for vertical and north-south loads, and anti-symmetrical boundary conditions are used for east-west loads. The shear wall plate bending elements have the out-of-plane rotational degree of freedom along the axis parallel to the wall fixed at the top node of the wall to account for the slab continuity with the wall. All remaining degrees of freedom are released. The shear wall membrane elements have all degrees of freedom released.

4.7.2 Application of Loads

The magnitude and distribution of loads applied to the basemat model are consistent with the cumulative results of the vertical and horizontal 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 cumulative horizontal and vertical loads and accompanying overturning moments, obtained from the shear wall analysis of the structure (as described in section 4.3) are used to compute the elastic stresses at the base of the shear walls using the principle $P/A \pm M/S$. The resulting linear triangular pressure distribution is divided into a series of stepped uniform loads that are applied to the basemat floor plate bending elements over a two-element width at the wall-basemat junction. Equipment and floor occupancy loads are applied as concentrated nodal forces and uniform pressure distribution on the top of the basemat.

4.7.3 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 using manual calculations. The design consists of determining the governing bending moments in different basemat zones and computing the area and spacing of steel reinforcing required to resist bending. Basemat shear is computed using the design moments from the finite element analysis and determining the moment gradient between adjacent elements. An independent manual basemat shear calculation is performed considering the mat as a beam on an elastic foundation to ensure identification of the governing basemat design shear. The basemat shear stresses are checked and shear reinforcement is provided where required.

4.7.4 Design Results

Representative results of the basemat analysis are provided in figure 12. In addition, table 5 shows the design results of critical elements with maximum moment. Representative design details are provided in figure 13.

5.0 MISCELLANEOUS ANALYSIS AND DESIGN

As described in section 4.1, the auxiliary 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 auxiliary building is evaluated. This section describes these analyses and significant special provisions employed in the auxiliary building design.

5.1 STABILITY ANALYSIS

The overall stability of the auxiliary 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 significantly 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, defined as the volume of the ground water 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 6.

5.2 TORNADO LOAD EFFECTS

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

Controlling roof and exterior wall panels are evaluated for tornado load effects, and the localized response is combined with the analysis results of the overall structural response, as applicable, to confirm that design integrity is maintained.

Additional reinforcing steel is provided, if necessary, to satisfy design requirements in accordance with the ACI 318 Code. In addition, barriers are provided for the openings in the exterior walls or roofs unless the systems or components located in the exterior rooms are nonsafety-related. In this case, the interior walls and slabs are treated as barriers for the safety-related systems or components located in the interior rooms. Any openings in the exterior walls or slabs and the interior walls or slabs that may be susceptible to missile entry are evaluated to ensure that no safety-related systems or components are located in a potential path of the missile.

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

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

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

5.3 ABNORMAL LOADS EFFECTS

Abnormal loads generated by a postulated high-energy pipe break accident occur in the main steam isolation valve (MSIV) and main feedwater isolation valve (MFIV) areas, and adjacent areas which are vented.

The MSIV/MFIV areas subjected to the abnormal loads consists of the break-exclusion zones between the containment building and the five-way restraints. The MSIV areas are located south of the containment building between column lines A_{17} and A_{18} for Unit 1, and A_2 and A_3 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, as per the design methodology described in section 4, is evaluated for the governing design forces resulting from the effects of abnormal loads, using the OPTCON computer program. OPTCON calculates the thermal moment, considering the relaxation effects of concrete-cracking and reinforcement-yielding. For each load combination analyzed, the state of stress and strain is determined before the thermal load is applied. Then the thermal moment is approximated based upon an iterative approach which considers equilibrium and compatibility conditions. The final force-moment set (which includes the cracked section final thermal moment) is checked to verify that it falls within the Code allowable interaction diagram.

5.4 WALLS AND CORBEL SUPPORTING THE CASK HANDLING CRANE

The wall and corbel supporting the cask handling crane in the auxiliary building are shown in figure 14. The wall and corbel are designed and detailed in accordance with the provisions of the ACI 318 Code. The concentrated cask handling crane truck loads are applied eccentrically at the rail centerline to the corbel shelf which is monolithic with the wall. The corbel shelf is designed to transfer the moment resulting from the load eccentricity and shear to the supporting wall. Level 3 walls and slab at the corbel location are analyzed like a frame for the out-of-plane moment resulting from the wall to rail eccentricity.

The walls and slab are designed for the appropriate applied moment and axial load.

Design results are shown in table 8.

5.5 FOUNDATION BEARING PRESSURE

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

6.0 CONCLUSION

The analysis and design of the auxiliary 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 of Nuclear Power Plants, Bechtel Power Corp., November 1974.

TABLE 1

AUXILIARY BUILDING SEISMIC ACCELERATION VALUES

Level	Elevation	Floor Accelerations (g's)(1)					
		SSE			OBE		
		E-W	N-S	Vert.	E-W	N-S	Vert.
Level D	119'-3"	0.18	0.19	0.29	0.11	0.12	0.18
Level C	143'-6"	0.19	0.19	0.29	0.12	0.12	0.19
Level B	170'-6"	0.21	0.22	0.30	0.13	0.14	0.19
Level A	195'-0"	0.22	0.25	0.30	0.14	0.16	0.19
Level 1 (grade level)	220'-0"	0.24	0.28	0.30	0.15	0.18	0.20
Level 2 East Wing	240'-0"	0.26	0.33	0.36	0.16	0.21	0.23
Level 2 West Wing	240'-0"	0.26	0.33	0.36	0.16	0.21	0.23
Level 3 East Wing	260'-0"	0.26	0.34	0.36	0.17	0.22	0.23
Level 3 West Wing	260'-0"	0.26	0.34	0.36	0.17	0.22	0.23
Level 4	288'-2"	0.38	0.36	0.36	0.25	0.24	0.23

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

TABLE 2

TORNADO MISSILE DATA

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

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

TABLE 3

DESIGN RESULTS OF FLOOR SLABS (Sheet 1 of 2)

Structural Element	Governing$^{(1)}$ Load Combination Equation	A_s Required	A_s Provided
Level 3 Roof Slab			
Slab between A_{11} and A_{12} from A_A to A_D and slab between A_{11} and $A_{12.5}$ from A_E to A_G	3	1.88 in.^{2} E-W 0.52 in.^{2} N-S$^{(2)}$	2.0 in.^{2} E-W 1.0 in.^{2} N-S
Slabs between A_{16} and A_{19} from A_{C3} to A_G	3	1.95 in.^{2} N-S 1.37 in.^{2} E-W	2.0 in.^{2} N-S 2.0 in.^{2} E-W

(1) Load combination equations correspond to equations in Appendix B.

(2) Governed by minimum Code reinforcing requirements.

TABLE 3

DESIGN RESULTS OF FLOOR SLABS (Sheet 2 of 2)

Structural Element	Governing[1] Load Combination Equation	Design Force or A_s Required	Design Capacity or A_s Provided
Slab between A_{17} and 5' east of A_{18} north of A_{C3}	9	1.60 in.^{2} E-W 0.52 in.^{2} N-S[2]	2.08 in.^{2} E-W 1.00 in.^{2} N-S
<u>Level 1 Floor Slab</u>			
Slab between A_9 and A_{11} from A_C to A_F	3	$M_u = 143.6^{ft\text{-}k}/ft$ $V_u = 424^k$	$\phi M_n = 180^{ft\text{-}k}/ft$ $\phi V_n = 711^k$
Slab between A_9 and A_{11} from A_F to A_G	3	$M_u = 73^{ft\text{-}k}/ft$ $V_u = 440^k$	$\phi M_n = 112^{ft\text{-}k}/ft$ $\phi V_n = 755^k$

(1) Load combination equations correspond to equations in Appendix B.

(2) Governed by minimum Code reinforcing requirements.

TABLE 4

DESIGN RESULTS OF SHEAR WALLS (Sheet 1 of 3)

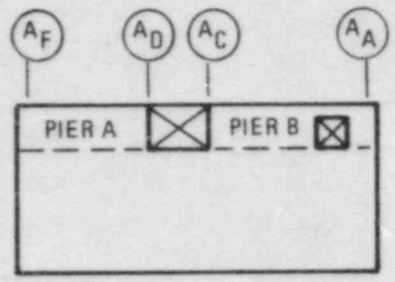
Structural Element	Governing[1] Load Combination Equation	Design Force	Design Capacity[2]
Exterior Wall			
Level D wall at column line A_{19}	3 In-plane	$M_u = 2,903,283^{ft\text{-}k}$ $\frac{V_u}{\phi} = 61,653^{k}$ $N_u = 22,272^{k}$	$\phi M_n = 3,087,164^{ft\text{-}k}$ $V_n = 72,637^{k}$
Level D wall column line A_{19}	3 Combined in-plane and out-of-plane loads	$M_u = 256.5^{ft\text{-}k}/ft$ interior face $M_u = 454.8^{ft\text{-}k}/ft$ exterior face $\frac{V_u}{\phi} = 78.4\ ^{k}/ft$ $N_u = 172.7\ ^{k}/ft$ tension	$\phi M_n = 320^{ft\text{-}k}/ft$ interior face $\phi M_n = 560^{ft\text{-}k}/ft$ interior face $V_n = 100.8\ ^{k}/ft$
Interior Walls In-Plane Loads			
Level D wall column line A_{11} between A_A and A_C	3	$M_u = 589,576^{ft\text{-}k}$ $\frac{V_u}{\phi} = 8,084^{k}$ $N_u = 9,716^{k}$	$\phi M_n = 813,320^{ft\text{-}k}$ $V_n = 8,158^{k}$

(1) Load combination equations correspond to equations in Appendix B.

(2) Design capacity is computed keeping N_u constant at the design force.

TABLE 4

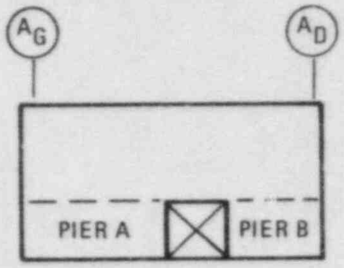
DESIGN RESULTS OF SHEAR WALLS (Sheet 2 of 3)

Structural Element	Governing(1) Load Combination Equation	A_s Required		A_s Provided	
		Vertical	Horizontal	Vertical	Horizontal
Level D wall column line A_{14} between A_A and A_F					
PIER A	3	321 in.^{2} total	2.24 in.^{2}/ft	561 in.^{2} total	3.12 in.^{2}/ft
PIER B	3	197 in.^{2} total	2.14 in.^{2}/ft	215 in.^{2} total	3.12 in.^{2}/ft

(1) Load combination equations correspond to equations in Appendix B.

TABLE 4

DESIGN RESULTS OF SHEAR WALLS (Sheet 3 of 3)

Structural Element	Governing[1] Load Combination Equation	A_s Required		A_s Provided	
		Vertical	Horizontal	Vertical	Horizontal
Level D wall column line A_{11} between A_D and A_G					
PIER A	3	145 in.^{2} total	2.64 in.^{2}/ft	417 in.^{2} total	3.12 in.^{2}/ft
PIER B	3	41 in.^{2} total	0.86 in.^{2}/ft	107 in.^{2} total	1.58 in.^{2}/ft

(1) Load combination equations correspond to equations in Appendix B.

TABLE 5

DESIGN RESULTS OF BASEMAT ELEMENTS

Structual Ele:	~overning(1) Load Combination Equation	Design Force [M_u (y-axis) in.-k]	Design Capacity (ϕM_n in.-k)
No. 1773	3	29,004	54,557
No. 21	3	111,540	117.113
No. 650	3	77,532	99,179
No. 1477	3	41,148	55,645
No. 460	3	43,104	55,644

(1) Load combination equations correspond to equations in Appendix B.

TABLE 6

FACTORS OF SAFETY FOR STRUCTURAL STABILITY

Load Combination[1][3]	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	1.7	See note (2)	1.5	1.7	-	-
D + H + E'	1.1	1.3	273	1.1	1.3	-	-
D + F'	-	-	-	-	-	1.1	2.8

(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 7

TORNADO MISSILE ANALYSIS RESULTS(1)

Panel Description and Location	Panel Size			Computed Ductility Ratio	Allowable Ductility Ratio
	Length (ft)	Width (ft)	Thickness (ft)		
Level 2 exterior wall along column line A_G between A_4 to A_9 (south wall)	107.5	20	3	4.5	10
Level 1 exterior wall along column line A_G, wall panel between railroad door and personnel door (A_9 to A_8)	20	10.8	3	4.4	10
Level 2 interior wall along column line A_{17} between A_{C3} and E_H	42	20	2	1.2	10
Level 3 roof slab panel between A_{17} and A_{18} north of A_{C3}	21.5	12.75	2	2.5	10
Level 3 roof slab panel between A_A and $A_{B.5}$ and A_{11} and A_{12}	32	30.5	2	1.9	10

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

TABLE 8

DESIGN RESULTS OF CORBELS SUPPORTING THE CASK HANDLING CRANE

Structural Element	Governing(1) Load Combination Equation	A_s Required		A_s Provided	
		Primary Tension	Shear	Primary Tension	Shear
Column lines A_9 and A_{11} between A_A and A_G	3	2.77 in.^{2} (2)	0.94 in.^{2}	3.39 in.^{2}	1.65 in.^{2}

(1) Load combination equations correspond to equations in Appendix B.
(2) Governed by minimum Code reinforcing requirements.

TABLE 9

MAXIMUM FOUNDATION BEARING PRESSURES[1]

Gross Static (ksf)	Net Static (ksf)	Gross Dynamic (ksf)	Net Dynamic (ksf)	Allowable Net[2] Bearing Capacity		Computed[3] Factor of Safety	
				Static (ksf)	Dynamic (ksf)	Static	Dynamic
10.2	-3.3	28.7	15.2	21.2	31.9	-[4]	4.2

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.

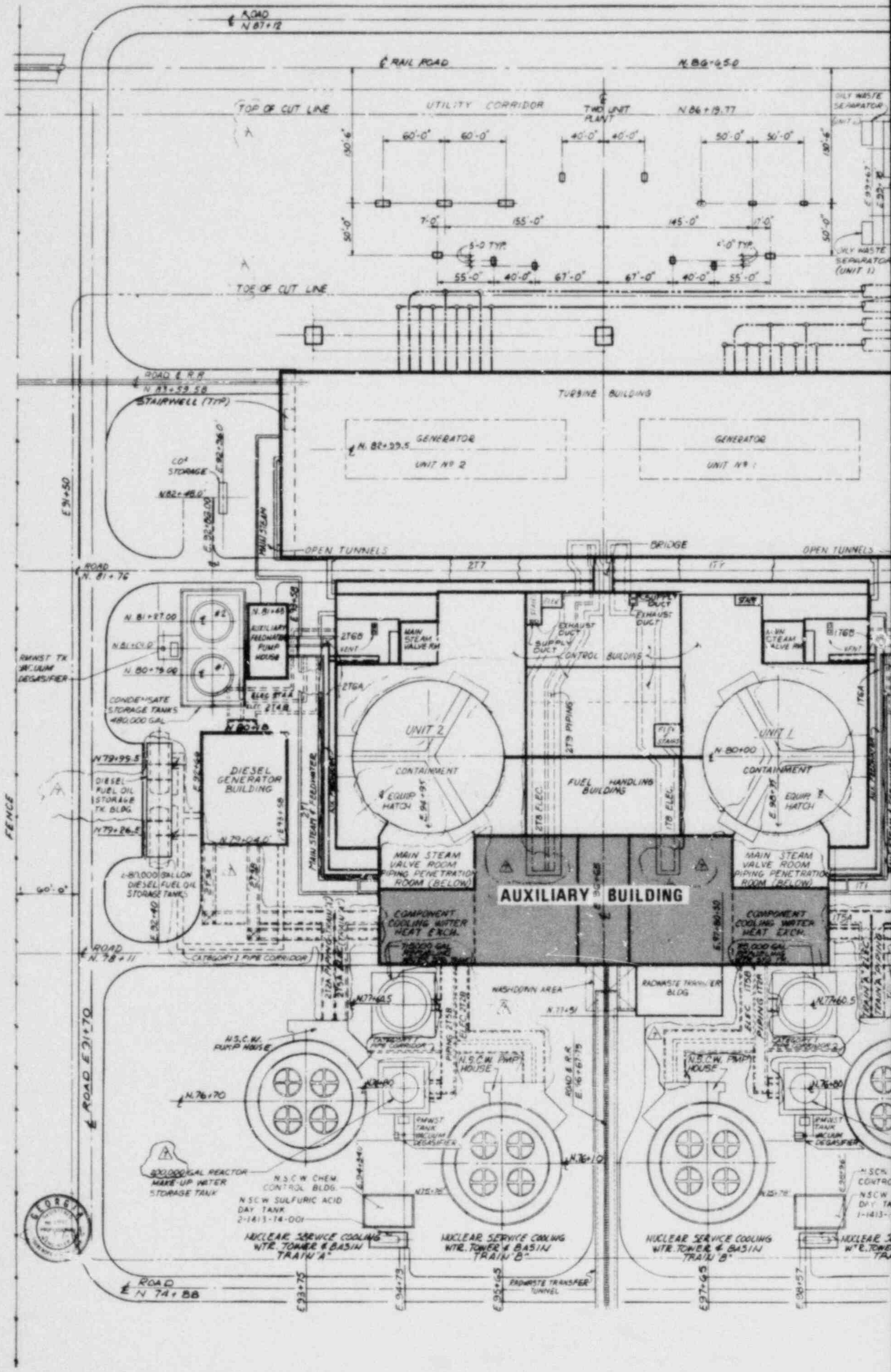
Gross Dynamic = Maximum soil pressure under dynamic loading conditions (i.e., unfactored SSE).

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 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+45.0
UTILITY CORRIDOR
TOP OF CUT LINE
TWO UNIT PLANT
N 86+19.77
OILY WASTE SEPARATOR
OILY WASTE SEPARATOR (UNIT 1)
TOE OF CUT LINE
ROAD & R.R. N 83+59.58
STAIRWELL (TYP)
TURBINE BUILDING
GENERATOR UNIT Nº 2
GENERATOR UNIT Nº 1
CO2 STORAGE
OPEN TUNNELS
BRIDGE
ROAD N. 81+76
AUXILIARY FEEDWATER PUMP HOUSE
RMWST TK VACUUM DEGASIFIER
CONDENSATE STORAGE TANKS 480,000 GAL.
MAIN STEAM VALVE RM
CONTROL BUILDING
EXHAUST DUCT
SUPPLY DUCT
UNIT 2 CONTAINMENT
UNIT 1 CONTAINMENT
EQUIP. HATCH
FUEL HANDLING BUILDING
DIESEL GENERATOR BUILDING
DIESEL FUEL OIL STORAGE TK BLDG.
680,000 GALLON DIESEL FUEL OIL STORAGE TANKS
FENCE
MAIN STEAM VALVE ROOM PIPING PENETRATION ROOM (BELOW)
AUXILIARY BUILDING
COMPONENT COOLING WATER HEAT EXCH.
ROAD N 78+11
CATEGORY I PIPE CORRIDOR
WASHDOWN AREA
RADWASTE TRANSFER BLDG
N.S.C.W. PUMP HOUSE
RMWST TANK VACUUM DEGASIFIER
ROAD E 91+70
500,000 GAL REACTOR MAKE-UP WATER STORAGE TANK
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

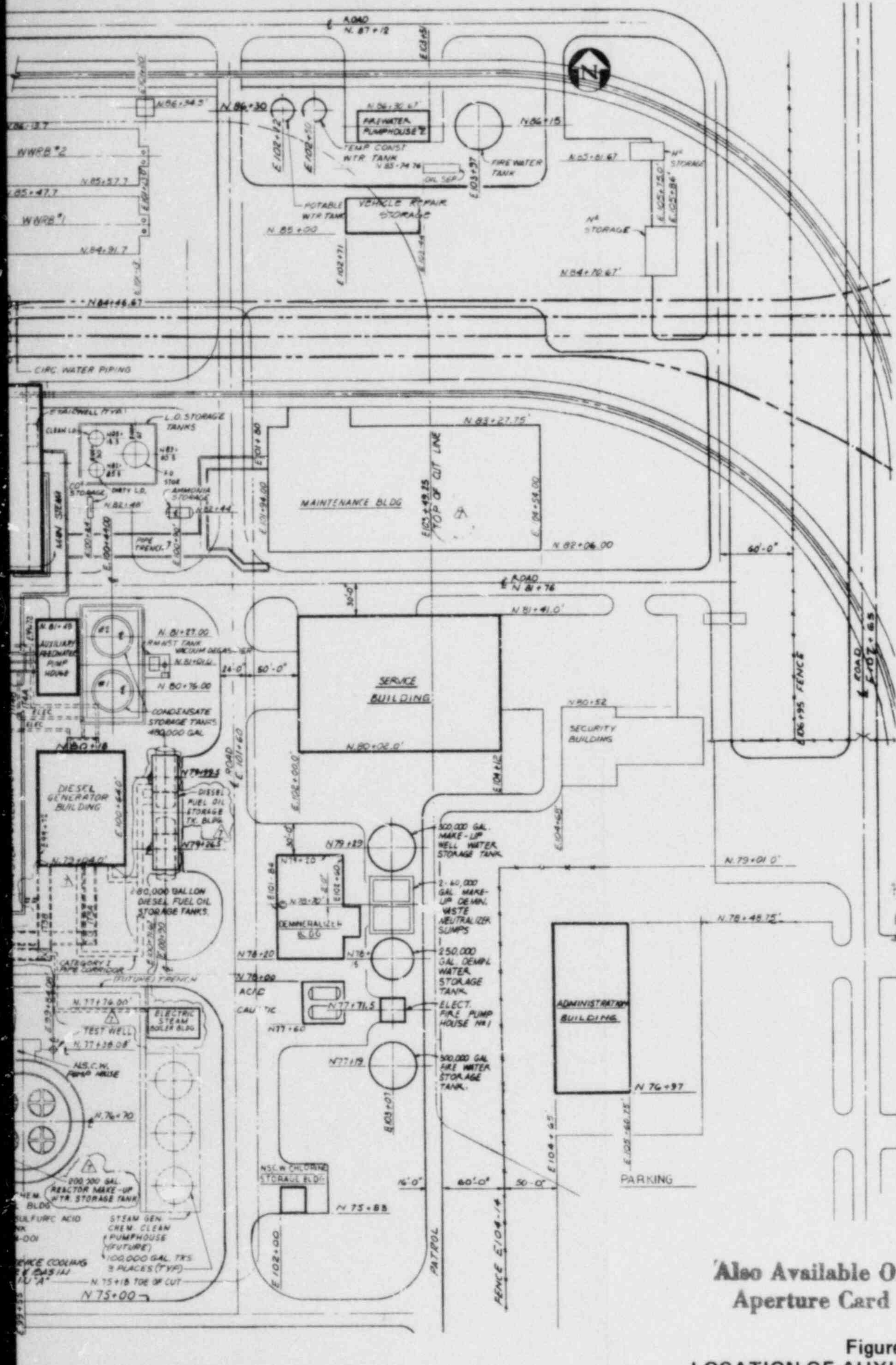


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Figure 1
LOCATION OF AUXILIARY BUILDING

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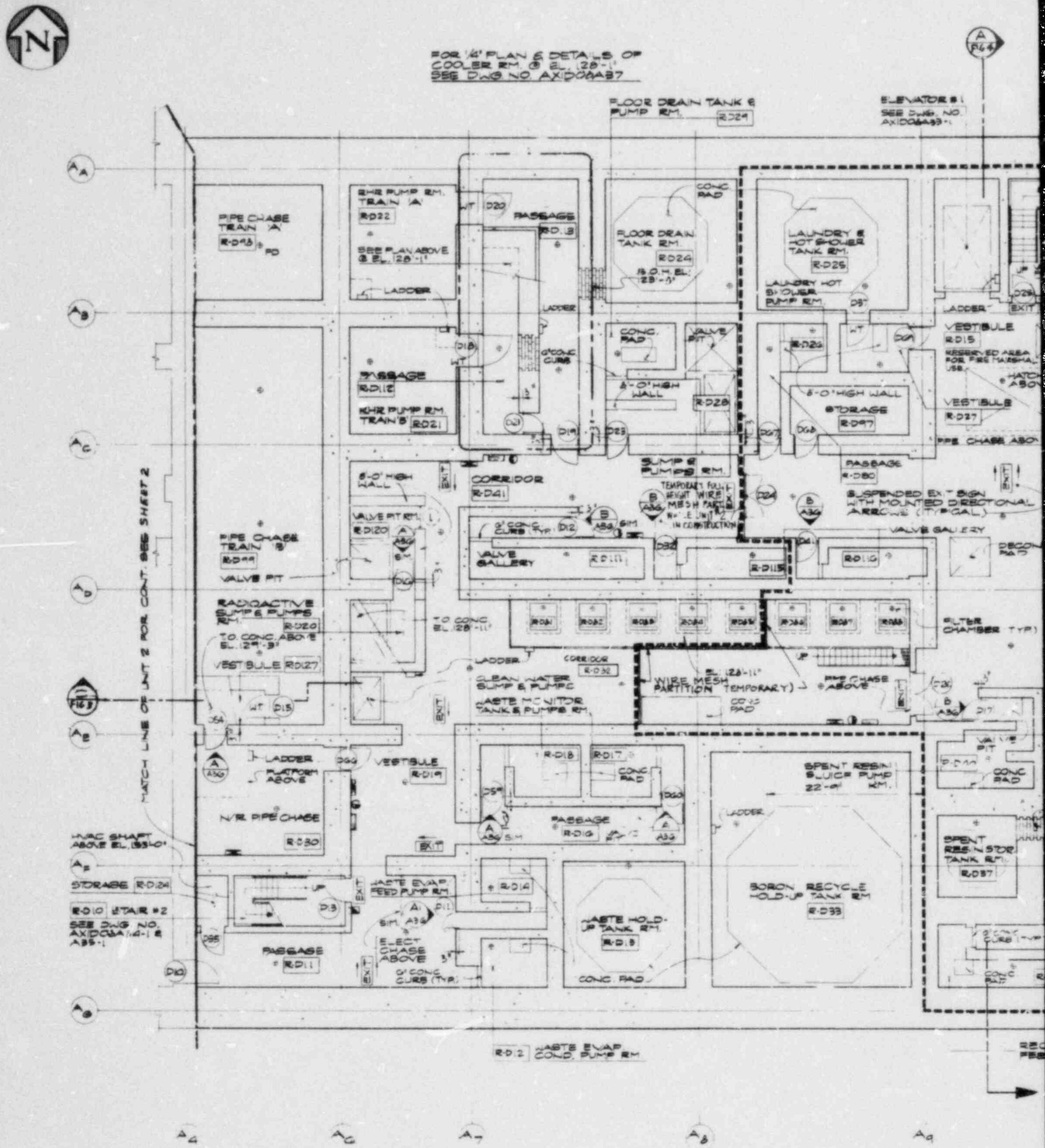

N
FOR 'A' PLAN & DETAILS OF COOLER RM. @ EL. 128'-1" SEE DWG NO. AX1D06A837
FLOOR DRAIN TANK & PUMP RM. R-D29
ELEVATOR #1 SEE DWG. NO. AX1D06A833-1
PIPE CHASE TRAIN 'A' R-D43
RHR PUMP RM. TRAIN 'A' R-D22
SEE PLAN ABOVE @ EL. 128'-1"
LADDER
PASSAGE R-D13
FLOOR DRAIN TANK RM. R-D24
CONC. PAD
LAUNDRY & HOT SHOWER TANK RM. R-D25
LAUNDRY HOT SHOWER PUMP RM.
VESTIBULE R-D15
RESERVED AREA FOR FIRE MARSHAL USE
VESTIBULE R-D27
PASSAGE R-D112
RHR PUMP RM. TRAIN 'B' R-D21
6'-0" HIGH WALL
STORAGE R-D97
VALVE PIT
SUMP & PUMPS RM.
CORRIDOR R-D41
PASSAGE R-D80
SUSPENDED EXIT SIGN WITH MOUNTED DIRECTIONAL ARROWS (TYPICAL)
VALVE GALLERY
PIPE CHASE TRAIN 'B' R-D44
VALVE PIT RM. R-D120
VALVE GALLERY R-D111
R-D115
R-D116
VALVE PIT
RADIOACTIVE SUMP & PUMPS RM. R-D20
TO CONC. ABOVE EL. 129'-3"
TO CONC. EL. 128'-11"
FILTER CHAMBER TYP.
VESTIBULE R-D127
LADDER
CORRIDOR R-D32
CLEAN WATER SUMP & PUMPS
WASTE MONITOR TANK & PUMPS RM.
WIRE MESH PARTITION TEMPORARY
PIPE CHASE ABOVE
CONC. PAD
MATCH LINE OF UNIT 2 FOR CONT. SEE SHEET 2
LADDER
PLATFORM ABOVE
VESTIBULE R-D19
R-D18
R-D17
CONC. PAD
SPENT RESIN SLUICE PUMP RM.
N/R PIPE CHASE R-D30
PASSAGE R-D16
HVAC SHAFT ABOVE EL. 133'-0"
EXIT
STORAGE R-D126
R-D10 STAIR #2
SEE DWG NO. AX1D06A414-1 & A35-1
WASTE EVAP. FEED PUMP RM.
R-D14
WASTE HOLD-UP TANK RM. R-D13
BORON RECYCLE HOLD-UP TANK RM R-D33
SPENT RESIN STOR. TANK RM. R-D37
ELECT CHASE ABOVE
PASSAGE R-D11
O' CONC. CURB (TYP.)
CONC. PAD
R-D12 WASTE EVAP. COND. PUMP RM

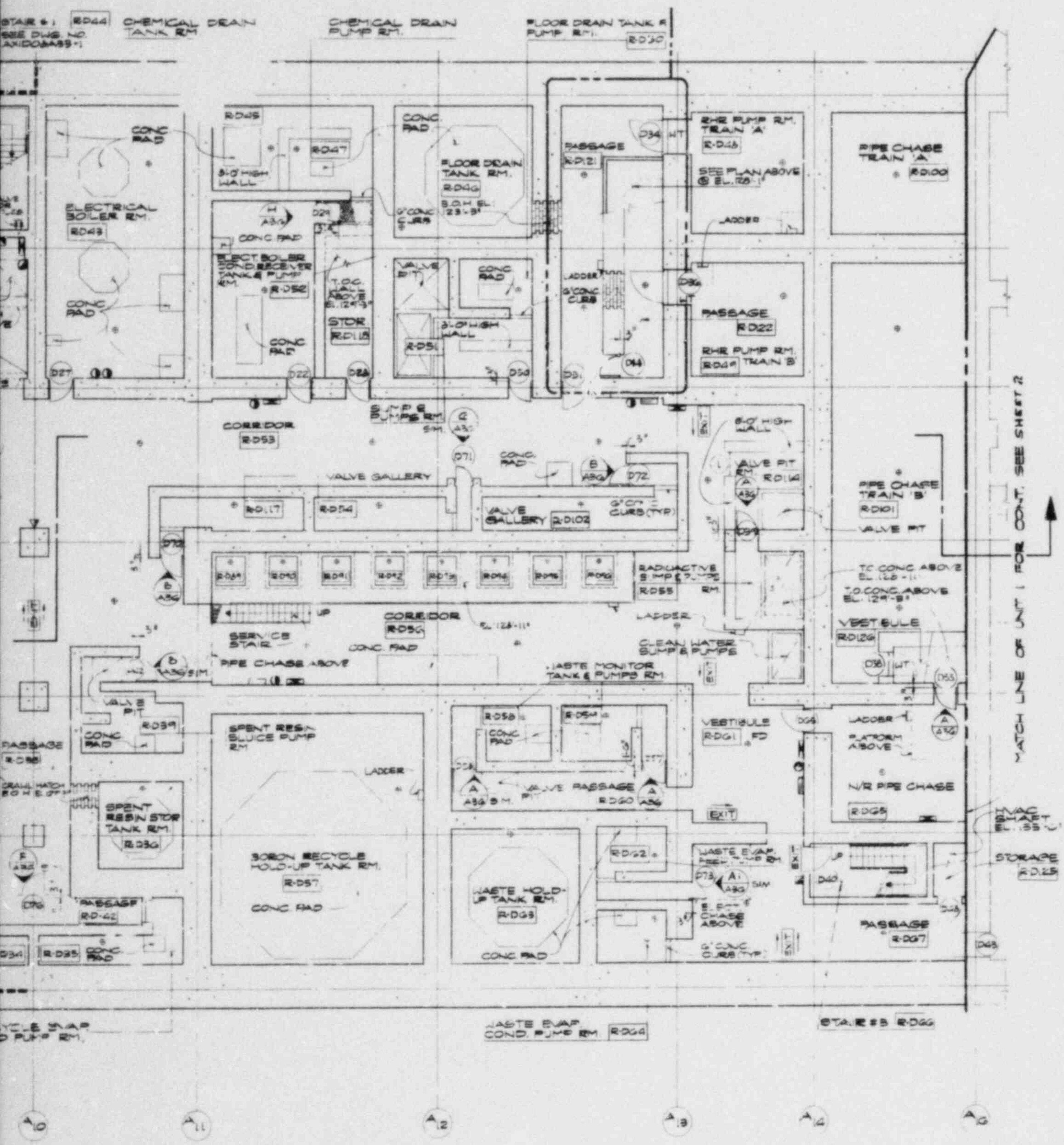

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Figure 2
AUXILIARY BUILDING FLOOR PLAN
ELEV. 119'-3" LEVEL D
(Sheet 1 of 2)

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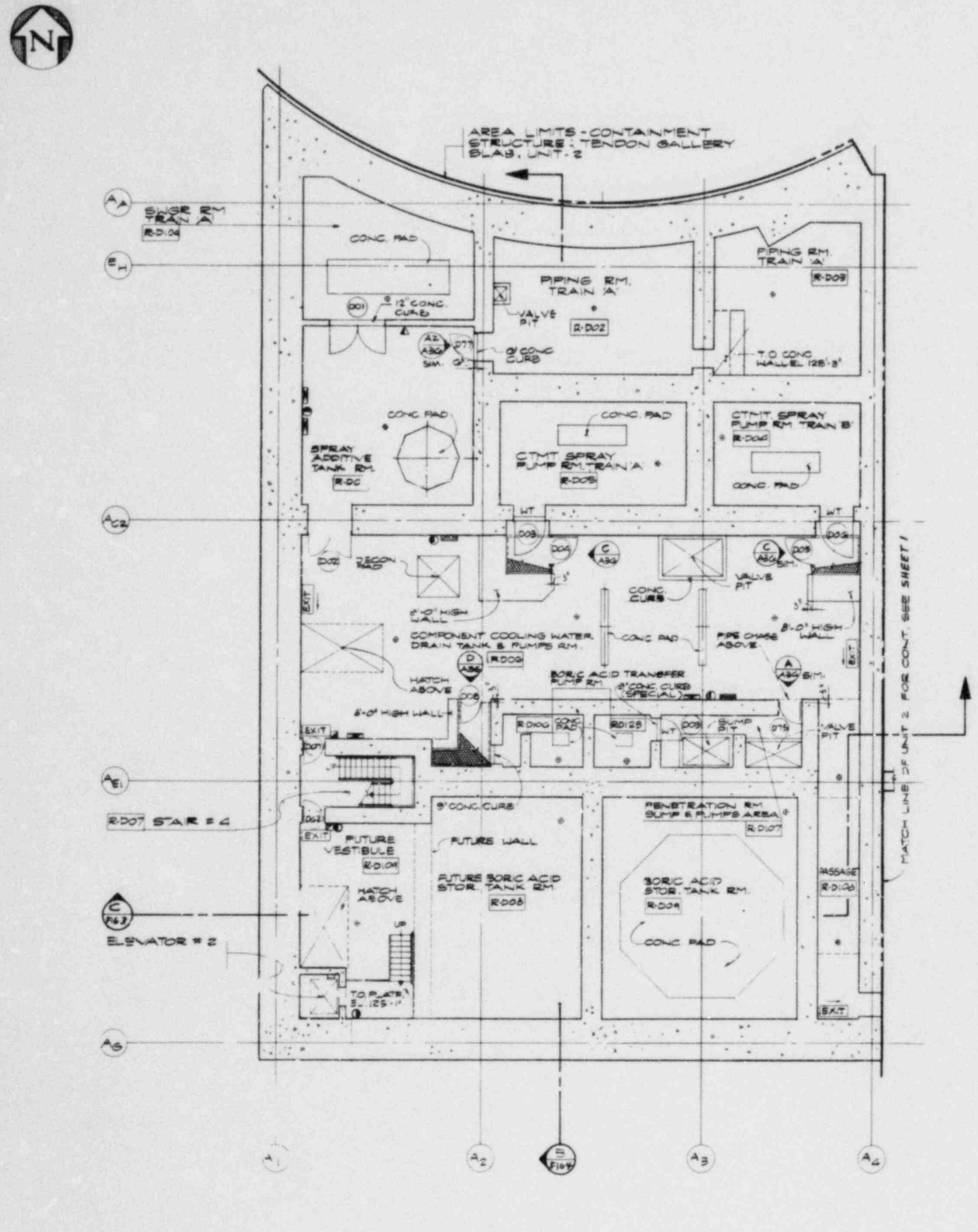
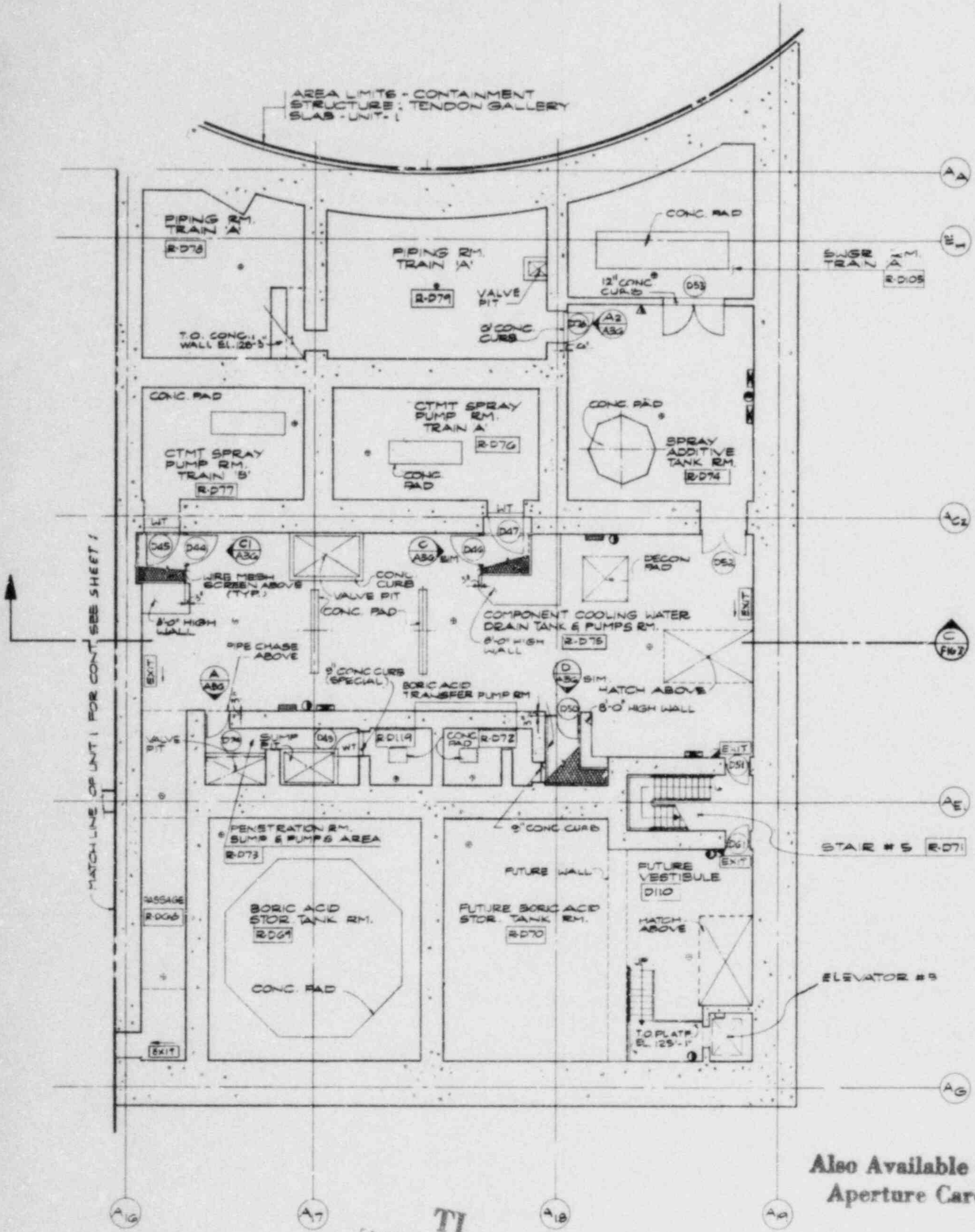


N
AREA LIMITS - CONTAINMENT STRUCTURE : TENDON GALLERY SLAB, UNIT - 2
SWGR RM TRAIN 'A'
R-D104
CONC. PAD
12" CONC. CURB
PIPING RM. TRAIN 'A'
R-D02
VALVE PIT
PIPING RM. TRAIN 'A'
R-D03
T.O. CONC WALL EL 128'-3"
SPRAY ADDITIVE TANK RM.
CONC. PAD
CTMT SPRAY PUMP RM. TRAIN 'A'
R-D05
CTMT SPRAY PUMP RM. TRAIN 'B'
CONC. PAD
DECON PAD
8'-0" HIGH WALL
COMPONENT COOLING WATER DRAIN TANK & PUMPS RM.
CONC. CURB
CONC. PAD
VALVE PIT
PIPE CHASE ABOVE
8'-0" HIGH WALL
HATCH ABOVE
BORIC ACID TRANSFER PUMP RM.
8'-0" HIGH WALL
SUMP PIT
VALVE PIT
EXIT
R-D07 STAIR # 4
9" CONC. CURB
FUTURE WALL
FUTURE VESTIBULE
HATCH ABOVE
FUTURE BORIC ACID STOR. TANK RM.
R-D08
PENETRATION RM. SUMP & PUMPS AREA
BORIC ACID STOR. TANK RM.
CONC PAD
PASSAGE
ELEVATOR # 2
T.O. PLATE
MATCH LINE OF UNIT 2 FOR CONT. SEE SHEET 1
EXIT

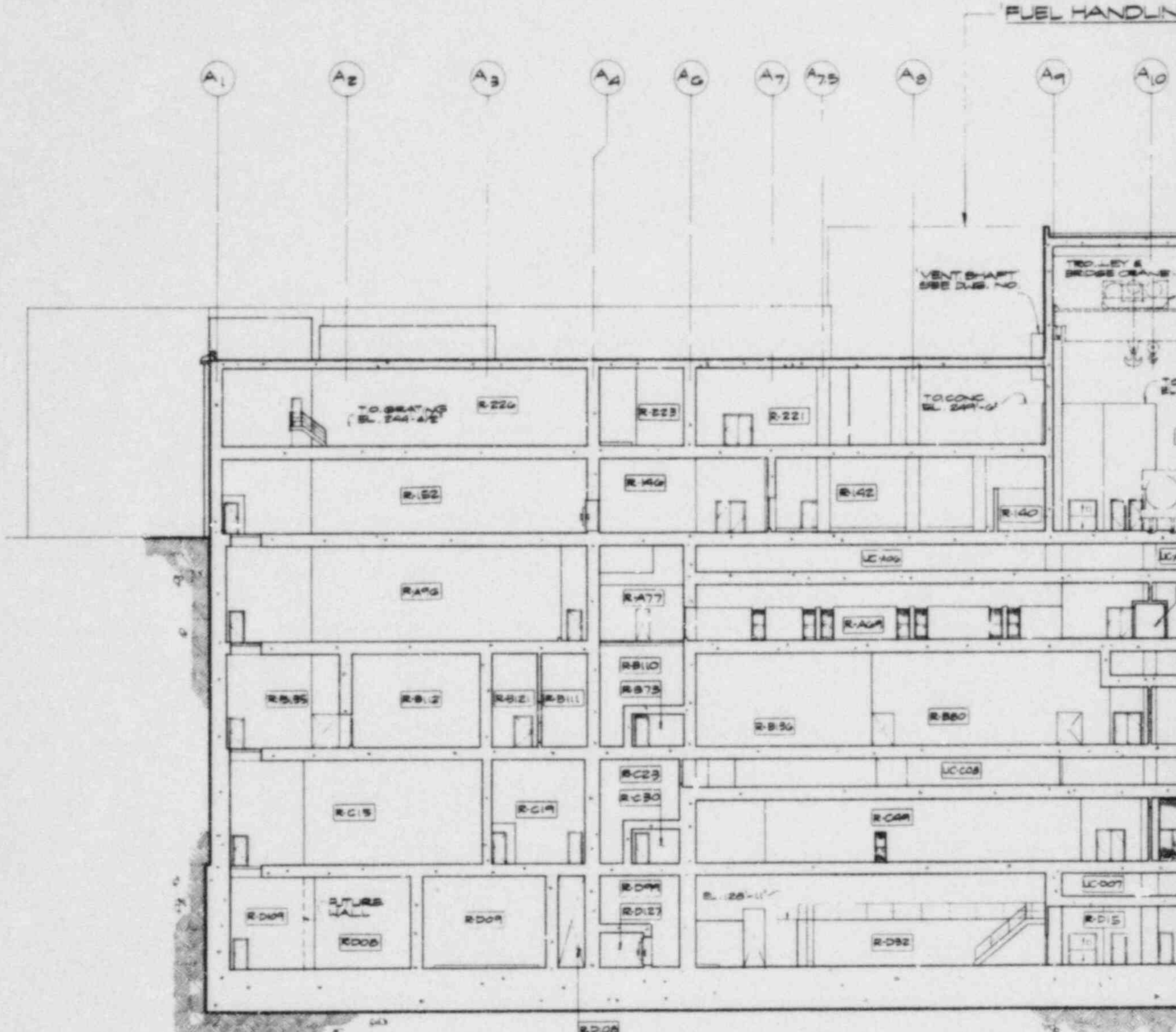


Also Available On
Aperture Card

TI
APERTURE
CARD

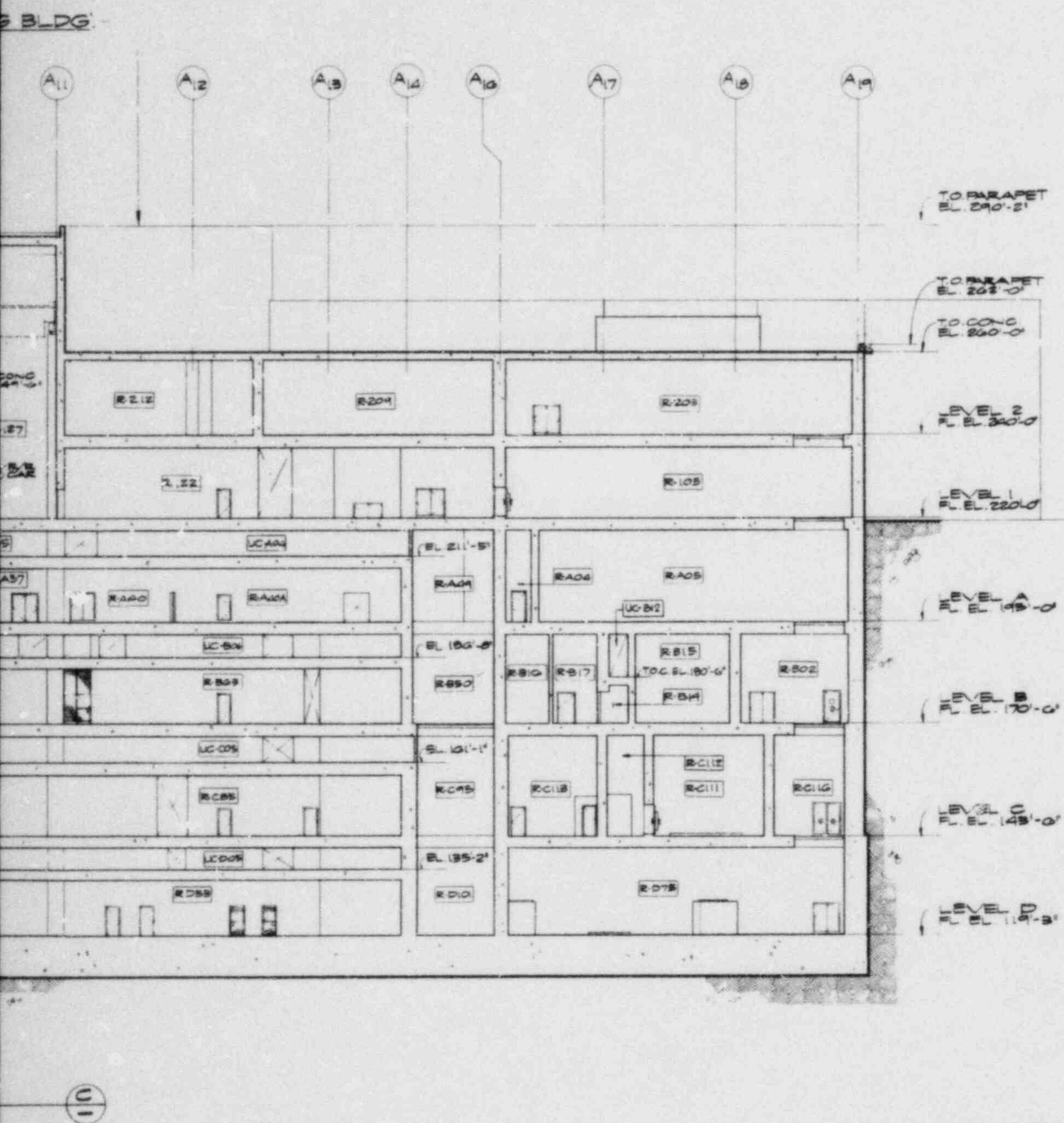
Figure 2
AUXILIARY BUILDING FLOOR PLAN
EL. 119'-3" LEVEL D
(Sheet 2 of 2)

8411050168-03



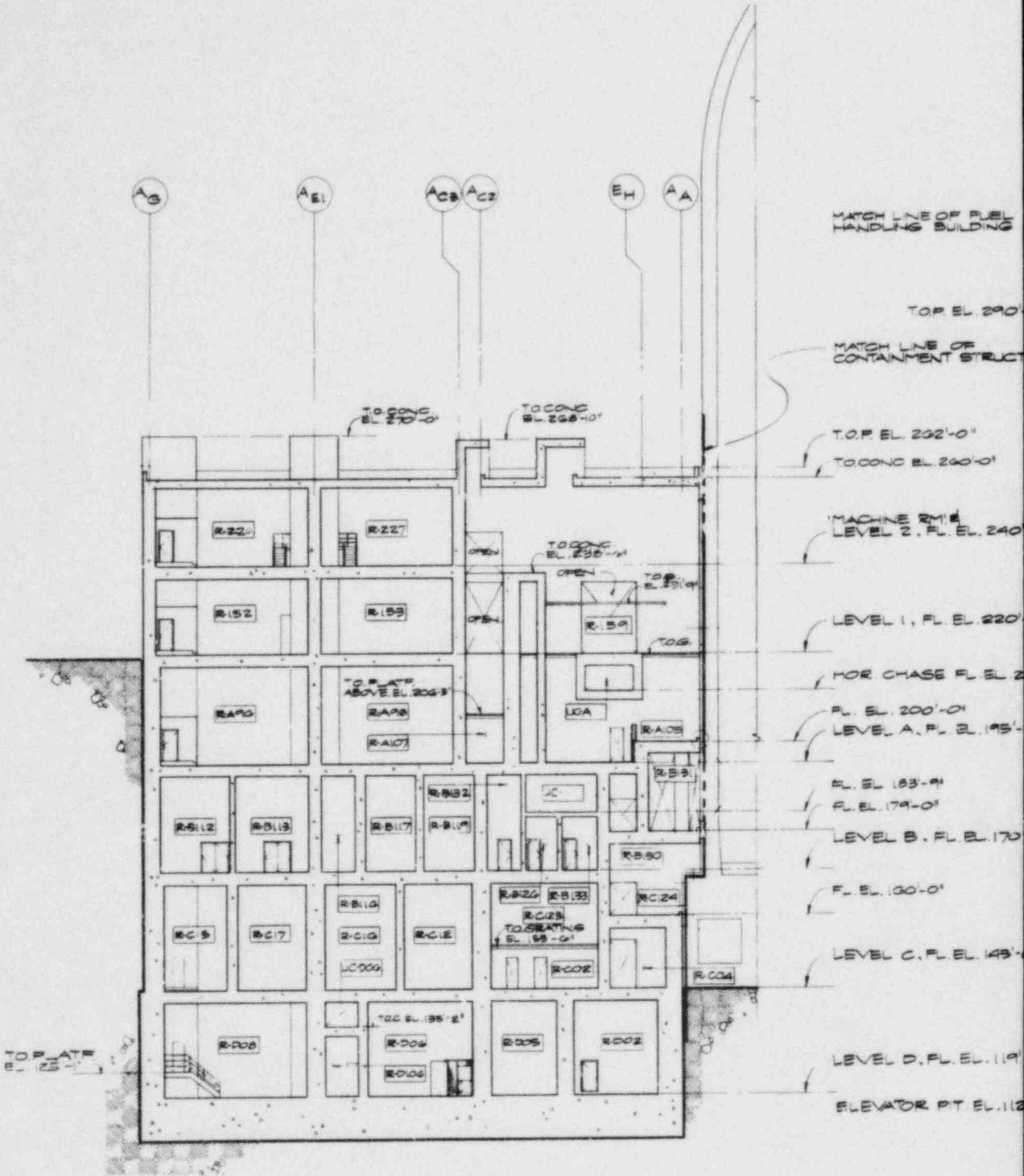
FUEL HANDLING
A1
A2
A3
A4
A6
A7
A7.5
A8
A9
A10
VENT SHAFT SEE DWG. NO.
TROLLEY & BRIDGE CRANE
R-226
R-223
R-221
R-152
R-146
R-142
R-140
R-A77
R-B135
R-B111
R-B110
R-B73
R-B80
R-C15
R-C19
R-C23
R-C30
FUTURE HALL
R-D09
R-D127
R-D15
R-D108
UC-C08
UC-007

SECTION



Also Available On
Aperture Card

Figure 3
AUXILIARY BUILDING
SECTION LOOKING NORTH



MATCH LINE OF FUEL HANDLING BUILDING
T.O.P. EL. 290'
MATCH LINE OF CONTAINMENT STRUCT
T.O.CONC EL. 270'-0"
T.O.CONC EL. 268'-0"
T.O.P. EL. 262'-0"
T.O.CONC EL. 260'-0"
MACHINE RM'S LEVEL 2, FL. EL. 240'
LEVEL 1, FL. EL. 220'
HOR. CHASE FL. EL.
FL. EL. 200'-0"
LEVEL A, FL. EL. 195'
FL. EL. 183'-9"
FL. EL. 179'-0"
LEVEL B, FL. EL. 170'
FL. EL. 160'-0"
LEVEL C, FL. EL. 145'
LEVEL D, FL. EL. 119'
ELEVATOR PIT EL. 112
T.O.PLATF. EL. 125'-1"
OPEN
R-227
R-152
R-153
R-159
R-A98
R-A107
R-A108
R-B112
R-B113
R-B117
R-B119
R-B130
R-C16
R-C17
R-C123
R-C124
R-C202
R-C204
R-D08
R-D06
R-D05
R-D02

SECTION B

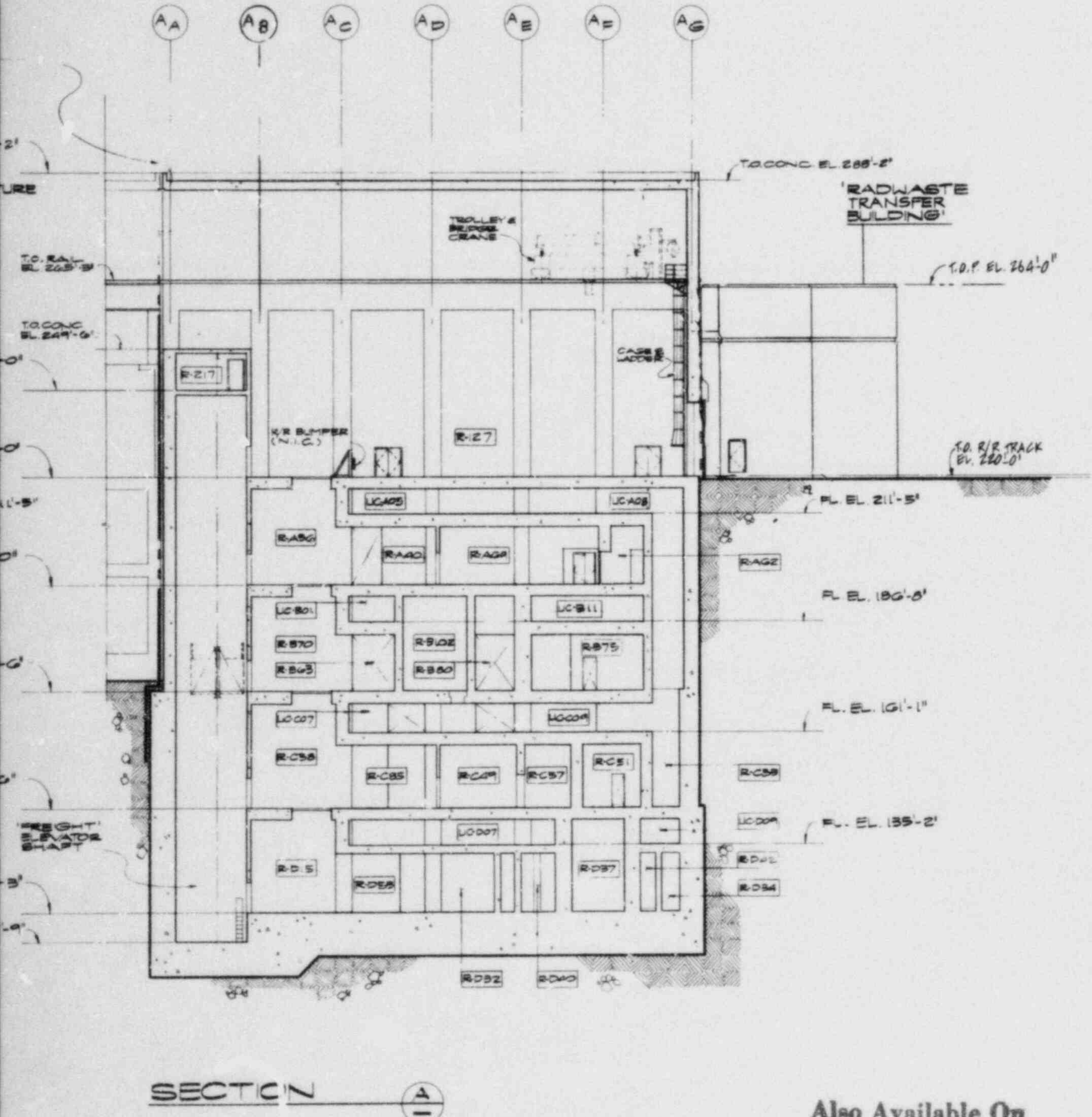
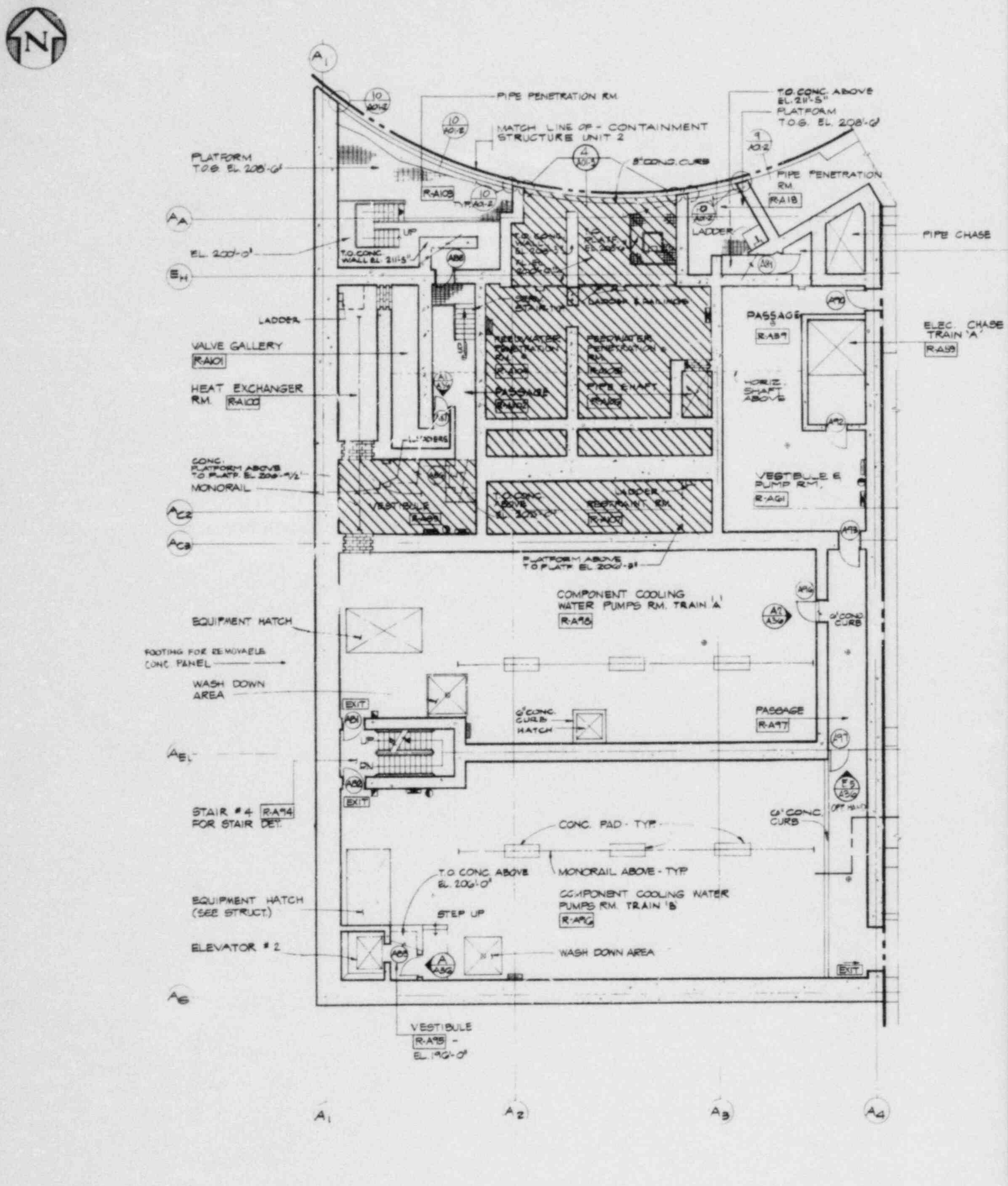


Figure 4
AUXILIARY BUILDING SECTIONS
LOOKING EAST AND WEST



LEGEND

FEEDWATER LINE AREA

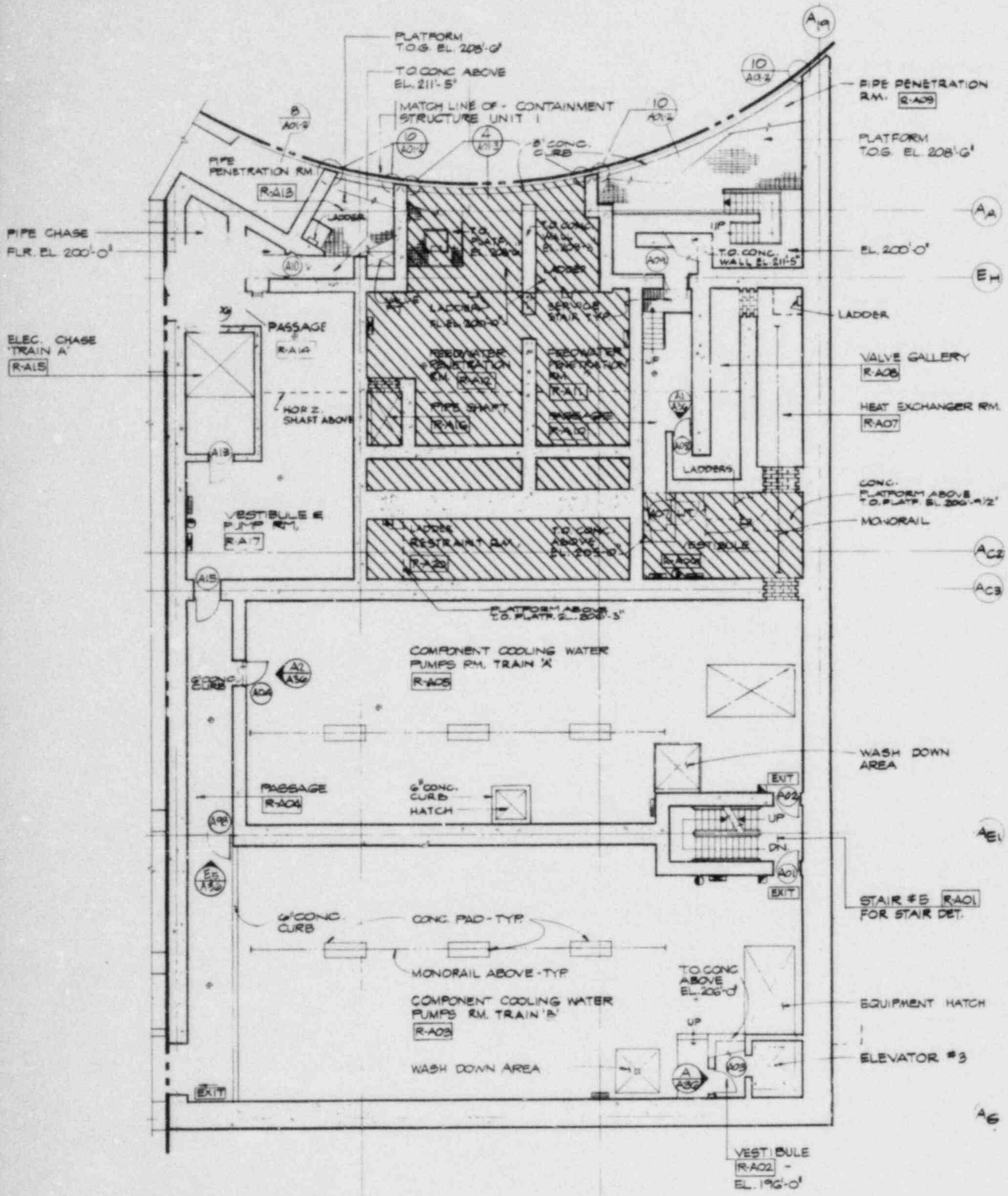


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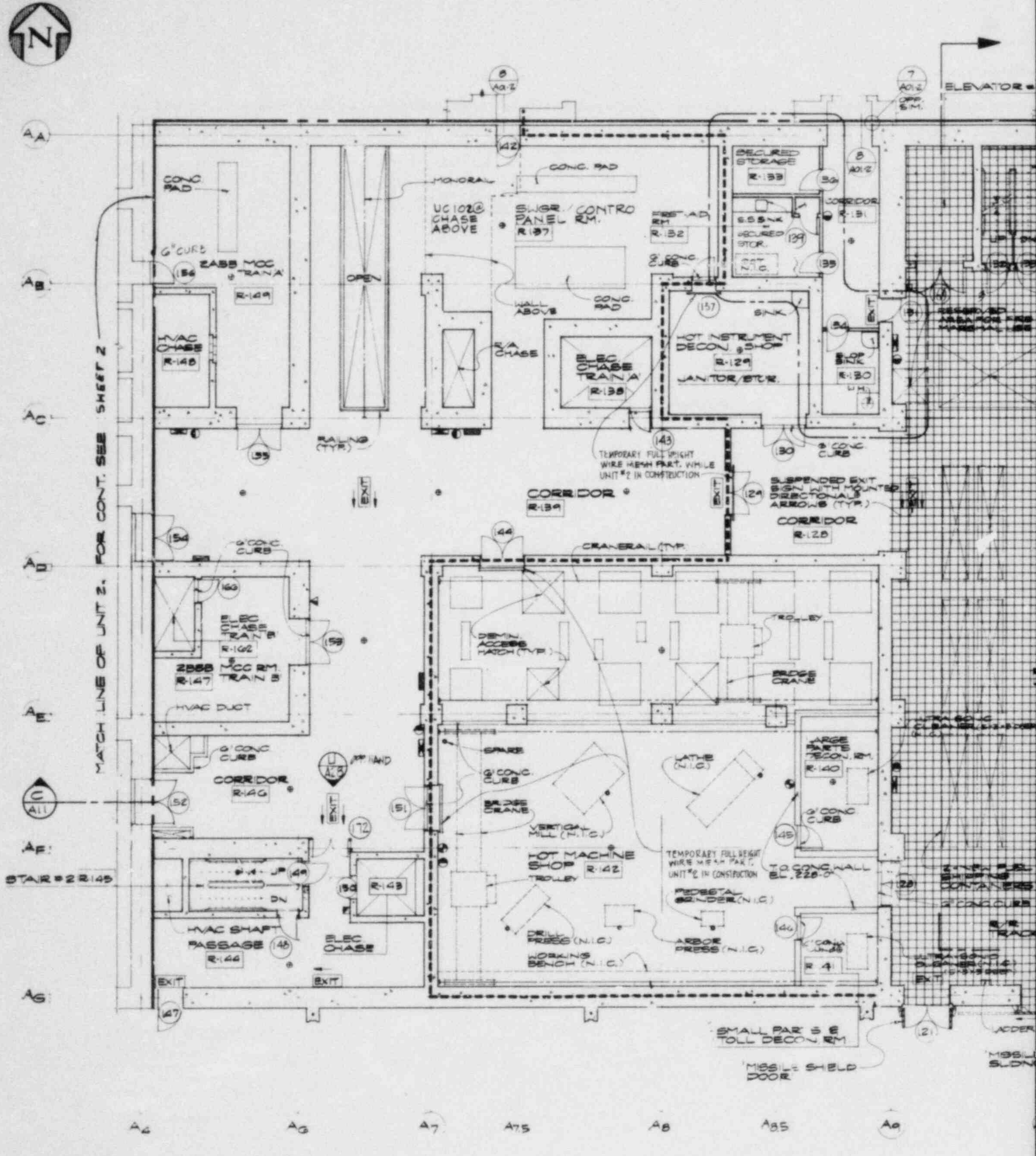
Figure 5
AUXILIARY BUILDING FLOOR PLAN
EL. 195'-0" LEVEL A

8411050168-06



LEGEND:

RAILROAD ENTRANCE AND CASK HANDLING CRANE BAY

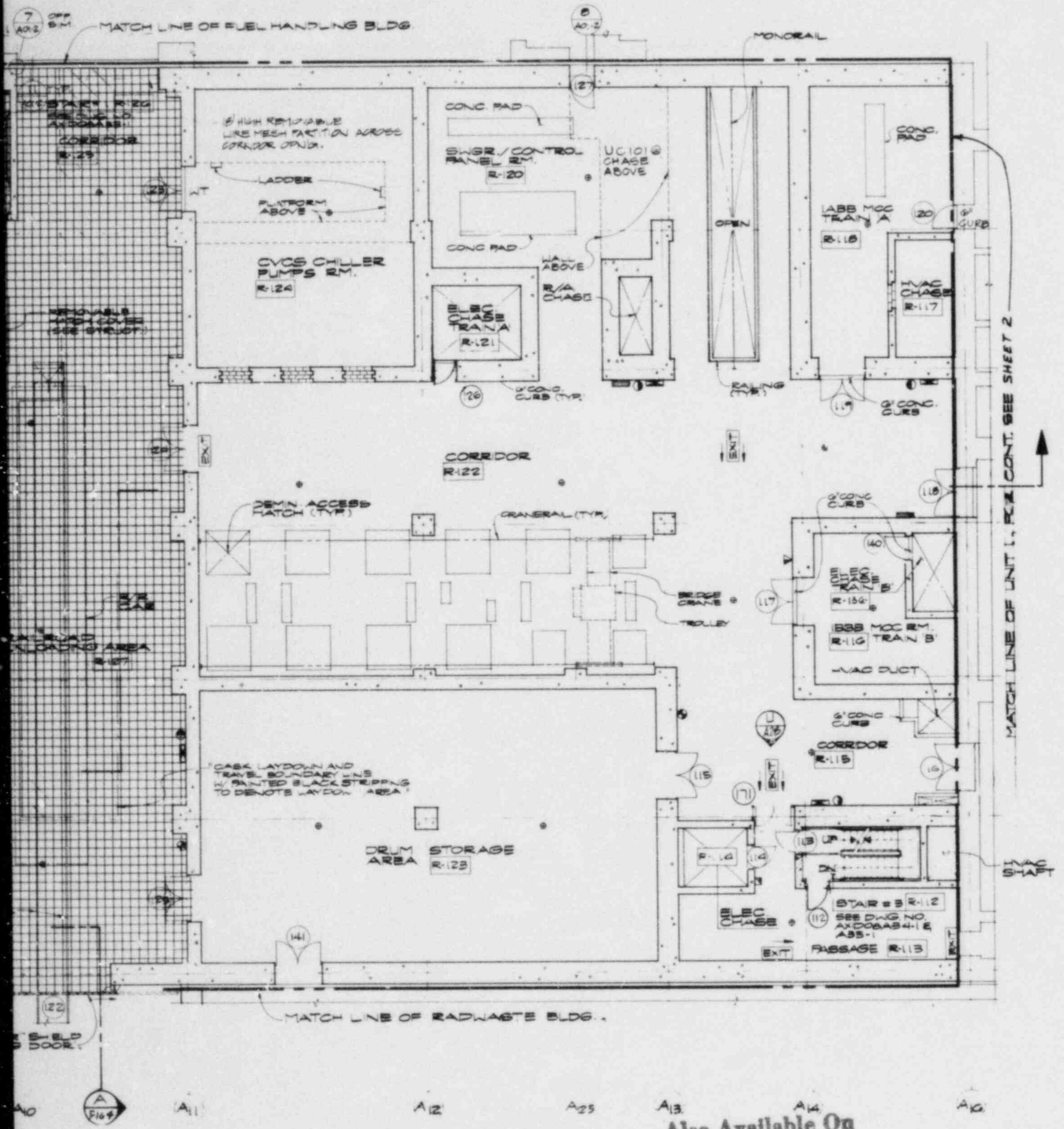
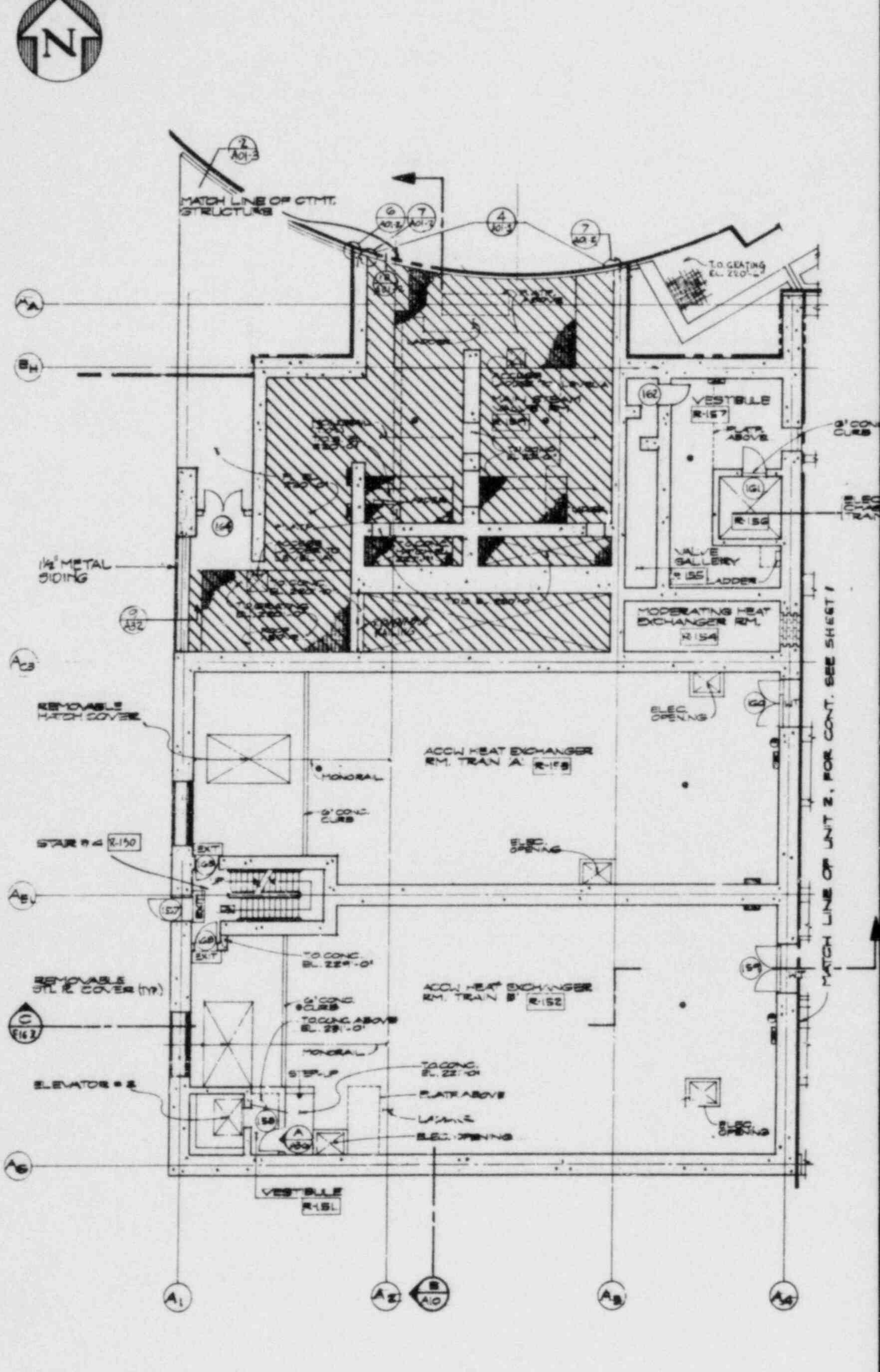
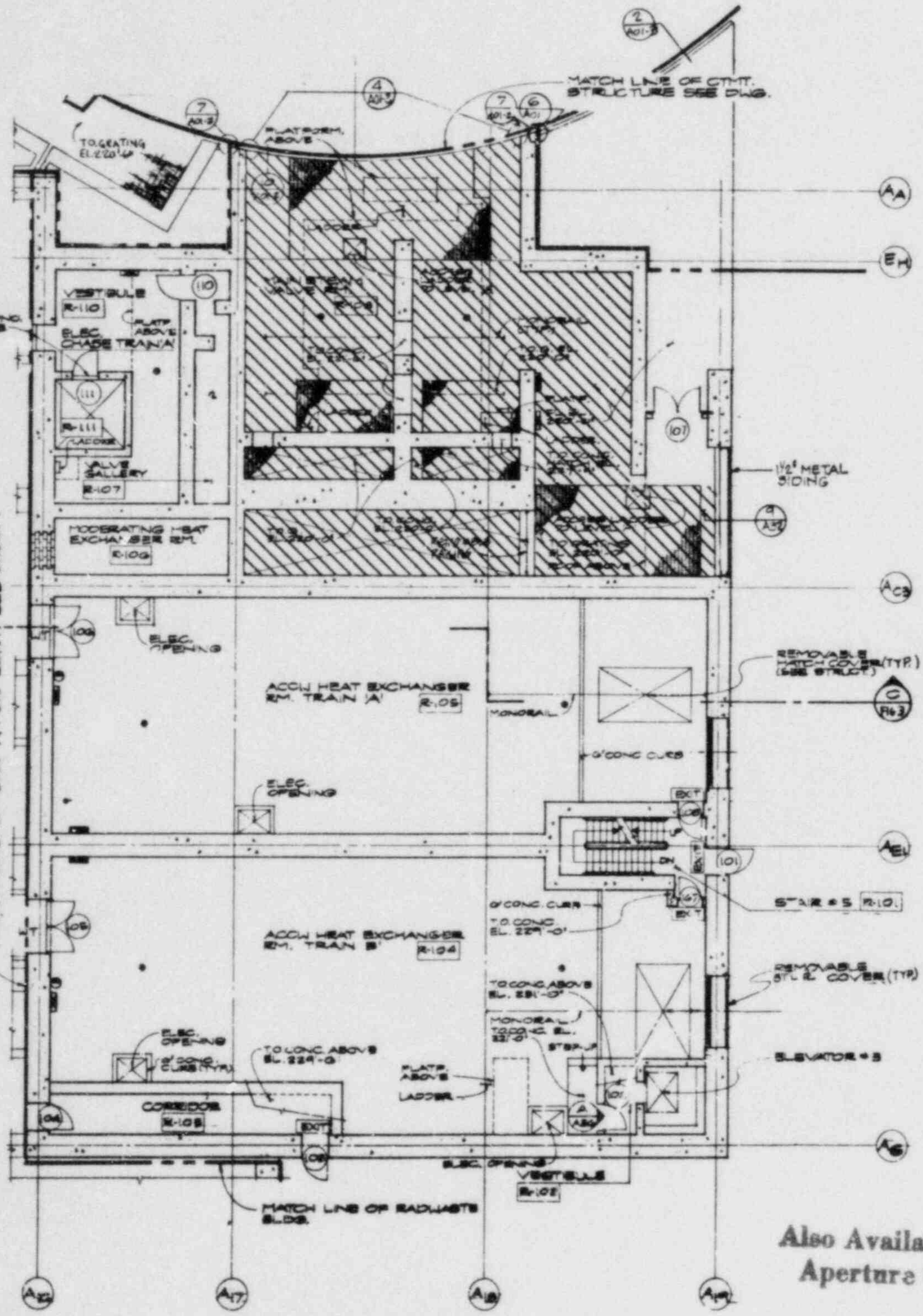


Figure 6
AUXILIARY BUILDING FLOOR PLAN
EL. 220'-0" LEVEL 1
(Sheet 1 of 2)



LEGEND:

MAIN STEAM LINE AREA



Also Available On
Aperture Card

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Figure 6
AUXILIARY BUILDING FLOOR PLAN
EL. 220'-0" LEVEL 1
(Sheet 2 of 2)

8411050168 -08

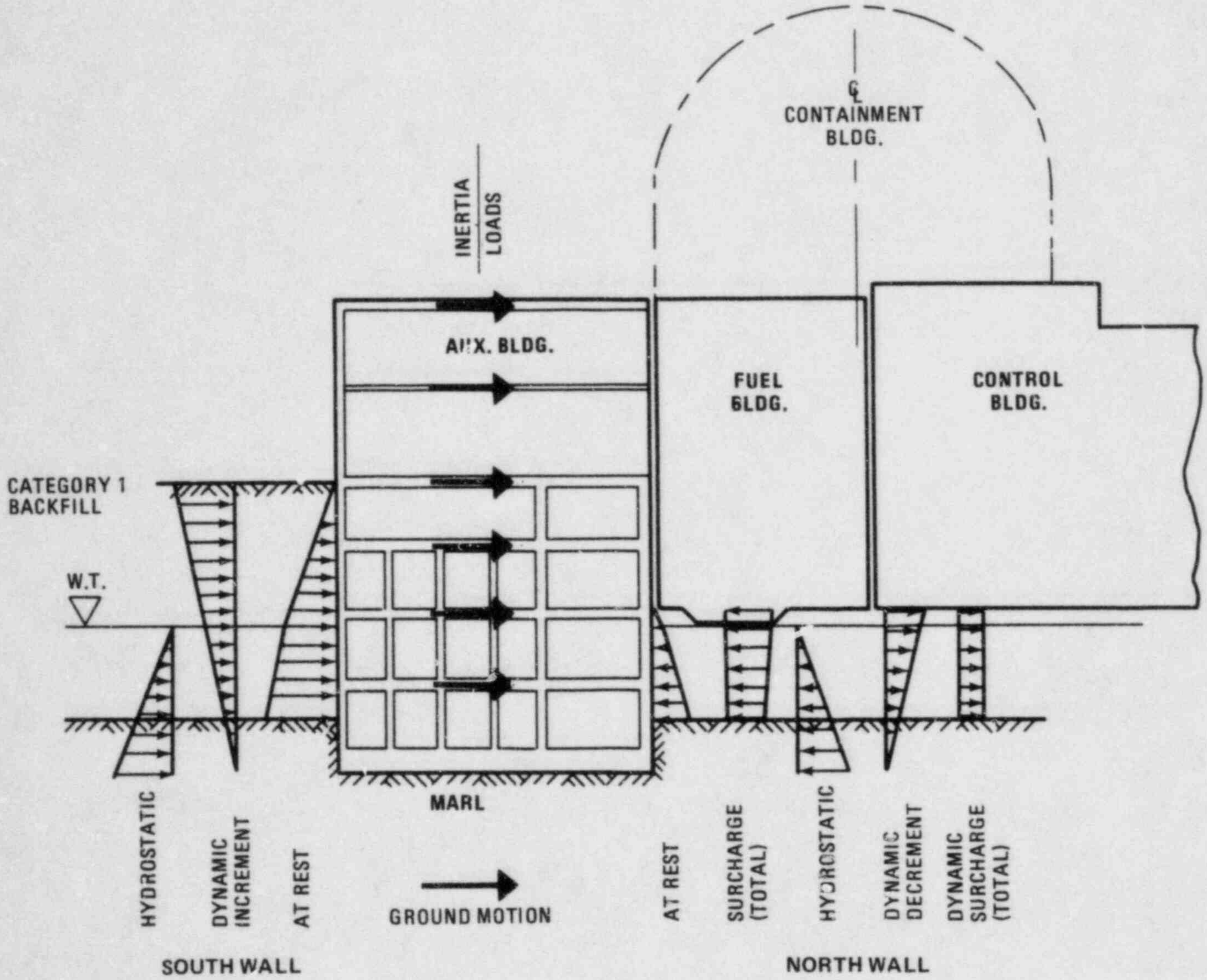
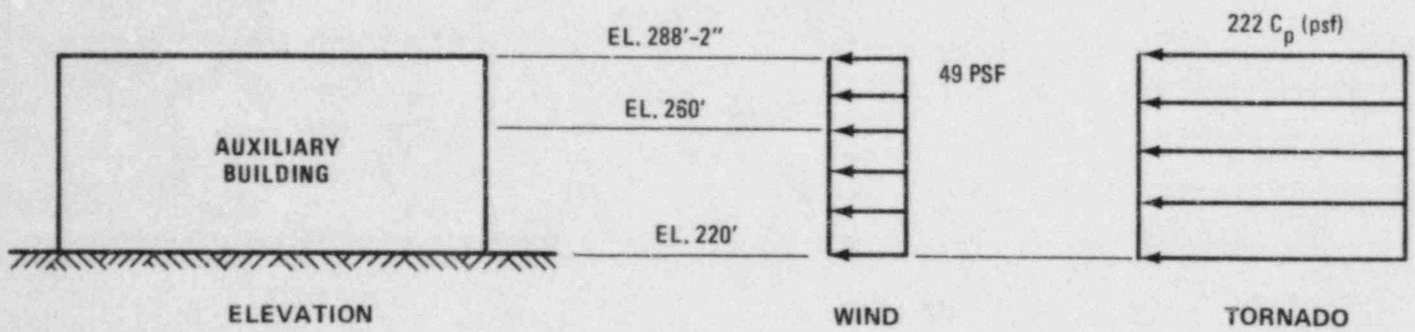


Figure 7
PICTORIAL REPRESENTATION OF LATERAL
EARTH PRESSURES AND STRUCTURAL SURCHARGES



$P = C_s P_{max} C_p$

WHERE:

C_s = SIZE COEFFICIENT
= .67

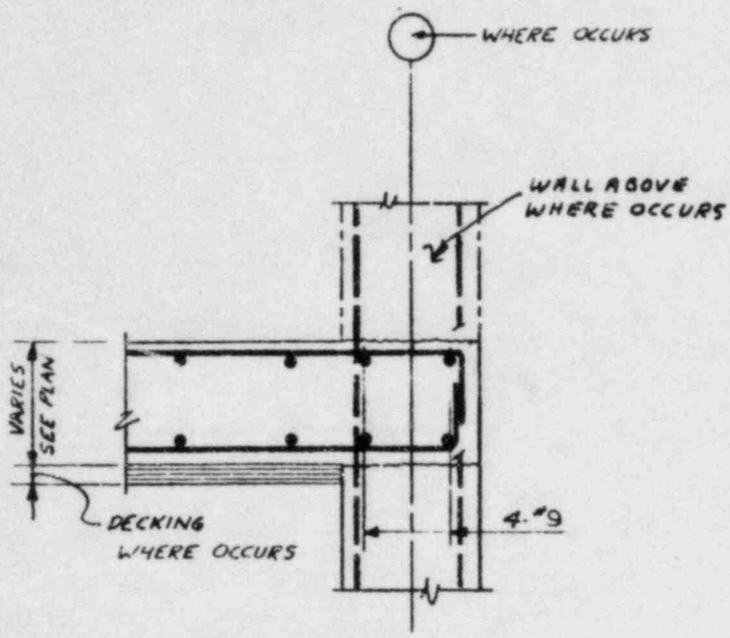
$P_{max} = 0.00256 (V_{max})^2$
$= 0.00256 (360 \text{ mph})^2$
$= 332$ Psf

C_p = EFFECTIVE EXTERNAL PRESSURE COEFFICIENT

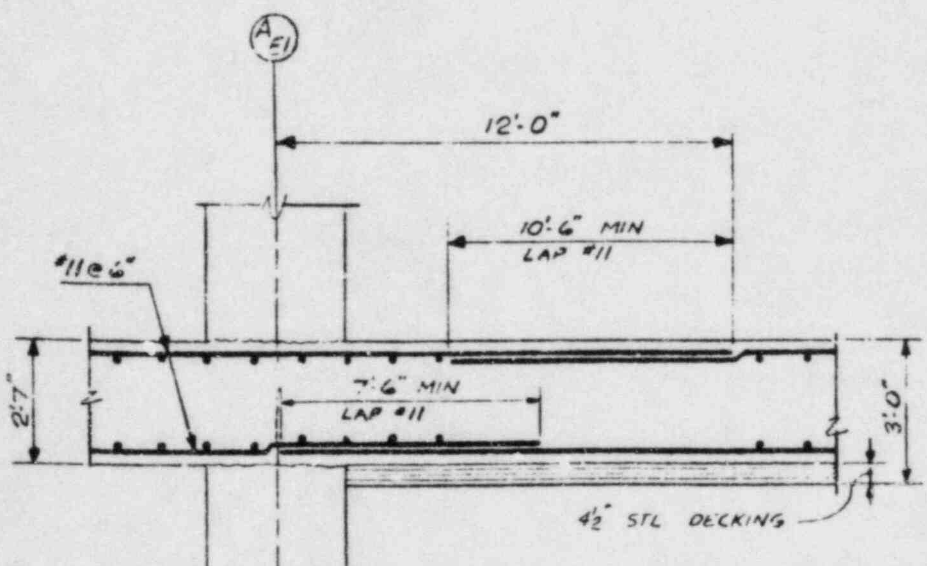
$P = (.67)(332 \text{ psf}) C_p$

$= 222 C_p$ (psf)

Figure 8
WIND AND TORNADO EFFECTIVE
VELOCITY PRESSURE PROFILES

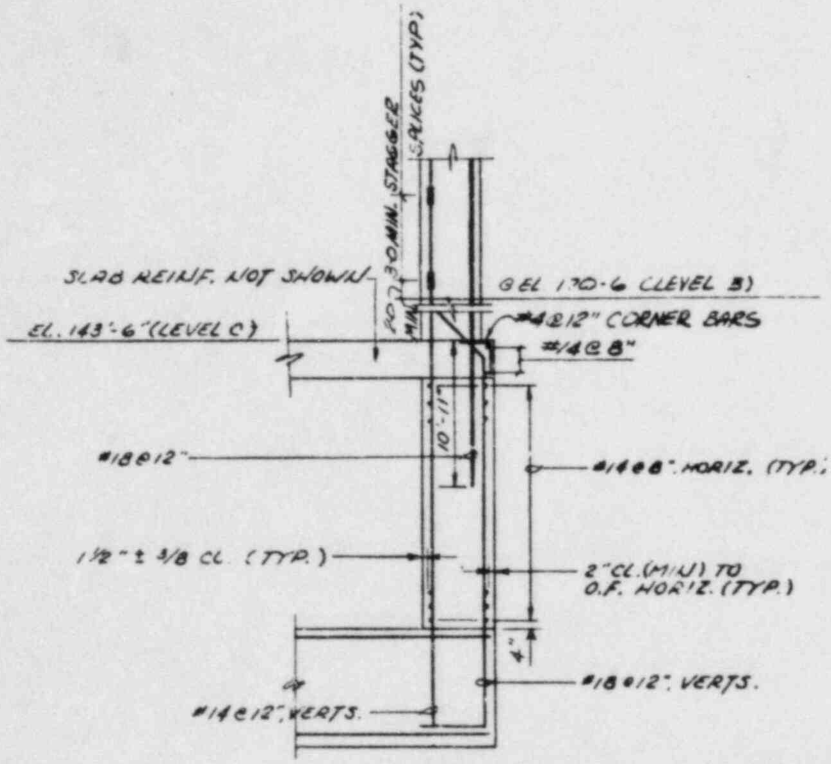


SECTION



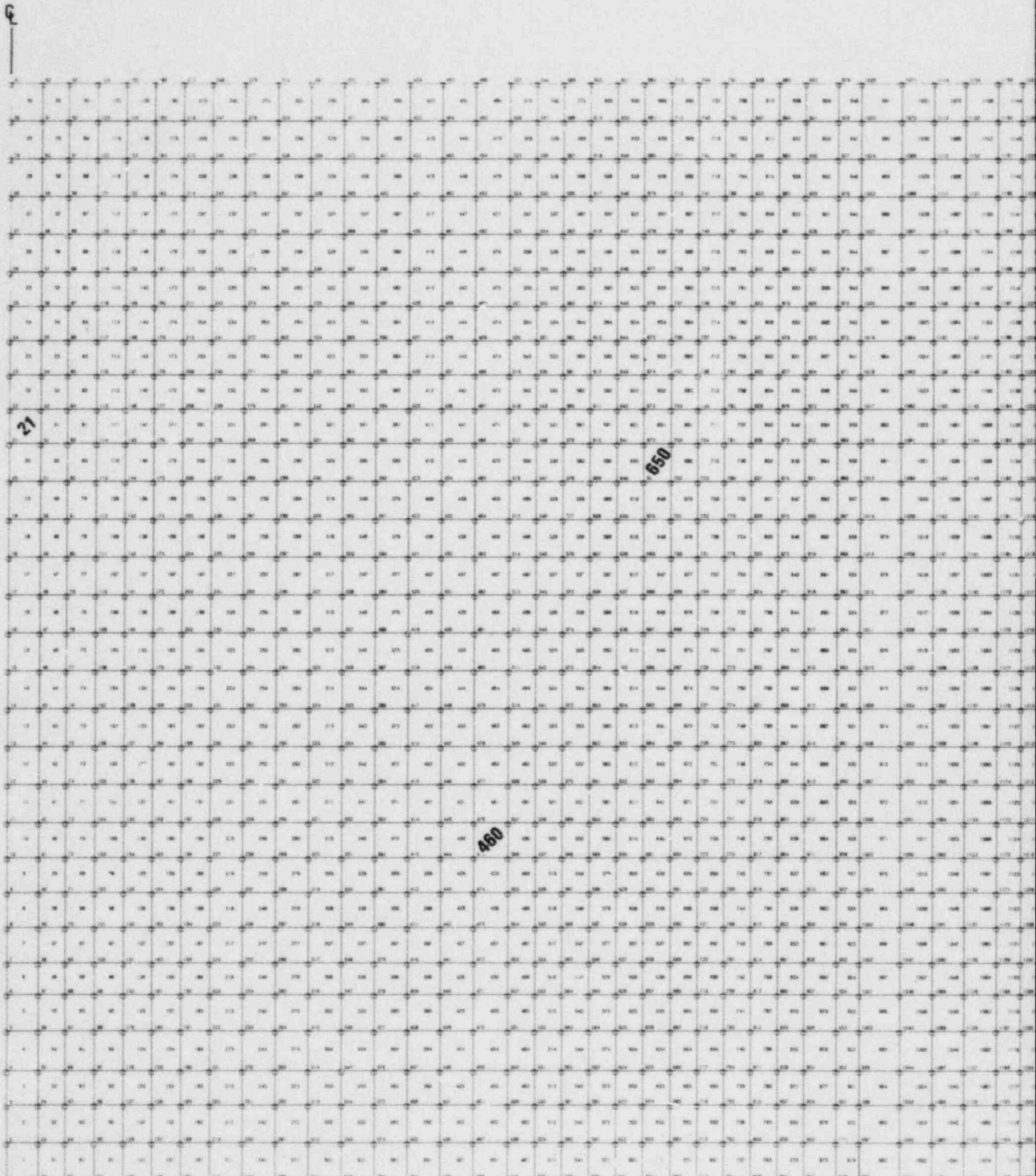
SECTION

Figure 9
REPRESENTATIVE SLAB DETAILS



SECTION

Figure 10
REPRESENTATIVE SHEAR WALL DETAILS


℄
21
650
460

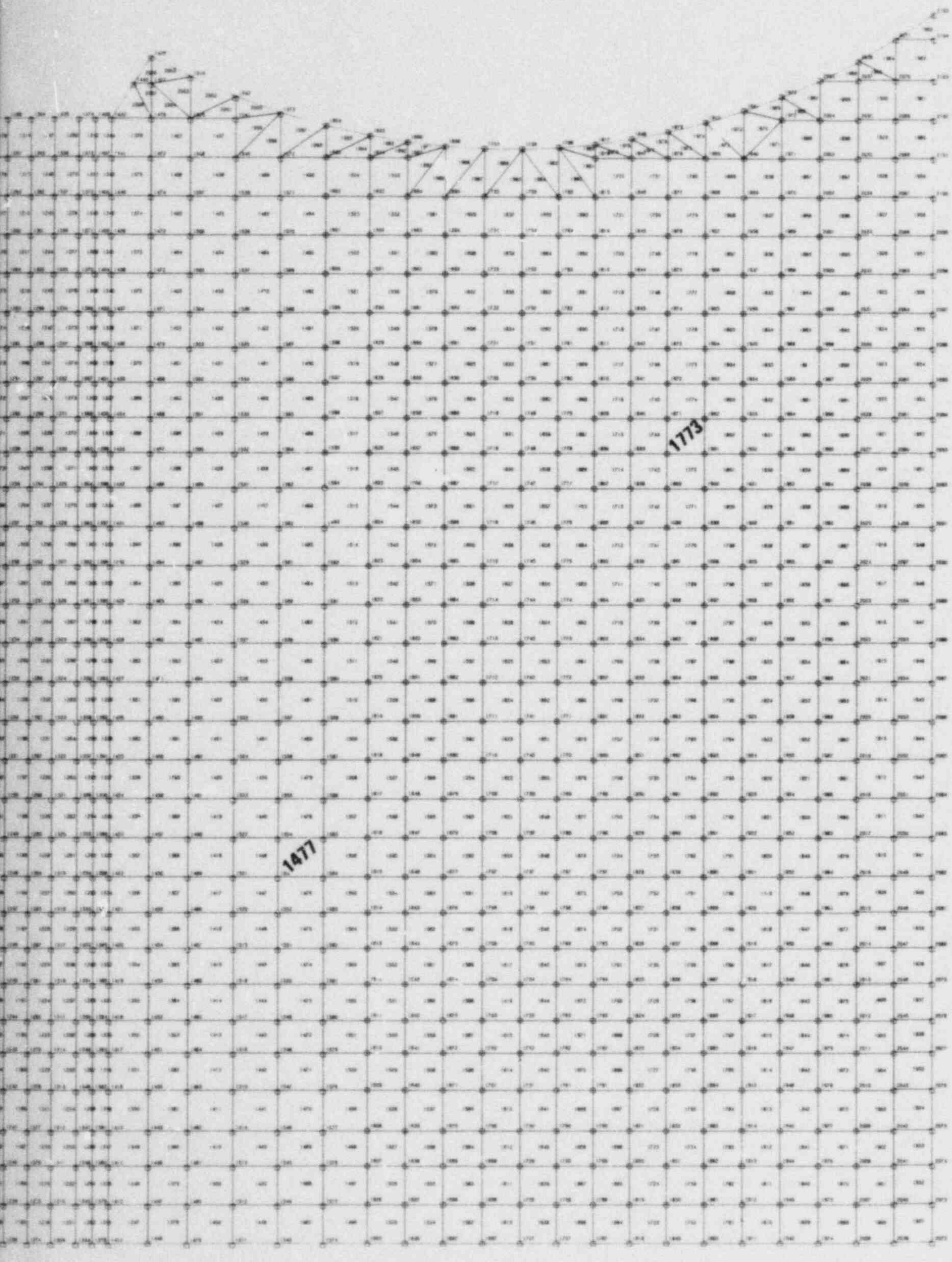


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Figure 11
BASEMAT COMPUTER MODEL

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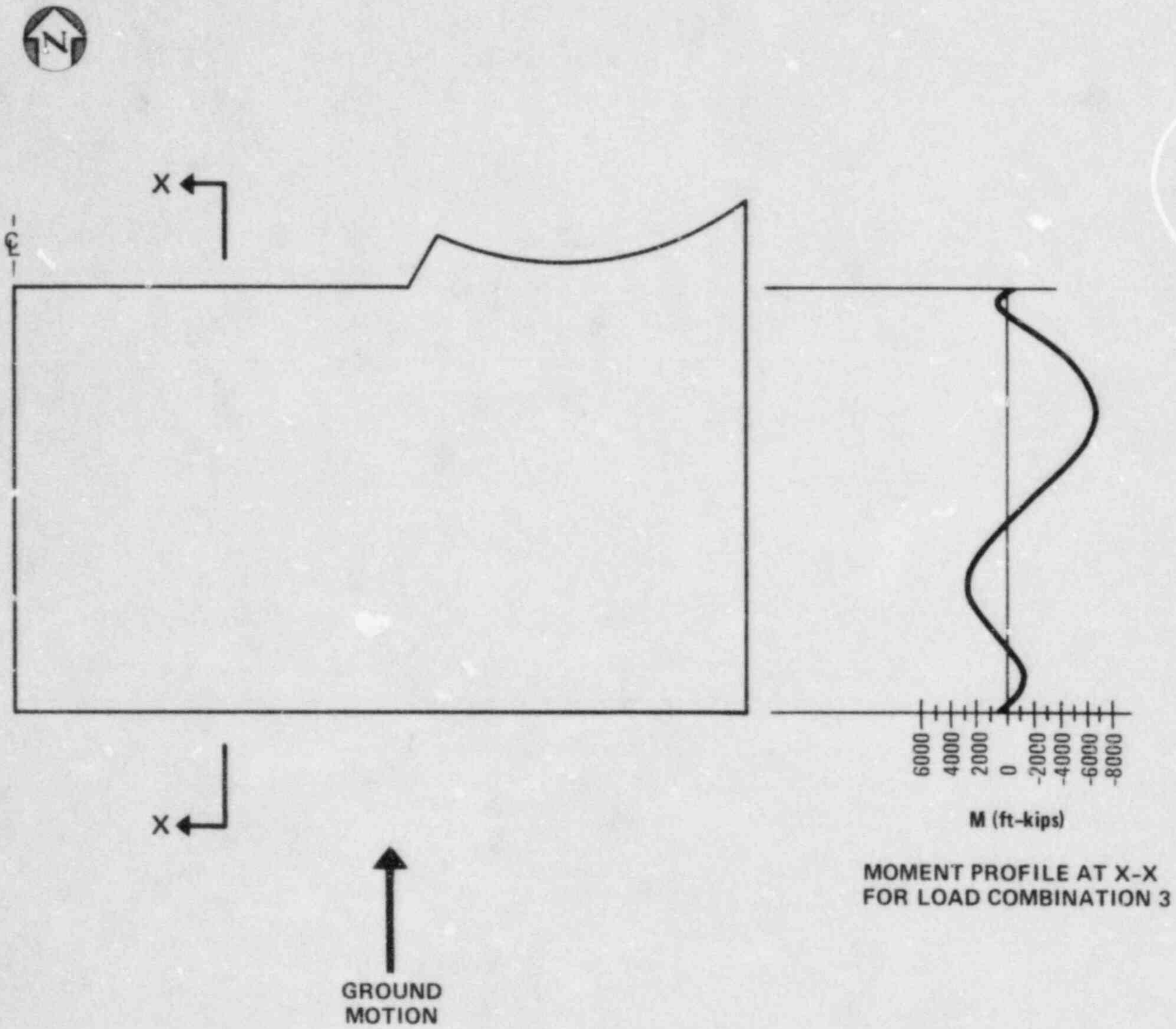
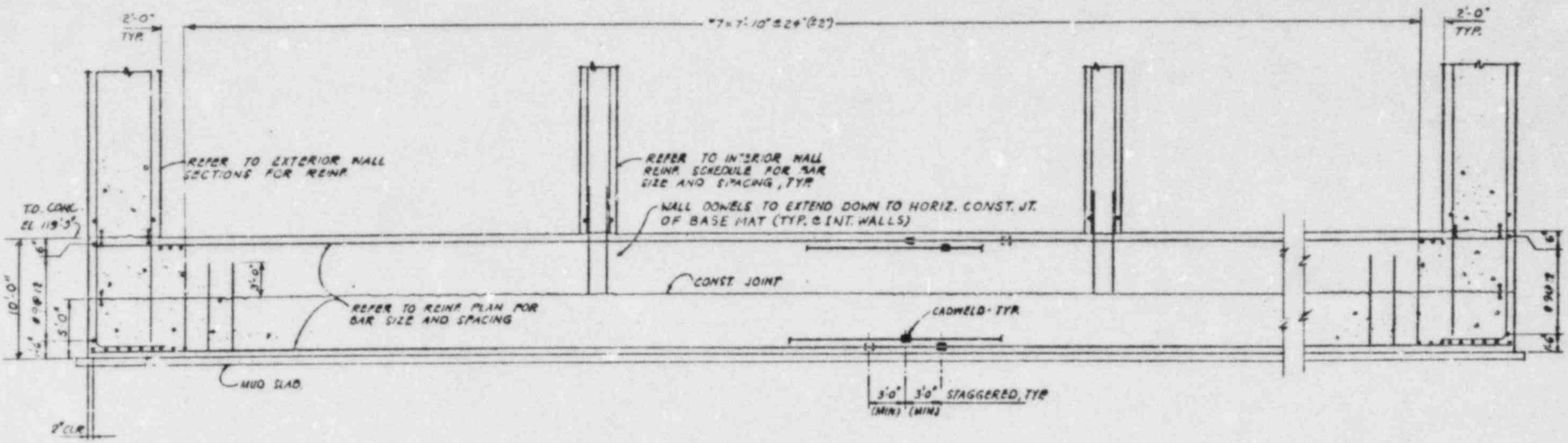
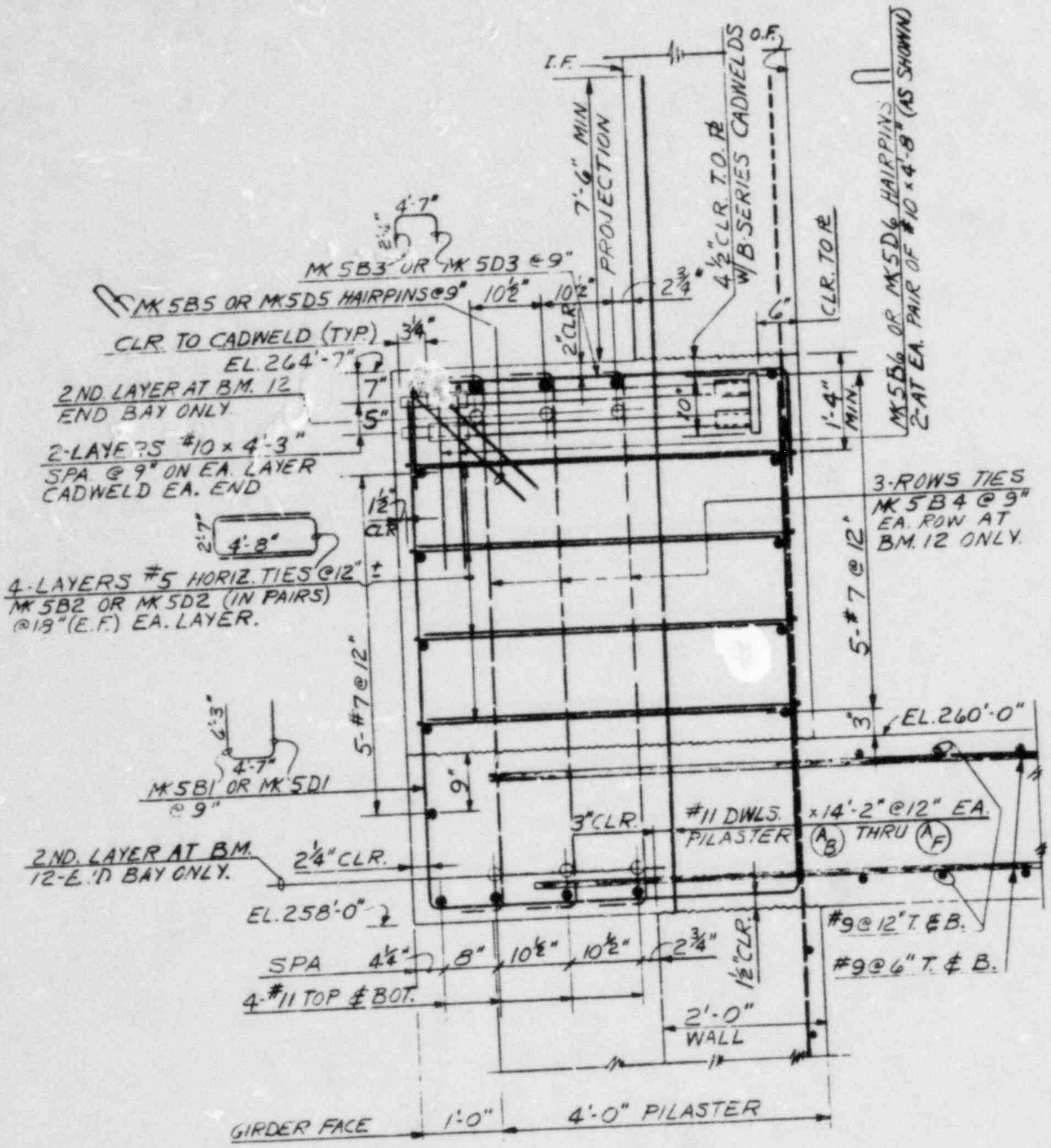


Figure 12
REPRESENTATIVE BASEMAT ANALYSIS RESULTS



SECTION

Figure 13
REPRESENTATIVE BASEMAT DETAILS



SECTION

Figure 14
CORBEL REINFORCING

APPENDIX A

DEFINITION OF LOADS

APPENDIX A

DEFINITION OF LOADS

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

A.1 NORMAL LOADS

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

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

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

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

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

A.2 SEVERE ENVIRONMENTAL LOADS

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

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

A.3 EXTREME ENVIRONMENTAL LOADS

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

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

A.4 ABNORMAL LOADS

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

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

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

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

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

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

APPENDIX B

LOAD COMBINATIONS

APPENDIX B

LOAD COMBINATIONS

B.1 STEEL STRUCTURES

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

B.2 CONCRETE STRUCTURES

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

TABLE B.1[a]

STEEL DESIGN LOAD COMBINATIONS
ELASTIC METHOD

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

---

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

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

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

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

TABLE B.2[a][f]

CONCRETE DESIGN LOAD COMBINATIONS
STRENGTH METHOD

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

---

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

APPENDIX C

DESIGN OF STRUCTURES FOR TORNADO MISSILE IMPACT

APPENDIX C

DESIGN OF STRUCTURES FOR TORNADO MISSILE IMPACT

C.1 INTRODUCTION

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

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

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

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

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

C.1.1 Procedures

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

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

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

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

d. Determining structural response considering other concurrent loading conditions.

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

C.2 LOCAL EFFECTS

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

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

C.2.1 Reinforced Concrete Elements

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

C.2.2 Steel Elements

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

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

where:

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

E_k = missile kinetic energy (ft-lb).

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

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.

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

where:

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

C.3 STRUCTURAL RESPONSE DUE TO MISSILE IMPACT LOADING

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

C.3.1 General

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

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

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

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

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

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

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

C.3.2 Structural Assessment

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

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

C.4 REFERENCES

1. Gwaltney, R. C., "Missile Generation and Protection in Light-Water-Cooled Power Reactor Plants," ORNL NSIC-22, Oak Ridge National Laboratory, Oak Ridge, Tennessee, for the USAEC, September 1968.

2. Rotz, J. V., "Results of Missile Impact Tests on Reinforced Concrete Panels," Vol 1A, pp 720-738, Second Specialty Conference on Structural Design of Nuclear Power Plant Facilities, New Orleans, Louisiana, December 1975.

TABLE C-1

DUCTILITY RATIOS (Sheet 1 of 2)

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

TABLE C-1

DUCTILITY RATIOS (Sheet 2 of 2)

Notes:

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

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

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

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

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

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

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

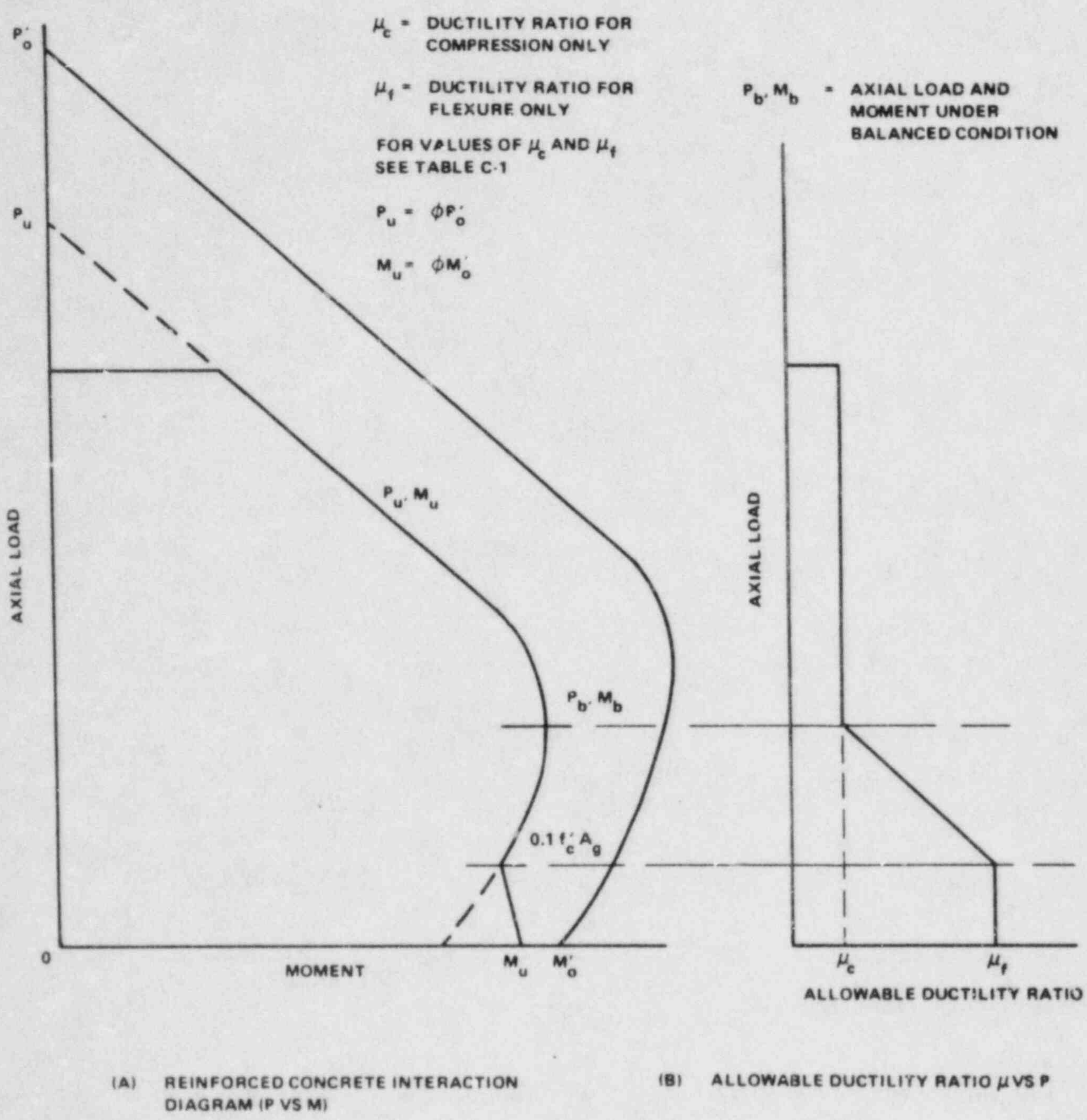


Figure C–1
MAXIMUM ALLOWABLE DUCTILITY RATIO
FOR REINFORCED CONCRETE SECTION
WITH BEAM–COLUMN ACTION