11/20/81

UNITED STATES OF AMREICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	
	)	
CONSUMERS POWER COMPANY	)	Docket Nos. 50-329 OM & OL
	)	50-330 OM & OL
(Midland Plant, Units 1 and 2)	)	

TESTIMONY OF DARL HOOD, JOSEPH KANE AND HARI SINGH
CONCERNING THE REMEDIAL UNDERPINNING OF THE AUXILIARY BUILDING AREA

Q.1. Please state your name and position.

A.1. My name is Darl Hood. I am a Senior Project Manager in the Division of Licensing, Office of Nuclear Reactor Regulation, U.S. Nuclear Regulatory Commission.

My name is Joseph Kane. I am a Principal Geotechnical Engineer in the Hydrologic and Geotechnical Engineering Branch, Division of Engineering, Office of Nuclear Reactor Regulation, U.S. Nuclear Regulatory Commission.

My name is Hari Singh. I am a Civil Engineer in the Geotechnical Branch of the Engineering Division, U.S. Army Corps of Engineers, North Central Division, Chicago.

Q.2. Have you prepared statements on your professional qualifications?

A.2. Yes, copies are attached. (Attachments 1, 2 and 3).

Q.3. Please state the nature of your responsibilities with respect to the Midland Plant.

A.3. I, Darl Hood, am the Project Manager for the Midland Plant application for operating licenses. I have served in that position from August 29, 1977, when the application for operating licenses was tendered to the NRC for acceptance review, up to the present time. My responsibilities include management of the Staff's environmental and radiological safety reviews. I am responsible for answers to Questions Q.5 through Q.7 and Q.20 in this testimony.

I, Joseph Kane, have served since November 1979 as the technical monitor for the interagency contractual agreement between the NRC and the U.S. Army Corps of Engineers, Detroit District (hereinafter the Corps). By this contract the Corps has been assisting the NRC Staff in the safety review of the Midland project in the field of geotechnical engineering. In addition to, and as a consequence of, my serving as contract technical monitor, I have become directly involved in the assessment of the adequacy of the remedial measures which have been proposed by Consumers to correct the plant fill settlement problem. I am responsible for answers to Questions Q.4, Q.8 through Q.10, and Q.19 in this testimony.

I, Hari Singh, became involved with the Midland plant in May 1980, when I was assigned the responsibility as the Corps' lead reviewer for the geotechnical aspects of the Midland plant. The U.S. Army Corps of Engineers and the U.S. Nuclear Regulatory Commission had signed an interagency agreement in September 1979. This agreement requires the Corps to provide technical assistance to the NRC on geotechnical engineering aspects for the foundation design of structures at the

Midland Plant. On May ⁻, 1980, I joined the Corps' team of engineers and geologists of ⁺ Geotechnical Section of the Detroit District, who were engaged in reviewing the foundation design of the plant. As the full-time lead reviewer, my responsibilities were to coordinate with all Corps reviewers, examine their comments, perform my own review, discuss comments with the Section and Branch Chiefs and prepare final letter reports for transmittal to the NRC. I am responsible for answers to Questions Q.11 through Q.18 in this testimony.

Q.4. What is the purpose of this testimony?

A.4. This testimony concerns the geotechnical engineering aspects of the underpinning in the Auxiliary Building area. The term "Auxiliary Building area" includes not only the Control Tower and Electrical Penetration Areas for both Units 1 and 2 but also the Feedwater Isolation Valve Pits for both Units 1 and 2. The testimony describes the history of the fill problem in this area up to the present status of review by covering the following topics:

(a) Brief description of the structures in the Auxiliary Building area affected by the plant fill problem, including the internal equipment of importance to safety.

(b) Description of the fill problem and selected remedial treatment (underpinning).

(c) Brief description of NRC earlier efforts and concerns regarding the fill problem at the Auxiliary Building.

(d) History and review comments on previously proposed remedial fixes for Auxiliary Building area.

(e) NRC consultant's request for borings and laboratory testing and the status of the engineering evaluation of this information.

(f) Evaluation and conclusions on current proposed remedial underpinning fix.

(g) Remaining safety issues.

Q.5. Describe the function and significance of the Auxiliary Building.

A.5. The Auxiliary Building is one of the principal buildings at the Midland Plant containing safety systems and necessary auxiliary support systems. It is a seismic Category I structure containing the control room, access control room, cable spreading rooms, engineered safety features systems, switchgear equipment, a portion of the radwaste equipment, and the facilities for fuel handling, storage and shipment. As shown on FSAR Figure 1.2-1 (Attachment 4), the Auxiliary Building is located north of the Turbine Building and between the two Containment Buildings which share its facilities.

As illustrated by FSAR Figure 3.8-51 (Attachment 5), the Auxiliary Building is subdivided into several areas, including at its southern end a "Control Tower Area." The north end of the Auxiliary Building includes the "Railroad Access Area" (also called the Railroad Bay). As illustrated by Attachments 6 and 7, the areas of the Auxiliary Building founded on plant area fill are the Railroad Bay (including the Liquid Radwaste Area shown in Attachment 5) to the north, and the Electrical Penetration Areas and the Control Tower to the south. The rest of the Auxiliary Building is founded on natural soil or on lean concrete backfill in localized areas. Consumers Power Company has proposed remedial action for the Electrical Penetration Areas and the Control Tower in the form of modifications to the foundation. As noted in Consumers' response to § 50.54(f), Request 12, the remedial action proposed for the Railroad Bay area is limited to the addition of a permanent areal dewatering system to eliminate liquefaction potential since the fill beneath this area includes medium to very dense sand, as well as concrete.

Equipment identification and location at various elevations beneath or within the Control Tower and Electrical Penetration Areas are shown on FSAR Figures 1.2-3, 1.2-5 through 1.2-10 and 1.2-13 (Attachments 8 through 15, respectively). The seismic Category I Auxiliary Building serves, in part, to protect its safety equipment and necessary support equipment from extreme environmental conditions, including earthquakes, tornadoes and floods.

The existing foundation design of the Auxiliary Building is described by FSAR Section 3.8.5.1.2 as follows:

> The Auxiliary Building is founded on reinforced concrete mat foundations at six different elevations as shown in Figure 3.8-61. [Attachment 16]. The figure shows the bottom elevations and thicknesses of the mat foundations at different areas. The major portion of the Auxiliary Building (between column lines A and H in the north-south direction and between column lines 5.6 and 7.4 in the east-west direction) rests on a 6 foot thick reinforced concrete mat, 158'-3" long and 79'-0" wide, founded in glacial till, with the bottom elevation at 562 feet. The southern portion of the Auxiliary Building, south of column line H, rests on a 5 foot thick reinforced concrete mat with the bottom elevation at 609 feet. It is founded on compacted fill. The elevations and the thicknesses of the mat foundations in the other areas are shown in Figure

3.8-61. All the mat foundations with their bottom elevations above 570 feet are founded on compacted fill. Figure 3.8-62 and 3.8-63 show the cross-sections of the foundations and typical reinforcement details.

Q.6. What are the Feedwater Isolation Valve Pits at the Midland plant?

A.6. The two Feedwater Isolation Valve Pits are seismic Category I, reinforced concrete structures which primarily enclose the main feedwater isolation valves and part of the associated main feedwater piping. As shown on FSAR Figures 1.2-5 (Attachment 9) and 1.2-13 (Attachment 15), and Attachment 17, the pits (or chambers) are located symmetrically at the sides of ea Containment Building and are adjacent to the Electrical Penetration Area o the Auxiliary Building, the Containment Tendon Service Shaft, and the Turbine Building. Each pit is C-shaped with the open end in contact with, but structurally separate from, the Containment Building. Access into the pit is from the top.

As shown in FSAR Figure 3.6-12 (Attachment 18) for Midland, Unit 1 (or similar FSAR Figure 3.6-13 for Unit 2), two main feedwater lines (one for each steam generator) enter each valve pit from the Turbine Building and exit the pit through the Containment (or Reactor Building) wall. The feedwater piping is anchored in the south wall of the valve pit. Inside the valve pit, each line is equipped with an isolation valve. No other safety-related equipment is located inside the valve pit. (Inside the Containment, a second isolation valve is also provided for each main feedwater line.) The valve pit protects its isolation valves and piping from tornado missiles and from the effects of any postulated pipe breaks within the Turbine Building. The valve pit design does not assume pipe breaks to occur within the valve pit; rather, special provisions described in FSAR Section 3.6.2.1.4 are utilized, including a program of augmented inservice inspection.

The pits for both Midland, Unit 1 and Midland, Unit 2 are founded on plant fill. Consumers proposes remedial action in the form of modified foundations due to the potential for settlement of the fill presently underlying the pits.

> Q.7. Is the main feedwater system important to safety?

A.7. Yes, some of the system is. Feedwater line isolation limits the energy release and the magnitude of Reactor Coolant System cooldown in the event of a main steam line break or main feedwater line break. The safety design bases for the main feedwater system are given in FSAR Section 10.4.7.1.1 as follows:

> SAFETY DESIGN BASIS ONE - Portions of the feedwater system within the containment and non-isolable portions between the containment penetration and the first anchor past the main feedwater isolation valves are designed to remain functional following a safe shutdown earthquake.
>
> SAFETY DESIGN BASIS TWO - The main feedwater system is designed to isolate the system automatically from the steam generators when required in order to mitigate the consequences of a steam line break accident or main feedwater line break.
>
> SAFETY DESIGN BASIS THREE - The main feedwater lines are designed so that failure in this piping outside of the containment will not prevent a safe shutdown of the unit.

The main feedwater isolation valves and all piping between them and the steam generators are designed to seismic Category I requirements. The main feedwater system is isolated from the steam generators by automatic closure of the feedwater isolation valves by a main steam line isolation signal generated by low pressure in either steam generator or a trip of a digital subsystem of the emergency core cooling actuation system.

Also, improper control of main feedwater after a reactor trip can lead to overcooling of the reactor coolant system, with a potential for loss of pressurizer level indication and challenge to the high pressure injection system. Eventually, control problems would lead to overfill of the once-through steam generator. To prevent steam generator overfill, a Class 1E high-high water level switch within the steam generator will also automatically initiate closing of the feedwater isolation valves.

Q.8. What is the fill problem in the Auxiliary Building area?

A.8. The problem is whether the fill materials beneath the foundations of these structures can provide adequate support and permit these structures to safely operate. Twelve soil borings completed in March of 1979 in the Auxiliary Building area (Attachment 19 for location) revealed that approximately the top fifteen feet of the fill materials placed under the foundations of the Electrical Penetration Areas and the Feedwater Isolation Valve Pits for both Units 1 and 2 were inadequately compacted. In addition, a void measuring approximately one foot in depth and of unknown lateral extent was encountered at elevation 590 beneath the concrete mudmat of the Control Tower in boring AX-9. After discovering the soft clay and loose sand in the foundation soils, Consumers decided to perform remedial underpinning and to add additional support to the Control Tower, Electrical Penetration Areas and Feedwater Isolation Valve Pits by extending the foundations of these structures through the fill, down to the competent natural glacial soils.

Q.9. Has there been any mainfestation of distress to these structures because of the fill problem?

A.9. Yes. The settlements which have been recorded since these structures were completed have been small, on the order of one-half inch, but some cracking of Auxiliary Building walls and floor has been observed and these cracks have been mapped. Based on discussions during a meeting held in Bethesda with Consumers on November 4, 1981, it is the Staff's understanding that Consumers will complete a thorough inspection of the structures to be underpinned just prior to beginning this work. The records on settlement and cracking established during that inspection can be verified by NRC's Regional Office, Region III and the records will serve as the reference data from which any changes caused by the underpinning operations can be evaluated.

Q.10. Characterize briefly the earlier efforts and concerns of the NRC staff regarding the fill problem at the Auxiliary Building area.

A.10. Although the fill settlement problem at the Diesel Generator Building area had been identified in August 1978, it was not until June of 1979 that the extent of the problem at the Auxiliary Building was made known to the NRC. (10 C.F.R. § 50.55(e), Interim Report No. 6, Transmitted to J. G. Keppler, Regional Director, NRC from S. H. Howell, Consumers Power Co., June 25, 1979). Once the NRC Staff realized the widespread extent of the plant fill problem and the likelihood that considerably more safety review effort was going to be required to evaluate the various proposed remedial treatments, the Staff began a search for obtaining technical assistance to supplement the limited staff resources. The Staff's efforts to obtain technical assistance in the area of geotechnical engineering resulted in an interagency agreement reached in September 1979 between the NRC and the U.S. Army Corps of Engineers. The consensus of the NRC Staff in late 1979 was that, although a basic concept for underpinning the Auxiliary Building had been submitted in June 1979, there were significant design considerations important to plant safety which had not been adequately addressed and acceptance criteria and their bases had not been established for either the installation or for the performance of the underpinning. The Staff's conclusions were later borne out by a comprehensive safety review completed by the Corps which specifically identified the unresolved issues. (July 7, 1980 Letter Report and April 18, 1981 Letter Report from P. McCallister, Corps of Engineers to R. E. Jackson and G. E. Lear, NRC). Some of the major issues identified in the Corps review are subsequently discussed in responses to Questions 11 and 12.

Q.11. Describe the various stages of design which have been developed by Consumers in arriving at the currently proposed remedial underpinning fix for the Auxiliary Building area.

A.11. The following paragraphs identify the significant design stages:

(a) The original remedial measure proposed by Consumers was reported in Interim Report 6, June 11, 1979, MCAR 24, 10 C.F.R. § 50.55(e). The original concept consisted of: (a) Pressure grouting to fill the void (Item 3 of Interim Report 6) under the mudmat of the Control Tower; (2) removing unsuitable backfill materials from beneath the Electrical Penetration Areas (EPA's) and the Feedwater Isolation Valve Pits (FWIVP's); and (3) replacing the soil backfill with lean concrete having a minimum compressive strength of 2000 lbs. per square inch.

(b) On July 18, 1979, in a meeting with the NRC Staff in Bethesda, Maryland, Consumers presented a revised plan for the remedial treatment of the Electrical Penetration Areas. The revised plan called for providing caissons at the extremities of the Electrical Penetration Areas for both Units 1 and 2. With the caisson supports at the ends, the EPA's would act as propped cantilevers on either side of the Control Tower, relieving the fill materials under the EPA's from the bearing pressures created by the structure loads, and transmitting these pressures both to the competent natural soils through the caissons and also to the foundation of the Control Tower. The remedial measure for the Feedwater Isolation Valve Pits remained the same as the original plan.

(c) On May 5, 1981, in a meeting with the NRC Staff and the Corps, Consumers presented another remedial action plan for the Electrical Penetration Areas. This plan consisted of providing mass concrete support at the extremities of both EPA's, instead of the previously proposed caissons. The mass concrete support was also to be extended under the nearby Turbine Building to spread the structure loads on larger foundation areas to keep the foundation pressure within permissible limits.

(d) On October 1, 1981, in a meeting with the NRC Staff and the Corps of Engineers, Consumers presented a further revised plan for the remedial measures for the Auxiliary Building (EPA's and FWIVP's) which had significant changes. This most recent plan calls for providing (1) a continuous remedial underpinning wall resting on undisturbed natural material, under the external walls of the Electrical Penetration Areas, the Control Tower, and the Feedwater Isolation Valve Pits; (2) three isolated supports beneath the Control Tower along an east-west line through the center of structure; and (3) underpinning wall supports to the external cross walls of the Control Tower, and also one intermediate cross wall support to each of the EPA's. Attachment 20, Appendix C, Figure 9 shows the details of this latest proposed remedial measure.

Q.12. Did the Corps of Engineers evaluate the various remedial measures proposed by Consumers?

A.12. Yes, except for the originally proposed plan of June 11, 1979 which had been superseded before the Corps became involved with the Midland project in September 1979.

Q.13. What review comments did the Corps of Engineers make on the previously submitted plans to underpin the Auxiliary Building area.

A.13. The Corps of Engineers review comments on past proposed plans to underpin the Auxiliary Building can be found in Attachments 3 and 4 of my previous testimony following Tr. 3488 prepared for the August 4 session of these ASLB hearings. A summary of those review comments in Attachments 3 and 4 follows:

1. Proposed Plan of July 18, 1979 Requiring Caisson Support.

a. The design information about caissons was inadequate. No information was provided concerning the capacity of each caisson to carry vertical and lateral loads and also the capacity of caissons to act as a group. Inadequate information was provided on estimates of settlement and lateral deflections, negative skin friction on the caissons due to future settlements of the fill materials in which the caissons were to be installed, bearing capacity of caissons and the factor of safety against shear failure of the soils supporting the caissons.

b. The soil parameters (shear strength of fill materials and glacial till) controlling the design of caissons were not furnished. The Corps of Engineers requested the Applicant through NRC to perform soil explorations and laboratory testing of representative soil samples to obtain shear strength parameters.

c. The ability of the Control Tower to safely carry the additional load imposed by underpinning the extremities of the EPA with caissons had not been addressed. This caisson proposal had the effect of transforming the continuously soil supported EPA structures into propped cantilever structures, fixed with the Control Tower at one end and supported on caissons on the other end. Consequently, approximately half of the loads of the EPA's (approximately 9000 kips) was going to be transferred to the Control Tower increasing the foundation pressure on the compacted fill supporting the Control Tower structure with the potential for additional settlements.

d. The procedures and criteria to be required in the field during the difficult installation operations had not been adequately addressed.

2. Proposed Plan of May 5, 1981 Requiring Mass Concrete Replacement.

a. Consumers did not furnish significant design information regarding this scheme and except for the brief verbal presentation on May 5, 1981, this proposed solution appeared not to be seriously considered.

Q.14. What is the Corps of Engineers evaluation of the current plan which was presented at the October 1, 1981 meeting?

A.14. The remedial measures currently under consideration to stabilize the Auxiliary Building area are described in Attachment 20. The Corps of Engineers has reviewed this technical report and associated appendices provided by Consumers and offers the following assessment:

1. Bearing capacity of underpinning walls.

a. The bearing capacity analysis using an average undrained shear strength of 6.6 ksf is not appropriate for the entire length of the underpinning wall. While this average value provides a conservative design for the underpinning walls which are adjacent to Boring No. COE-18, (tests on samples from Boring COE-18 shows shear strength more than 6.6 ksf), the 6.6 ksf shear strength results in an overestimation of the bearing capacity of the foundation soils supporting the underpinning walls adjacent to Boring No. COE-17. The soil samples taken from Boring COE-17 within the potential zone of influence under the footings of the underpinning walls have indicated shear strengths less than 6.6 ksf (shear strengths of 5.18 ksf and less). The factor of safety using the lower (5.18 ksf) shear strength should be determined and provided to the NRC.

b. The estimated factors of safety against the shear failure of foundation soils under dynamic loading for various underpinning walls have not been furnished.

c. The bearing capacity analysis and the resulting factors of safety for foundation soils under drained condition have not been furnished. It is advisable to verify the ultimate bearing capacity on the basis of drained shear test results in recognition of the high undrained strength which may not be permanently available.

2. Settlements.

a. The settlements for the proposed underpinning walls provided on Page 9 of Attachment 20 have not been demonstrated to be acceptable. The total settlement of foundation soils consists of three parts: (1) immediate settlement at constant volume, (2) consolidation settlement due to change in soil volume caused by expulsion of excess pore water, and (3) secondary settlement. For highly overconsolidated soils where settlement is primarily the result of recompression, the soil would behave elastically, and it would be reasonable to compute settlements using Young's modulus of the soil. However, such settlement computations do not include secondary setttlement; therefore, secondary settlements should be computed separately using coefficients of secondary consolidation which are added to the immediate settlement.

Consumers' computations for settlement appear to be based on the assumption that the glacial till soil is highly overconsolidated, and the settlements will be the results of recompression of foundation soils. However, Consumers has not computed and presented the preconsolidation pressures for the foundation soils to demonstrate that the foundation soils are highly overconsolidated. Therefore, whether the elastic approach used by the Applicant to compute settlements is applicable or not has not been demonstrated.

b. The method for computing secondary settlement was not presented in the technical report on underpinning (Attachment 20). It is the Corps' understanding that the Applicant has used coefficients of secondary consolidation, C , determined from the consolidation tests to determine the secondary settlement. However, as mentioned in answer to Q.17(3), the results of consolidation tests are questionable and the C determined from consolidation tests may not be appropriate for computing the secondary consolidation.

c. Settlement Monitoring During Construction.

The program proposed by Consumers to ensure stability of the existing structures (EPA's and Control Tower), during the period when foundation soils will be sequentially removed to install the remedial underpinning walls, consists of monitoring the settlements of the structures at critical points. The Applicant's monitoring program presented in the technical report (Attachment 20) has been reviewed by the Corps and the NRC Staff and their review comments were transmitted to

Consumers on October 30, 1981, by telephone. Attachment 21 is a record of the October 30, 1981 conference call and lists the specific review comments. Monitoring of settlements of the structures to be underpinned, and determination of acceptance criteria for settlements, during the construction are of paramount importance for preserving the structural integrity of the EPA's and Control Tower. Therefore, resolution of questions raised by the Corps and NRC Staff regarding the monitoring program is essential.

Q.15. Has the Corps of Engineers completed its review of the laboratory test results which were presented in Parts 1 and 2 technical reports by Consumer's consultant, Woodward-Clyde Consultants?

A.15. No. At the writing of this testimony the Corps has not received Part 2 of the reports which are entitled "Test Results, Foundation Soils, Auxiliary Building."

Q.16. Has the Corps completed its review of Part I report?

A.16. Yes. Part I was received by the Corps in the last week of September 1981 and contained the following information:

(1) Boring log information for Boring Nos. COE-17 and COE-18 which are the two borings completed in the Auxiliary Building area.

(2) Results of index tests of soils from these borings.

(3) Results of unconsolidated undrained (UU) tests.

(4) Results of consolidated undrained (CIU) tests.

(5) Results of consolidation tests.

(6) Backup materials for UU tests.

(7) Backup materials for CIU tests.

(8) Backup materials for consolidation tests.

Q.17. What review comments does the Corps have on the test results submitted in Part I?

A.17. Based on the Corps' evaluation of lab test results in Part I, the following comments are provided:

1. The drained shear strength parameters obtained from the CIU tests indicate that the shear strengths of the natural soils (at lower normal stresses) are lower than the undrained shear strengths (Su = 5.18 ksf and more); therefore, it is our opinion that the bearing capacity of foundation soils should be checked using the drained shear strength from the CIU tests.

2. Preconsolidation pressures and the over consolidation ratio for the natural soils in the zone of influence beneath underpinning walls have not been determined. Therefore, use of the elastic approach to compute the settlements, which is applicable in cases where soil is highly over-consolidated and the settlement would be the result of recompression, has not been resolved. The absence of volume change during UU tests and the development of zero to slight negative value for pore pressure parameter, A, at failure loads indirectly indicates that the soil is moderately over-consolidated; however, their definite values were not provided.

3. The e-log p curves and data for the consolidation tests indicate that the inundation of consolidation samples was not performed until the

21 tsf stress increment was placed. This appears to have considerable influence on the shape of e-log p curves and as such the results of the consolidation tests are questionable. Normal testing standards require test specimens to be inundated immediately after applying the first load increments. If swelling occurs with the addition of water, additional loading is applied until swelling ceases. An explanation for this deviation from normal testing procedures should be provided.

Q.18. What are the conclusions of the Corps of Engineers following its engineering evaluation of the latest proposal (Attachment 20) for underpinning the Auxiliary building area?

A.18. The conclusions of the Corps of Engineers' evaluation are as follows:

1. The overall concept of the currently proposed remedial measures appear to be satisfactory. The remedial measures, if built satisfactorily, would transmit the structure loads to the competent soil layers, relieving the fill materials from any external load. Underpinning will also eliminate the problem of overstressing the foundation soils of the Control Tower, which would have been possible with the previously proposed caisson support. However, a proper foundation design based on actual soil parameters, as mentioned in answer to Q.14.1.a is essential and resolution of the outstanding issues identified in Table A.20 and in this testimony is required.

Q.19. What are the conclusions of the NRC Geotechnical Engineering Staff in their evaluation of the proposed remedial underpinning of the Control

Tower, Electrical Penetration Areas and Feedwater Isolation Valve Pits?

A.19. The document provided by Consumers which is the major source of technical information on the currently proposed remedial underpinning fix was submitted as Enclosure 3 to their letter of September 30, 1981. Enclosure 3 is Attachment 20 and is entitled "Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits." This report provides both design and construction requirements for the proposed fix and in matters related to geotechnical engineering covers the following topics:

a. Engineering soil properties adopted in underpinning design.

b. Groundwater control needed to stabilize foundation soils during excavation and underpinning construction.

c. Criteria on safe bearing capacity, absolute and differential settlements and the extent of tolerable cracking.

d. Plans and frequency of monitoring efforts to detect vertical and lateral movements during construction and permanent transfer of loads to the newly installed underpinning elements.

e. Sequence of underpinning construction operations.

The Staff evaluation of the information provided in Enclosure 3 which was supplemented by discussion in meetings with Consumers on October 1 and 2 and on November 4, 1981 permits us to make the following conclusions:

1. The continuous remedial underpinning wall fix is a viable, positive solution to the fill problem in the Auxiliary Building area.

2. The planned foundation underpinning design and construction sequence now being developed by Consumers, if properly carried out in the field, will provide a stable and safe foundation for the involved structures.

3. The criteria and/or monitoring requirements on bearing capacity, soil modulus, settlement and groundwater which are in the final stages of resolution between the NRC Staff and Consumers should provide, when fully resolved, reasonable assurance that the underpinning work will be completed in a manner that will result in a safe foundation and permit operation of the Auxiliary Building without undue risk to the health and safety of the public.

Q.20 Is the NRC Staff prepared at this time to concur with Consumers poroceeding with the remedial underpinning fix at the Auxiliary Building?

A.20 Yes. The NRC Staff is prepared to concur with the start of underpinning, subject to the successful completion by Consumers and approval by the Staff of the work required in the following special license conditions listed in Table A.20. Each of the proposed license conditions listed for a specific milestone will be required to be completed and approved prior to beginning the construction work covered by that milestone.

Table A.20 includes the required special license conditions resulting from both geotechnical and structural engineering reviews. The information and conditions associated with the structural engineering review are further discussed in the testimony of Frank Rinaldi.

Table A.20

Construction Milestone	Date Information Available for Staff Review	Requested Starting Date of Construction Milestone
1. Install Vertical Access Shaft to El. 609 and Complete Freeze Wall Installation.	No submittal required	12/29/81

Proposed Special License Condition: None

Construction Milestone	Date Information Available for Staff Review	Requested Starting Date of Construction Milestone
2. Activate Freezing of Soil along Freeze Wall Alignment	12/15/81	2/1/82

Proposed Special License Conditions:

2a. Provide documentation demonstrating the Freeze Wall, when activiated, will not adversely affect seismic Category I structures, conduits and pipes by causing ground heave or resettlement upon unfreezing.

2b. Provide a plan, with established criteria and basis, for field monitoring of the effects of the Freeze Wall. The required plan will include a commitment to monitor both vertical and lateral movements at a minimum of four locations where safety related structures and utilities could potentially be affected.

2c. Provide responses for questions identified in Attachment 21 except for Questions 9, 18, 23, 25, 28 and 30.

2d. Provide responses for review concerns identified in answers to questions 14 and 17 of this testimony.

Construction Milestone	Date Information Available for Staff Review	Requested Starting Date of Construction Milestone
3. Extend Vertical Access Shaft below El. 609 and begin to remove soil foundation support from beneath Feedwater Isolation Valve Pit.	1/15/82	2/15/82

Proposed Special License Conditions:

3a. Provide design analysis for temporarily supporting the Feedwater Isolation Valve Pits (FIVP) on beams extending from the Buttress Access Shaft to the Turbine Building. The design will identify actual loads and displacements and demonstrate the adequacy and safety of the temporary support system.

3b. Provide an acceptable monitoring program with criteria for avoiding adverse impact on FIVP.

Construction Milestone	Date Information Available for Staff Review	Requested Starting Date of Construction Milestone
4. Begin drift excavation beneath the Turbine Building.	1/15/82	2/15/82

Proposed Special License Conditions:

4a. Provide design analysis (including supporting calculations, drawings and specifications) which evaluates the anticipated undermining and temporary construction loading on the Turbine Building at this stage. The analysis will be required to demonstrate an acceptable margin of safety for the Turbine Building to safely carry the imposed temporary construction loads so as to avoid adverse impact on the adjacent Auxiliary Building.

4b. Provide an acceptable monitoring program for affected Category I structures, conduits and pipes with criteria and basis for this construction stage. Criteria basis should describe how movements to be measured are related to code allowable stresses and allowable strains.

4c. Provide documentation demonstrating the adequacy of the final permanent support system along the north side of the Turbine Building in safely providing long-term support for the Turbine Building without adversely impacting the Auxiliary Building.

4d. Provide responses for questions 9, 25 and 30 which are identified in Attachment 21.

Construction Milestone	Date Information Available for Staff Review	Requested Starting Date of Construction Milestone
5. Begin removal of soil foundation support from beneath Auxiliary Building.	2/1/82	4/1/82

Proposed Special License Conditions:

5a. Provide design analysis (including supporting calculations, drawings and specifications) which evaluates the temporary support system for the Auxiliary Building at appropriate sequential stages of excavation and jacking. The design analysis will be required to demonstrate acceptable margins of safety at the various stages of temporary construction.

5b. Provide an acceptable monitoring program with criteria and basis for temporary conditions of loading at this stage of construction.

5c. Provide responses for questions 18, 23 and 28 which are identified in Attachment 21.

5d. Provide design analysis (including supporting calculations, drawings and specifications) demonstrating the adequacy of the installed temporary post-tensioning system.

5e. Provide an engineering evaluation of all cracks (existing and new) and propose a plan for the detailed evaluation of through cracks.

6. Begin construction of permanent underpinning wall.	5/17/82	11/1/82

Proposed Special License Conditions:

6a. Provide design analysis (including supporting calculations, drawings and specifications) which evaluates the permanent underpinning structure. The design analysis will be required to address all load combinations including stability under seismic loading.

6b. Provide results of the evaluation of through cracks.

6c. Provide an acceptable monitoring program with criteria and basis for long-term plant operation condition.

LIST OF ATTACHMENTS

ATTACHMENT NO.	DESCRIPTION
1	Professional Qualifications - D. Hood
2	Professional Qualifications - J. Kane
3	Professional Qualifications - H. Singh
4	FSAR Fig. 1.2-1 - Station Arrangement
5	FSAR Fig. 3.8-51 - Auxiliary Building Plans
6	Auxiliary Building Plan
7	Auxiliary Building Sections
8	FSAR Fig. 1.2-3 - Equipment Location
9	FSAR Fig. 1.2-5 - Equipment Location
10	FSAR Fig. 1.2-6 - Equipment Location
11	FSAR Fig. 1.2-7 - Equipment Location
12	FSAR Fig. 1.2-8 - Equipment Location
13	FSAR Fig. 1.2-9 - Equipment Location
14	FSAR Fig. 1.2-10 - Equipment Location
15	FSAR Fig. 1.2-13 - Equipment Location
16	FSAR Fig. 3.8-61 - Auxiliary Building Foundation
17	Fig. 13-12 - Feedwater Isolation Valve Pit
18	FSAR Fig. 3.6-12 - Feedwater and Condensate System
19	Fig. No. 67 - Plant Area Boring Plan
20	September 30, 1981 Letter with Enclosure 3
21	Record of October 30, 1981 Telephone Conversation

ATTACHMENT 1

DARL S. HOOD

OFFICE OF NUCLEAR REACTOR REGULATION
U.S. NUCLEAR REGULATORY COMMISSION

PROFESSIONAL QUALIFICATIONS

I am a Senior Project Manager in the Division of Licensing, Office of Nuclear Reactor Regulation. I am responsible for managing licensing activities by the Commission with respect to Midland Plant, Units 1 and 2.

I have served in the position of Project Manager with the Commission since August 1976. This position provides for the managing of radiological safety reviews of applications for licenses and authorization to construct or operate light water nuclear power plants. As of April 1980, the position also provides for the managing of the environmental reviews of such applications. I assumed responsibility for Midland Plant, Units 1 and 2, when the application for operating licenses was tendered in August 1977. Other nuclear plants for which I have previously served in this capacity are the standardization design of Westinghouse which is designated RESAR-414 (Docket STN50-572), Catawaba Nuclear Station, Units 1 and 2 (Dockets 50-413 and 50-414), and River Bend Station, Units 1 and 2 (Dockets 50-458 and 50-459).

Between June 1969 and August 1976 I held two sequential positions within the Nuclear Power Systems Division of Combustion Engineering, Inc. (C-E) at Windsor, Connecticut. After March, 1973, I was Assistant Project Manager for the Duke Power Project. This position provided assistance in directing all efforts by C-E to design, fabricate, purchase and license the nuclear steam supply systems, reactor core, and associated auxiliary systems for Cherokee Units 1, 2 & 3 and Thomas L. Perkins Units 1, 2 & 3. The position assured that all aspects of the contracts were met and that safe and reliable systems were provided to the required schedule and at a reasonable profit to C-E. I assisted Duke Power in preparing the Preliminary Safety Analysis Report (PSAR) and provided for all C-E licensing support for these units. I also provided coordination of all other nuclear plants referencing the C-E Standard Safety Analysis Report to assure compatibility with C-E standard reference design. Until March, 1973, I was a Project Engineer in C-E's Safety and Licensing Department and was responsible for licensing of nuclear power plants. I coordinated the preparation of the Millstone Unit 2 PSAR and FSAR and the Calvert Cliffs Units 1 & 2 FSAR and interfaced with NRC, the utility, architect engineer and all C-E functional departments on licensing support matters. I ensured that NRC criteria, standards, and guides were incorporated into the nuclear steam supply system design.

Between August 1966 and June 1969, I was a Nuclear Safety and Radiation Analysis Engineer in the Nuclear Safety Unit, Nuclear Division of the Martin Marietta Corporation at Baltimore, Maryland. The purpose of this position was to perform hazard evaluations for nuclear power sources applied in space missions. My primary duty was to determine public exposure to radiation for malfunctions occurring during the intended mission. I also determined means by which the hazard potential for nuclear space systems could be mitigated to the extent that nuclear safety criteria were met. I conducted research with regards to the development of suitable criteria for permissable exposure levels and their probabilities, taking into account the dependence of acceptable risk on the benefit to be derived. My primary assignment was with the SNAP 29 (Systems for Nuclear Auxiliary Power) project. My evaluations of this nuclear power source included the formulation and application of computerized models for the transport of fuel released at high altitudes, in deep ocean and in shallow waters. I derived models for these release areas to incorporate the activity into human food chains and determined the expected ingestion dose, the number of people involved and the exposure probabilities. Inhalation dose was determined for radioactive fallout from the high-altitude release.

Between February 1965 and August 1966 I was a Nuclear Quality Control Engineer within the Electric Boat Division of General Dynamics at Groton, Connecticut. The purpose of this position was to provide control of quality for naval reactor systems, components, and shielding during the construction or overhaul of submarines by this shipyard. My primary area of responsibility was shielding. Duties included establishing procedures for the inspection of fabrication and installation of lead and polyethylene shielding, and resolving problems in complying with these or other shielding procedures. The position required a knowledge of nuclear theory, S5W systems design, Bureau of Ships contract and design requirements, non-destructive testing techniques, and quality control requirements.

Between November 1963 and February 1965, I was an Aeronautical Engineer for Nuclear Propulsion and Power at the George C. Marshall Space Flight Center, National Aeronautics and Space Administration in Huntsville, Alabama. I performed investigations of the nature and magnitude of the nuclear radiation environment, shielding systems and safety systems associated with proposed nuclear space vehicles for candidate space missions.

Between November 1963 and college graduation in 1962, I held various positions including chief of a missile electronics training unit at Redstone Arsenal, Alabama; student at the U.S. Army Signal Officer's Orientation Course at Fort Gordon, Georgia; and Marine Engineer for ordinance and special weapons within the Design Division of the Norfolk Naval Shipyard, Portsmouth, Virginia.

I received a Bachelor of Science Degree in Nuclear Engineering from North Carolina State University in 1962. I am a member of the Health Physics Society.

ATTACHMENT 2

PROFESSIONAL QUALIFICATIONS AND EXPERIENCE

NAME: Joseph D. Kane

ADDRESS: 7421 Miller Fall Road
Derwood, MD 20855

EDUCATION: B.S. Civil Engineering 1961
Villanova University

M.S. Civil Engineering 1973
Villanova University

Post-degree studies, Soils and Foundation Engineering
University of California 1972
University of Maryland 1978

PROFESSIONAL REGISTRATION:

Registered Professional Engineer (1966) - Pennsylvania 12032E

PROFESSIONAL SOCIETY:

American Society of Civil Engineers

EMPLOYMENT POSITIONS:

February 1980 - Present	Principal Geotechnical Engineer U.S. Nuclear Regulatory Commission
May 1977 - February 1980	Geotechnical Engineer U.S. Nuclear Regulatory Commission
October 1975 - May 1977	Soils Engineer U.S. Nuclear Regulatory Commission
August 1973 - October 1975	Supervisory Civil Engineer Chief, Soils Design Section U.S. Army Corps of Engineers Philadelphia District
January 1963 - August 1973	Civil Engineer Soils Design Section U.S. Army Corps of Engineers Philadelphia District
January 1962 - January 1963	Design Engineer McCormick - Taylor Associates Philadelphia, Pa.

PROFESSIONAL EXPERIENCE SUMMARY:

1975 to Present

In NRC Division of Engineering, Geotechnical Engineering Section, Mr. Kane has specialized in soil mechanics and foundation engineering. Experiences in this position have included the following:

a. Evaluation of the foundation adequacy of proposed sites for nuclear facilities with respect to design and operational safety. This work has included evaluation of geotechnical, soils and rock mechanics, foundation and earthquake engineering related aspects. The results of this review effort are summarized in a safety evaluation report for each of the proposed facilities which have included nuclear power plants, nuclear fuel reprocessing plants and uranium mill tailings waste systems.

b. Serving as a technical adviser for soil and foundation engineering related aspects in the development of regulatory guides, acceptance and performance criteria that are intended to assure construction and operational safety of nuclear facilities.

c. Serving as a technical representative for the Office of Nuclear Reactor Regulation on the NRC Advisory Group concerned with federal dam safety.

d. Serving as an instructor for the Office of State Programs in the training of state personnel who are responsible for construction and operational inspections of uranium mill tailings embankment retention systems.

1963 to 1975

During this period Mr. Kane was employed with the U.S. Army Corps of Engineers, Philadelphia District and attained the position, Chief, Soils Design Section, Foundations and Materials Branch, in 1973. Professional experiences with the Corps of Engineers have included the following:

a. The embankment and foundation design of four large multi-purpose earth and rockfill dams with appurtenant structures (spillways, inlet and outlet structures, control towers, flood protection facilities, etc.). Responsibilities ranged from the initial planning of subsurface investigations to select the most feasible sites through all design stages which were culminated in the final preparation of construction plans and specifications. This work included planning and evaluation of laboratory testing programs, studies on slope stability, seepage control and dewatering systems, settlement, bearing capacity, liquefaction embankment safety instrumentation and slope protection.

b. Served as a technical consultant to field offices charged with construction inspections for assuring completion of structures in compliance with design analysis and contract specifications. Participated in the development of needed modifications during construction whenever significant changed site conditions were uncovered.

c. Directed the efforts of engineers in the Soils Design Section in other fields of civil work projects that included the embankment and foundation design of levees, waterfront pile supported structures and disposal basins for the retention of hydraulic dredge waste.

1962 to 1963 — Served as design and project engineer for private consulting firm. This work included the design of large federally funded highways, a race track and various structures constructed to provide a Pennsylvania State park marina.

HONORS AND AWARDS:

High Quality Award	1972
Outstanding Performance Award	1978

ATTACHMENT 3

STATEMENT OF PROFESSIONAL QUALIFICATIONS
OF HARI NARAIN SINGH, P.E.

Name: Hari Narain Singh

Address: 855 Hinman Avenue
Evanston, Illinois 60202

Professional Licenses:

(1) Registered Structural Engineer - Pennsylvania - 1970, 16552E.

(2) Registered Civil Engineer - Pennsylvania - 1978, 16552E.

Education:

(1) B.S. (Civil) - 1956 - University of Patna, India
(2) M.S. (Civil) - 1969 - University of Colorado, Boulder, U.S.A.

Completed 30 additional semester hours beyond M.S. degree.

(3) (Geotechnical) - Wayne State University, Detroit (Presently working for Ph.D. degree).

Professional Experience:

A. August 1981 to Present: Civil Engineer, U.S. Army Corps of Engineers, North Central Division, Chicago, Illinois.

B. October 1978 to July 1981: Civil Engineer, U.S. Army Corps of Engineers, Detroit, Michigan.

C. April 1978 to September 1978: Civil Engineer (bridges & foundation) Arizona State Highway Department, Phoenix, Arizona.

D. March 1970 to March 1978: Civil Engineer, Pennsylvania Department of Transportation, Franklin, Pennsylvania 16323.

E. September 1965 to September 1969: Graduate student and Research Assistant, University of Colorado, Boulder, U.S.A.

F. May 1959 to July 1965: Assistant Professor of Civil Engineering, Department of Industries, Government of Bihar State, India. Posted at the Ranchi School of Engineering (1959-1961) and the Regional Institute of Technology, Jamshedpur, India.

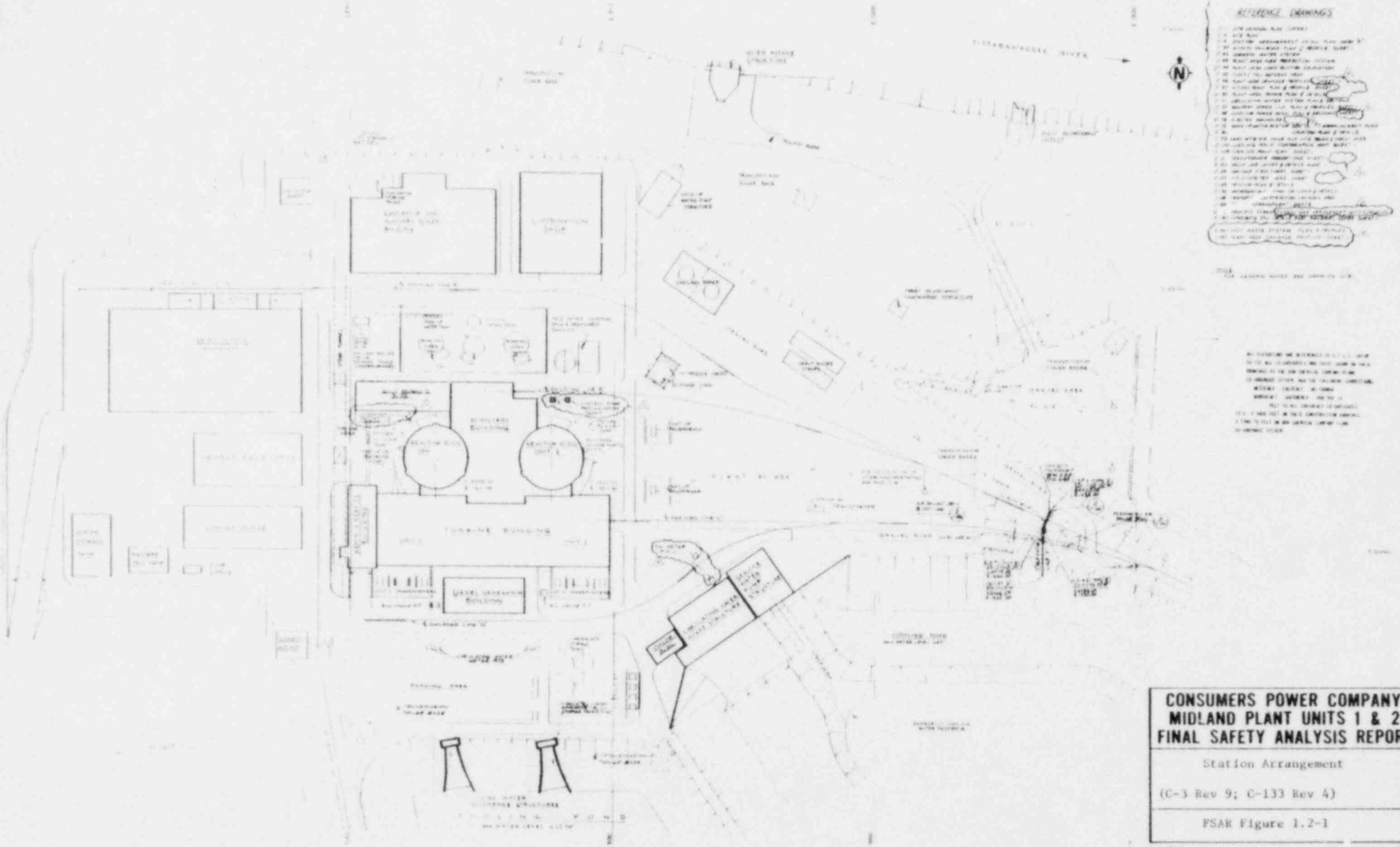
G. April 1958 to April 1959: Assistant Civil Engineering, Government of India (Tripura Administration), India.

H. July 1956 to April 1959: Engineer Assistant (Civil), Government of Bihar State, India.

Summary of Experience:

Twenty-four (24) years experience in civil engineering activities which include teaching, research, design, construction and maintenance. Completed design and reviewed design for more than fifty (50) bridge structures and their foundations. Carried out soil explorations and foundation investigations for structures.

ATTACHMENT 4



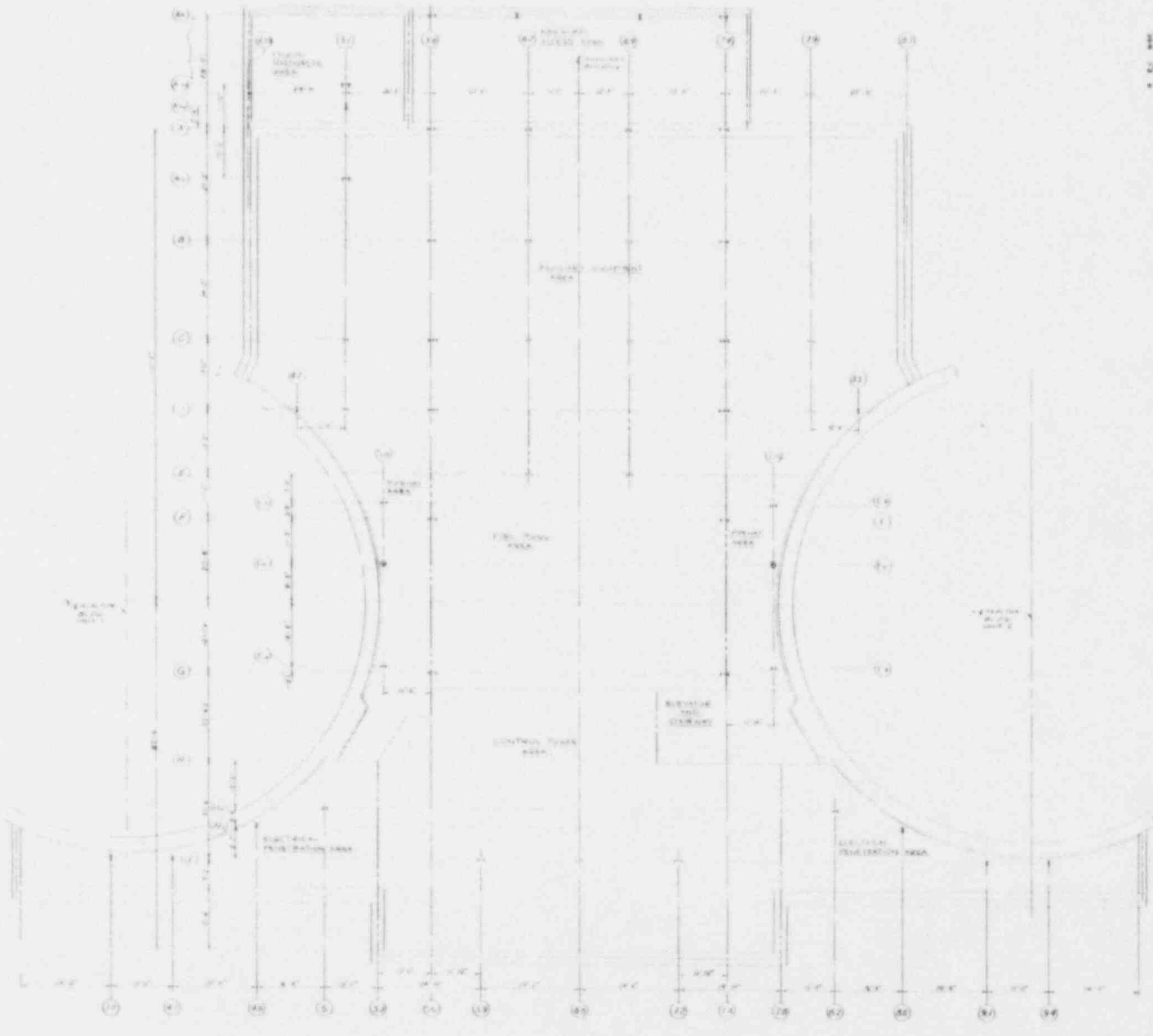
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

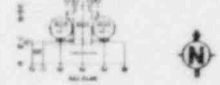
Station Arrangement

(C-3 Rev 9; C-133 Rev 4)

FSAR Figure 1.2-1

ATTACHMENT 5





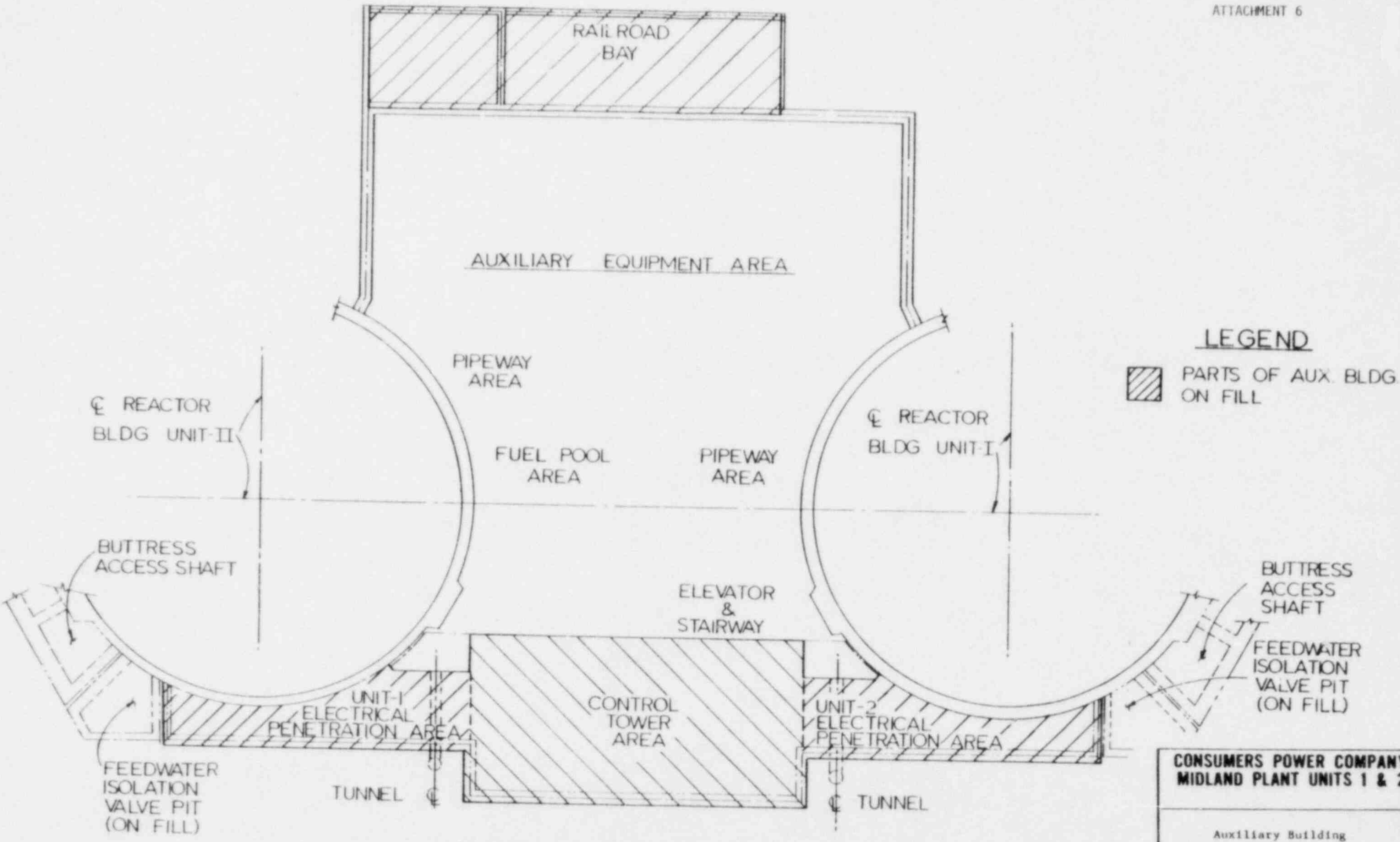
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Auxiliary Building Plans

(C-239, Rev 7)

FSAR Figure 3.8-51

ATTACHMENT 6

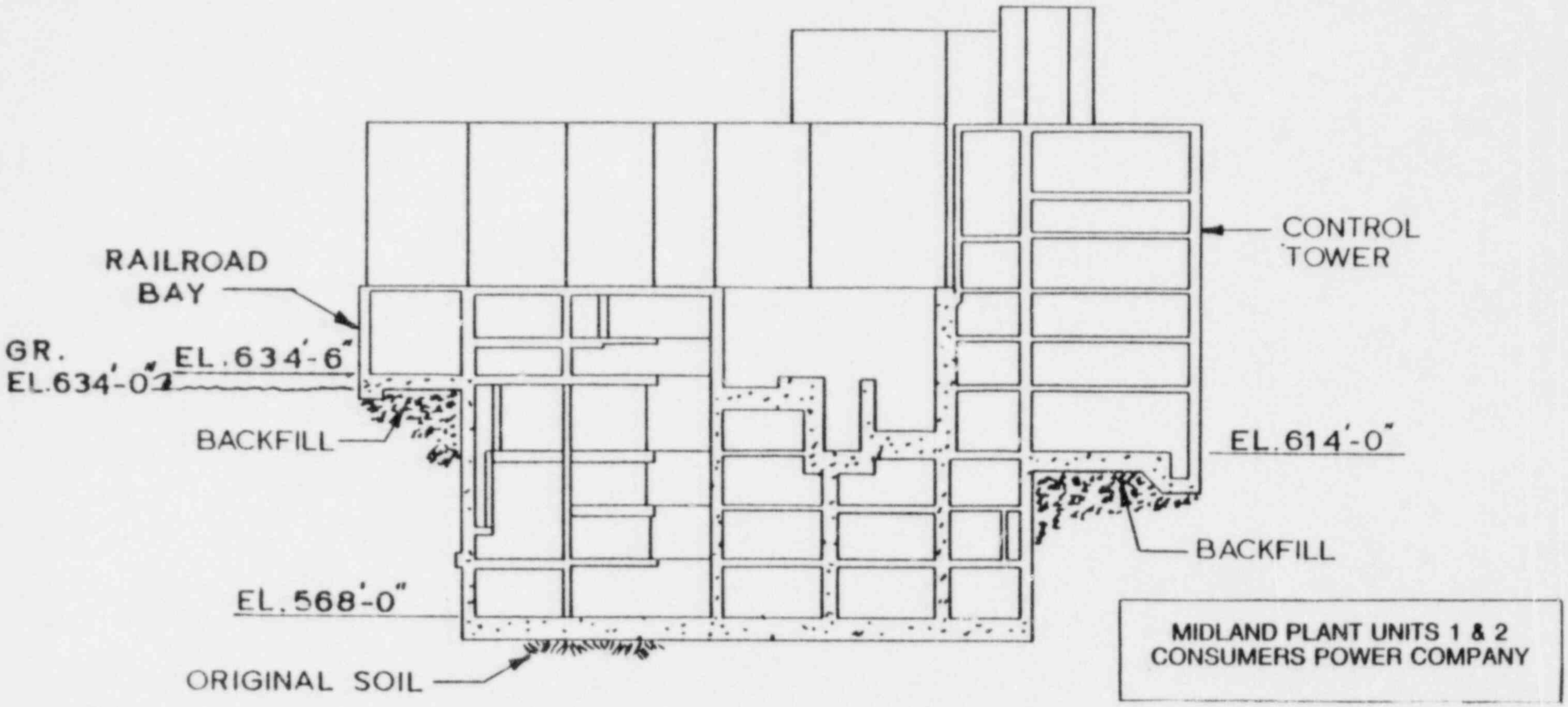


AUXILIARY BUILDING PLAN

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

Auxiliary Building Plan

FIGURE 1



TYPICAL SECTION (LOOKING EAST)

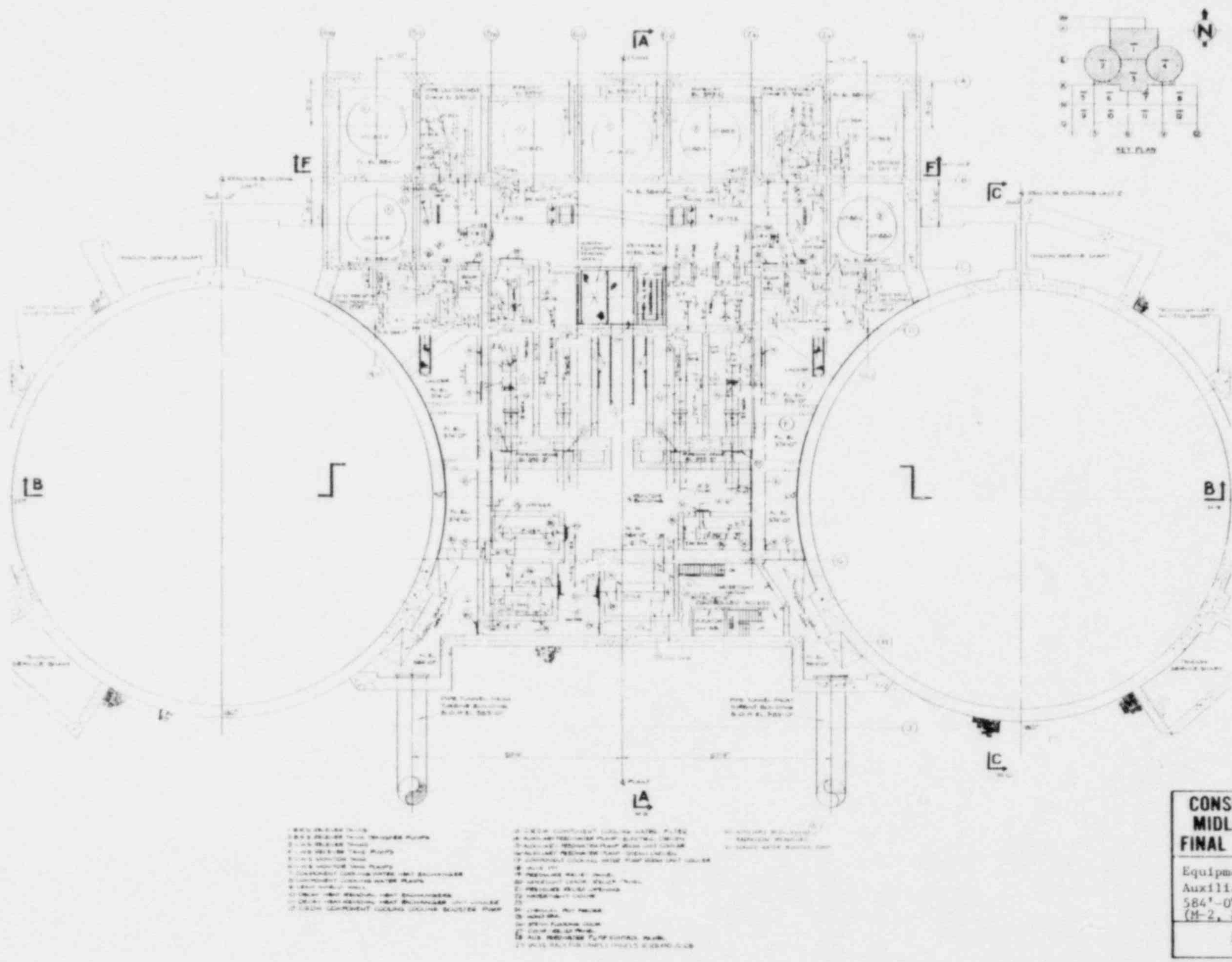
ATTACHMENT 7

AUXILIARY BUILDING

SECTIONS

CONSUMERS POWER COMPANY MIDLAND PLANT UNITS 1 & 2
Auxiliary Building Sections
FIGURE 2

ATTACHMENT 8



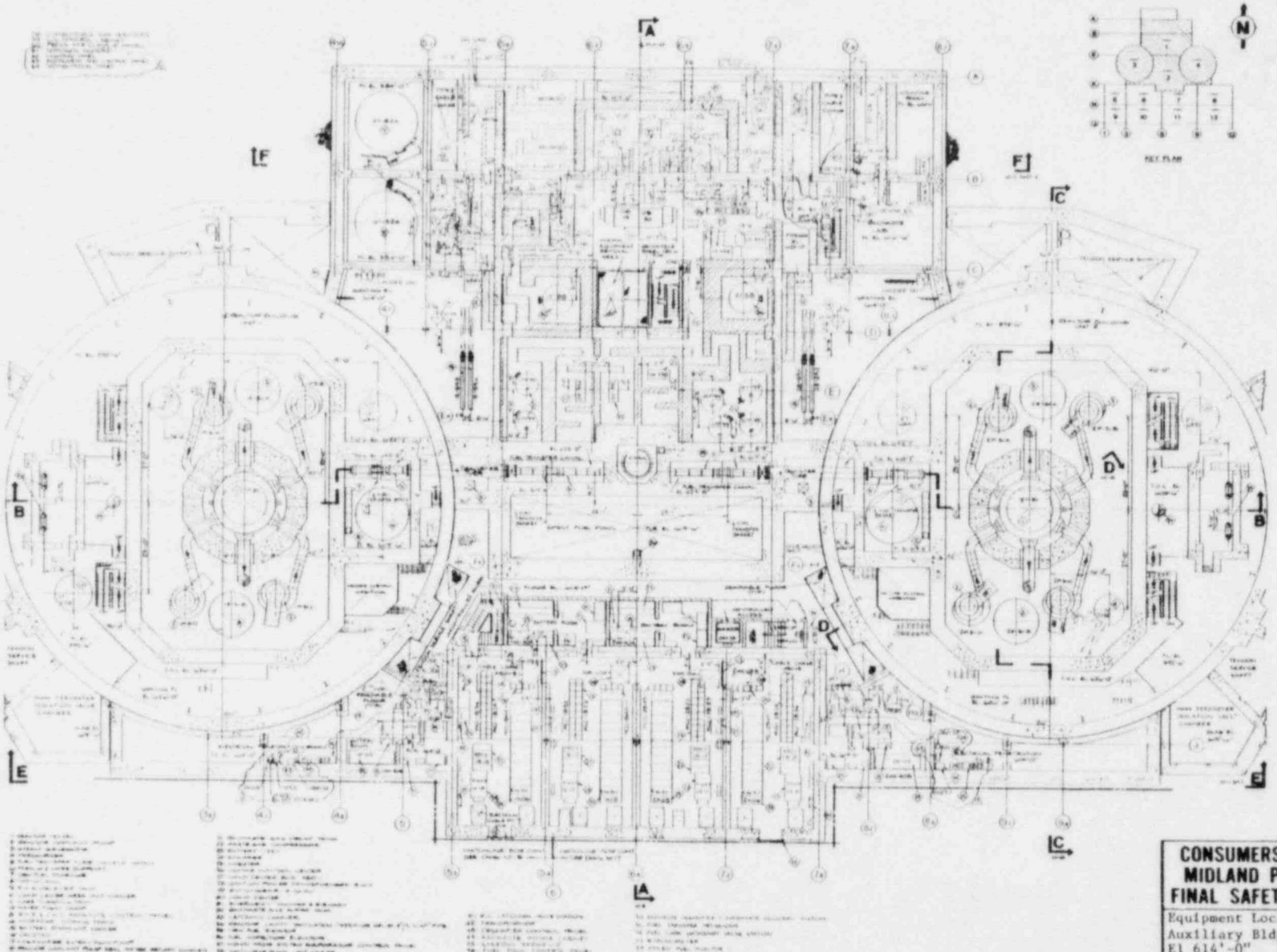
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Equipment Location - Reactor & Auxiliary Bldgs. Plan at El 584'-0"
(M-2, Rev 12)

FSAR Figure 1.2-3

1/81 Revision 32

ATTACHMENT 9



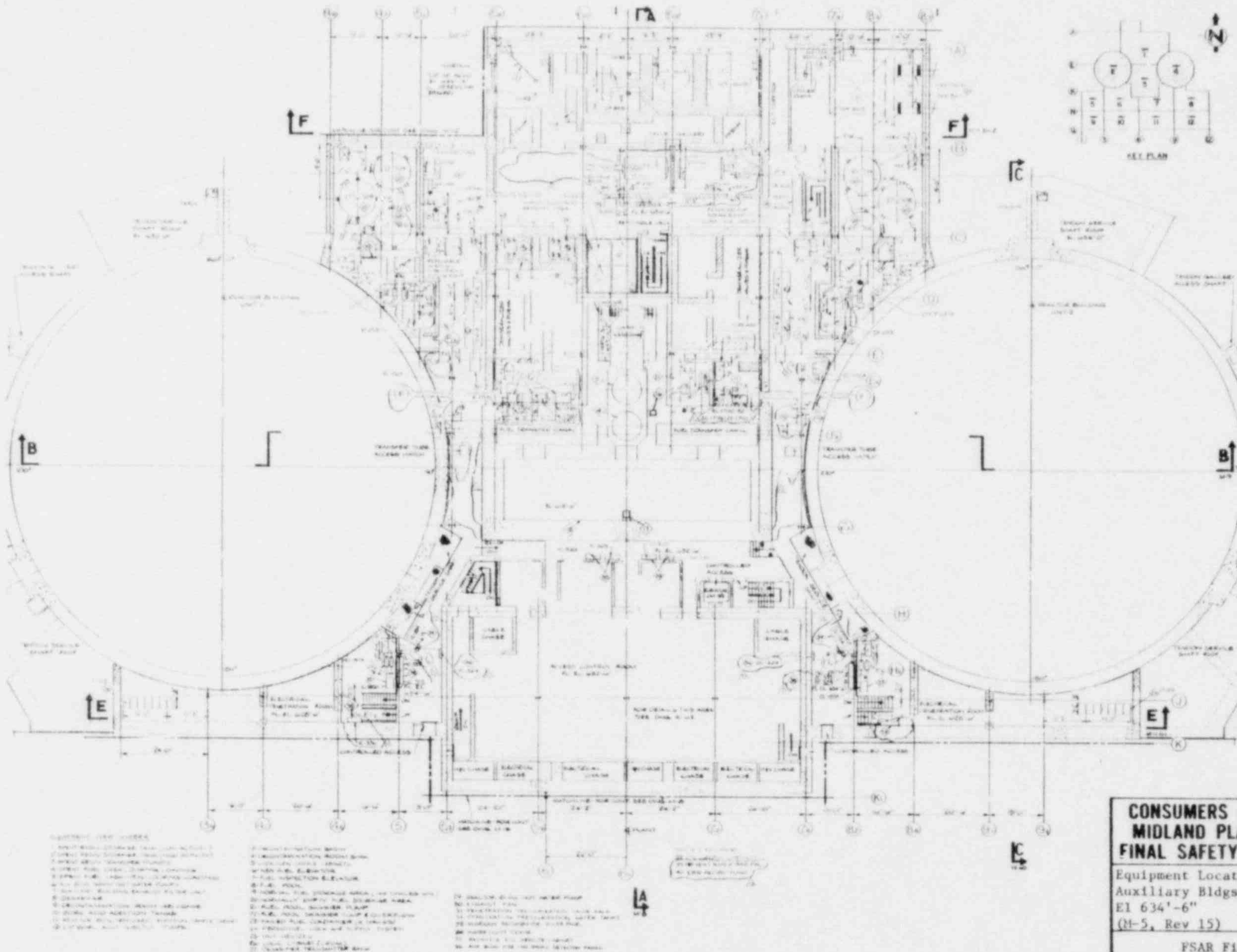
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Equipment Location - Reactor and Auxiliary Bldgs. Plan at El 614'-0"
(M-4, Rev 13)

FSAR Figure 1.2-5

9/81 Revision 37

ATTACHMENT 10



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

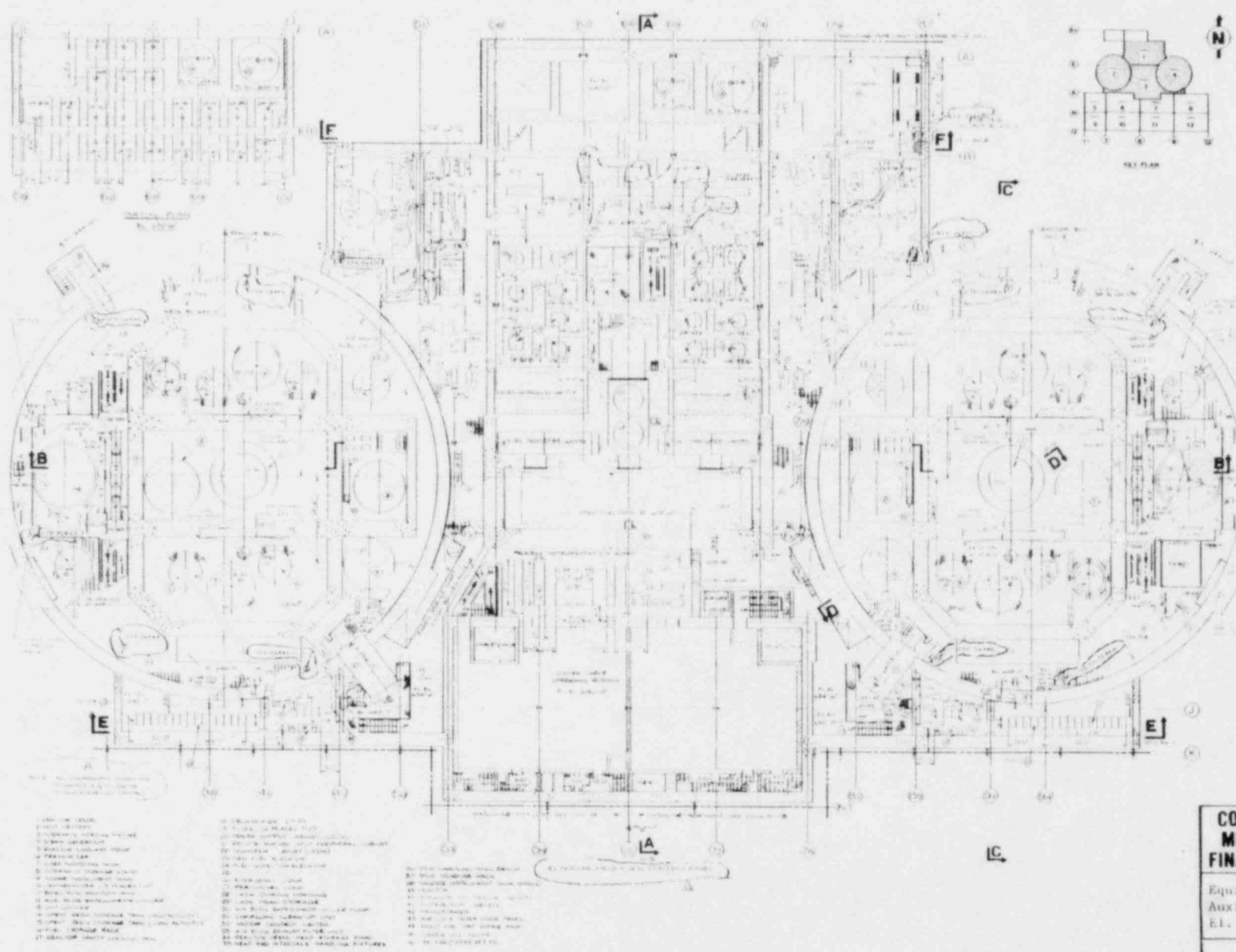
Equipment Location - Reactor and Auxiliary Bldgs. Plan at El 634'-6"
(M-5, Rev 15)

FSAR Figure 1.2-6

9/81

Revision 37

ATTACHMENT 11



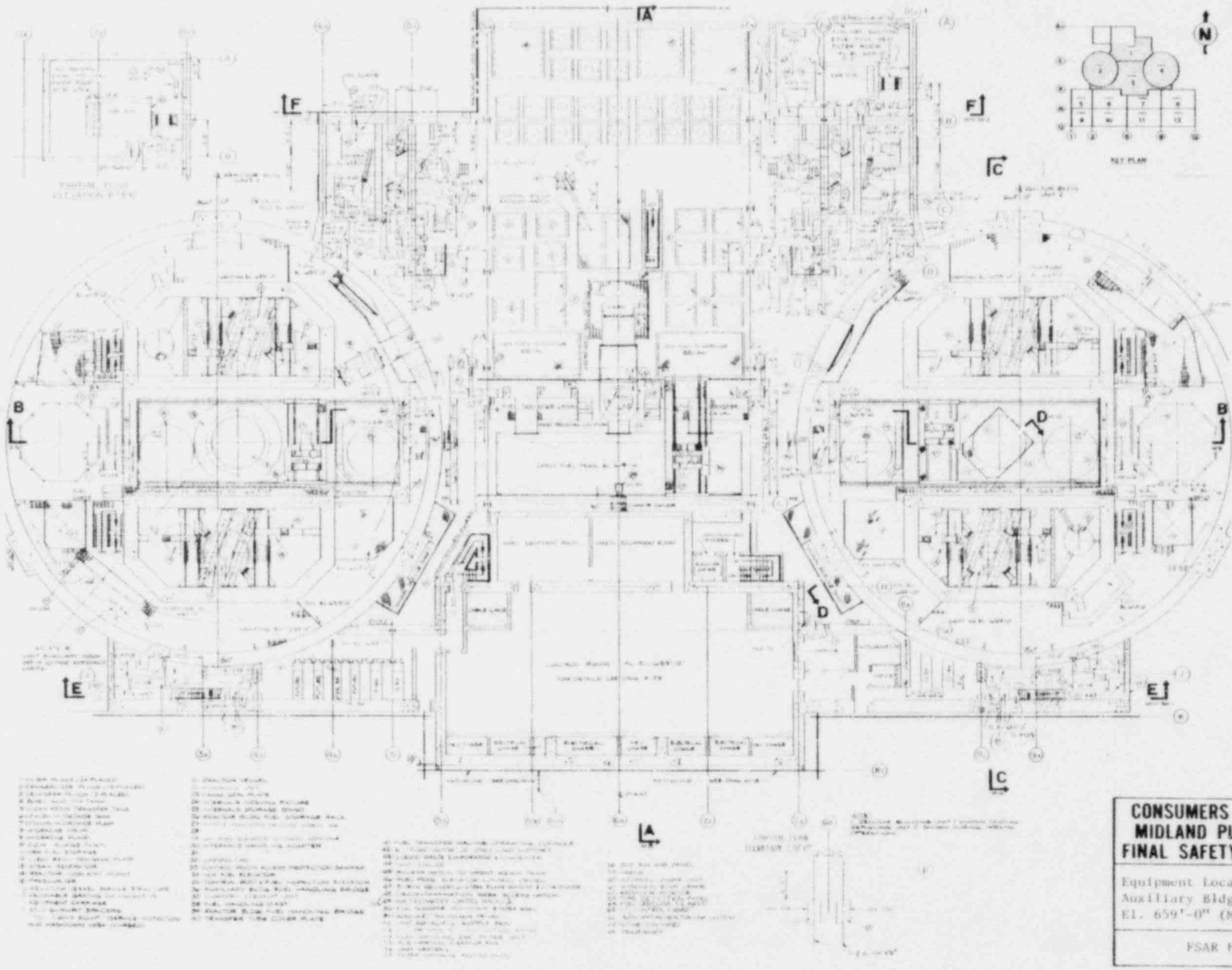
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Equipment Location - Reactor & Auxiliary Bldgs Plan at El. 645'-0" (M-6(Q), Rev 13)

FSAR Figure 1.2-7

4/81 Revision 33

ATTACHMENT 12



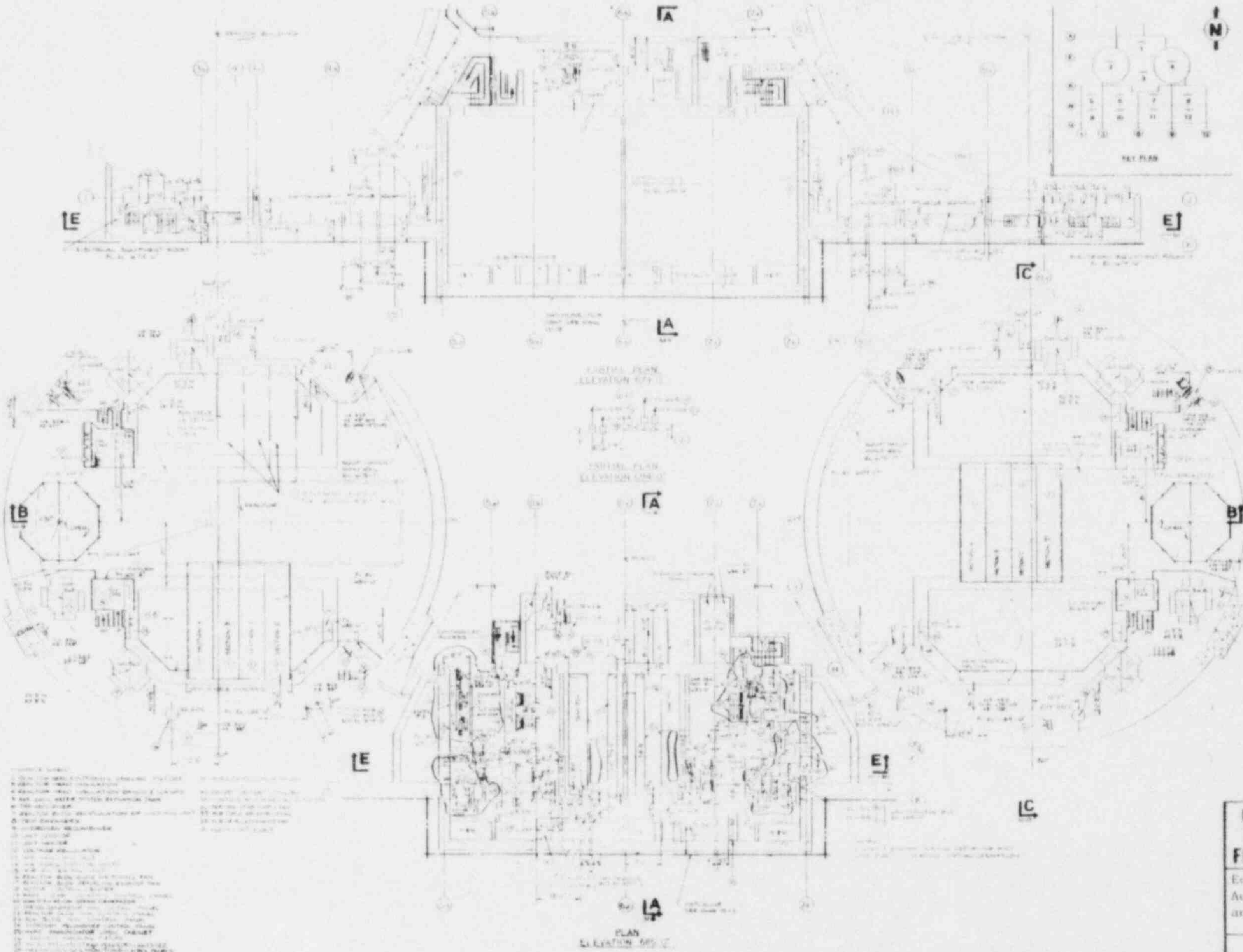
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Equipment Location - Reactor & Auxiliary Bldgs Plan at El. 659'-0" (M-7, Sh 1(Q), Rev 13)

FSAR Figure 1.2-8

4/81 Revision 33

ATTACHMENT 13



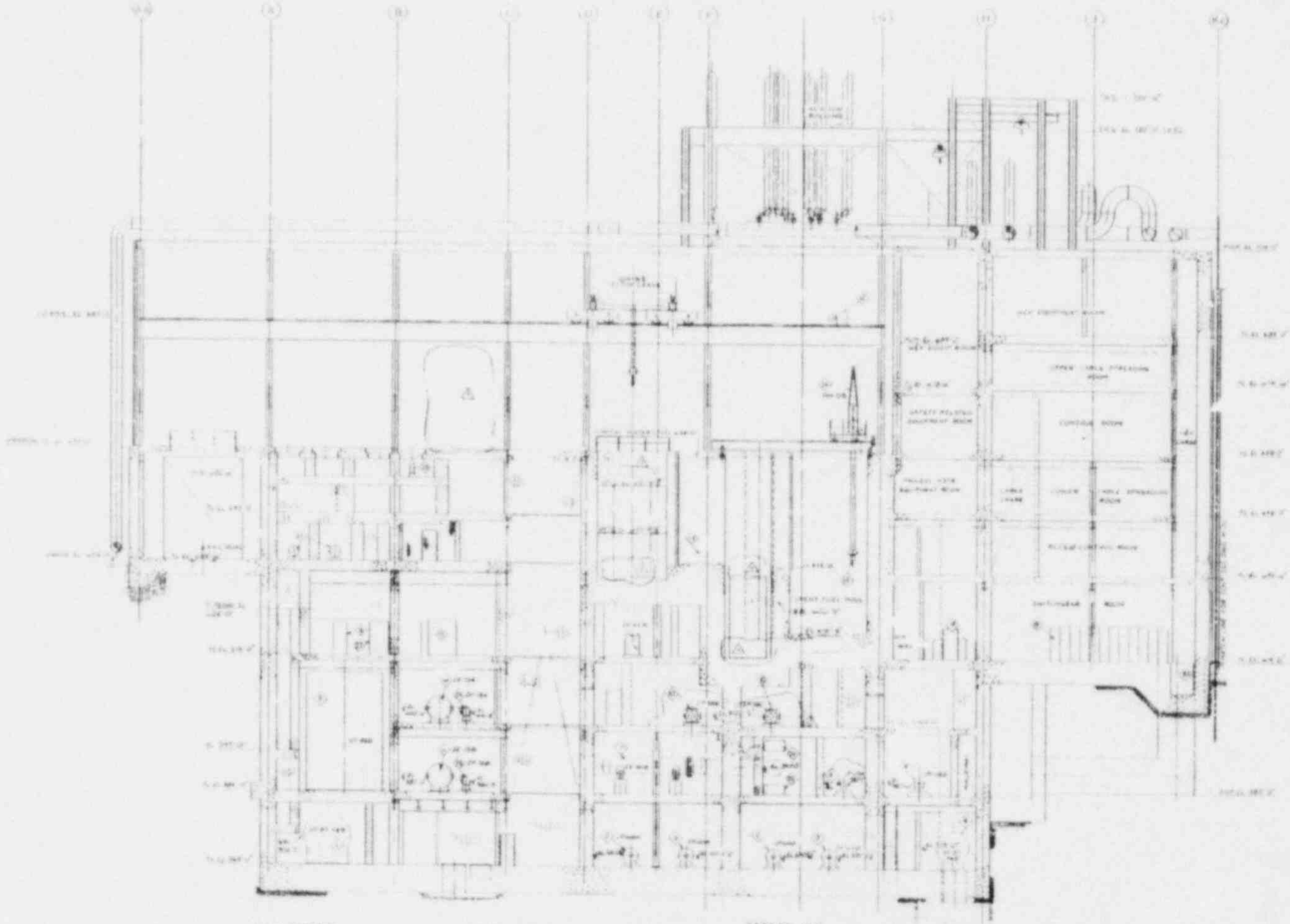
CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Equipment Location - Reactor & Auxiliary Bldgs Plan at El. 674'-0" and 685'-0" (M-7 Sh 2 (Q), Rev 12)

FSAR Figure 1.2-9

4/81 Revision 33

ATTACHMENT 14



SECTION A-A

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

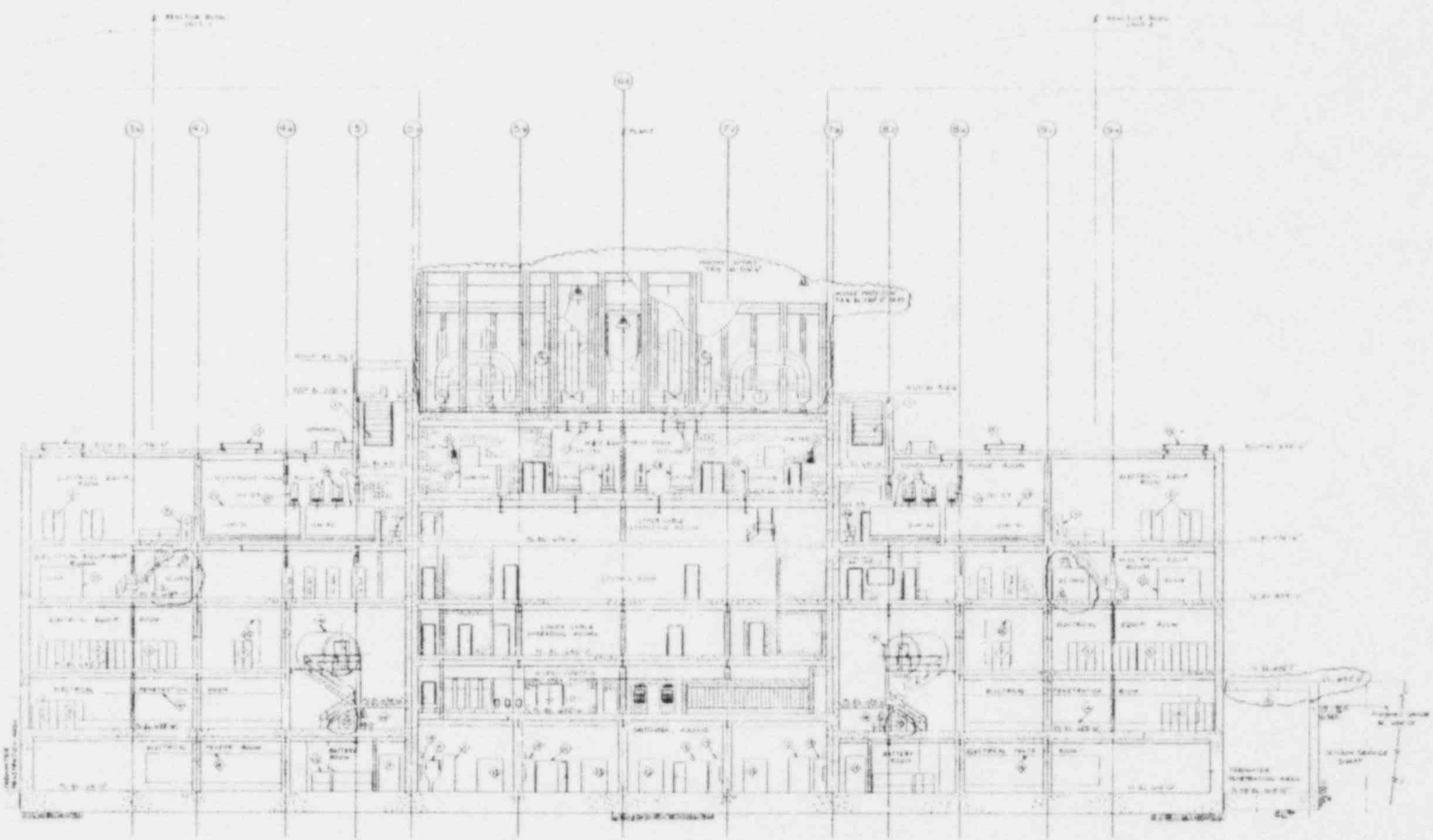
Equipment Location - Auxiliary Building Section A-A

(M-6, Rev. 7)

FSAR Figure 1.2-10

2/79 Revision 18

ATTACHMENT 15



SECTION "E-E"

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

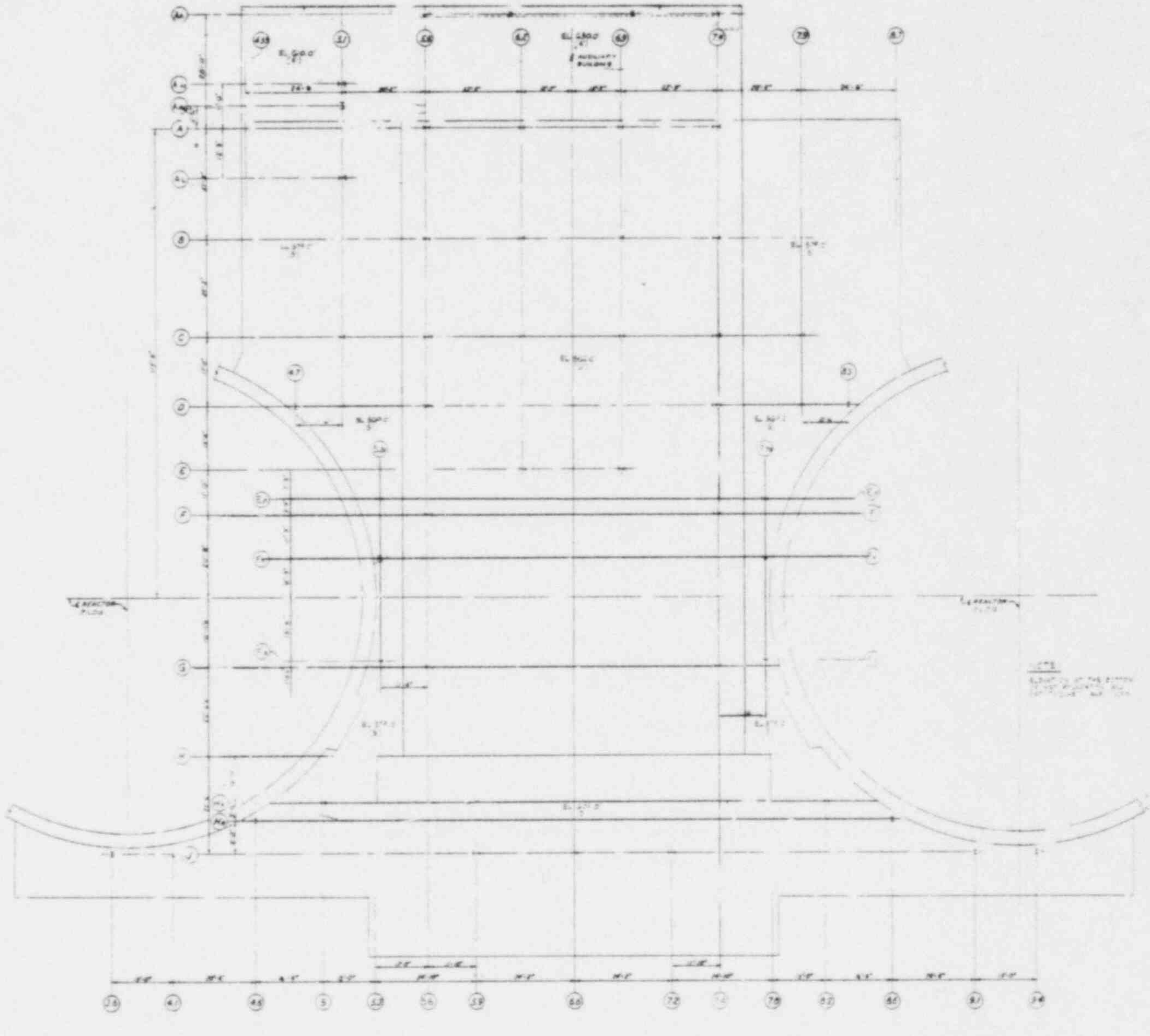
Equipment Location - Auxiliary Building Section E-E

(M-11 sh 1, Rev 6)

FSAR Figure 1.2-13

5/79 Revision 21

ATTACHMENT 16

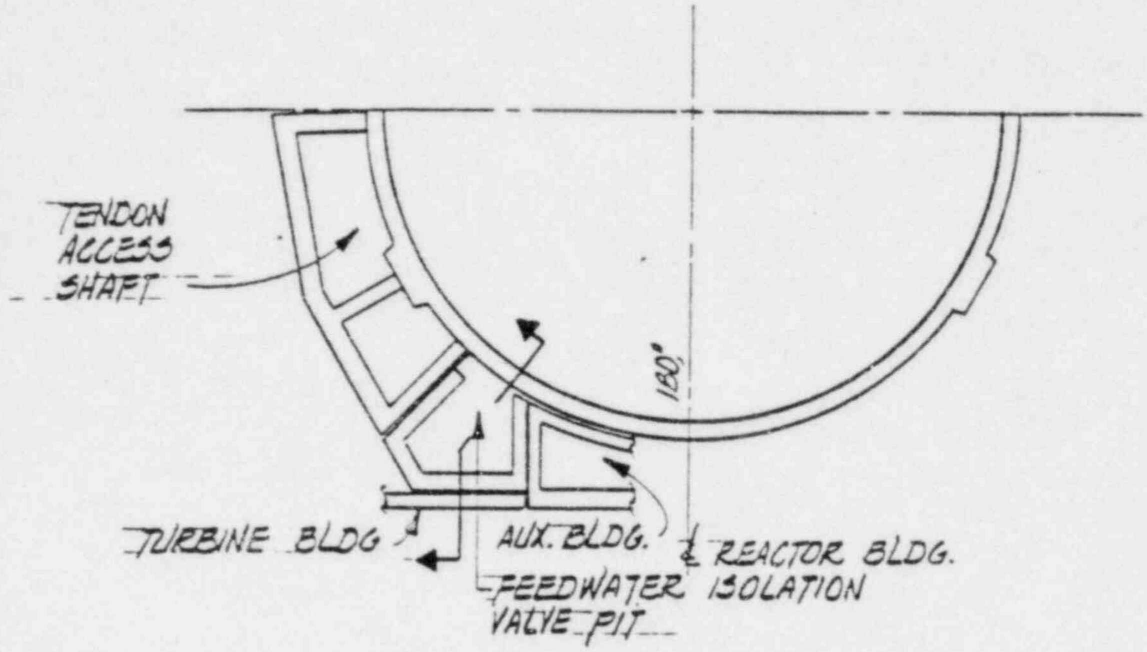


CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Auxiliary Building Foundation
Plans and Sections

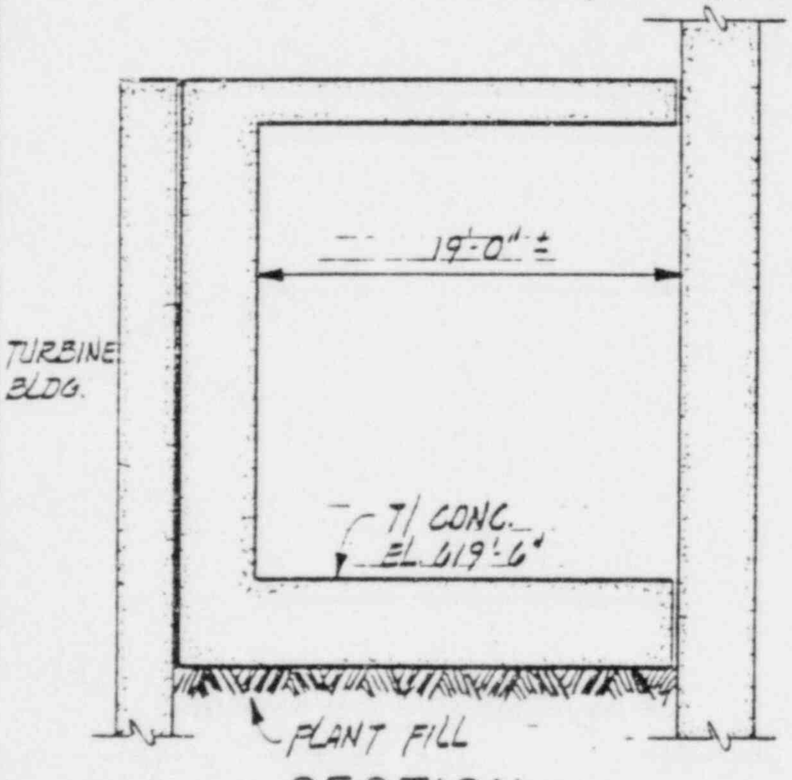
FSAR Figure 3.8-61

2/79 Revision 18



PLAN

UNIT 1 SHOWN
UNIT 2 OPPOSITE HAND

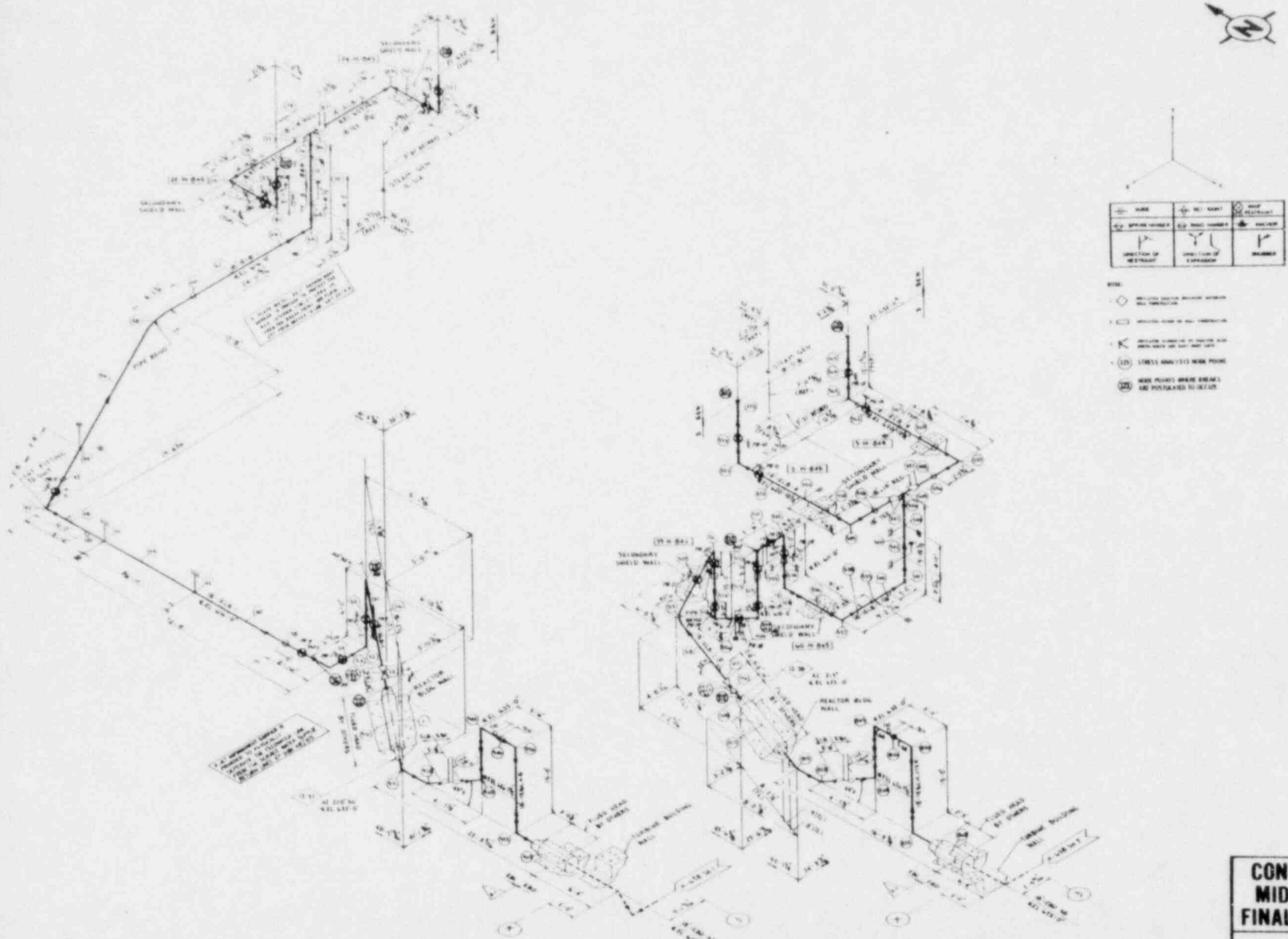


SECTION

MIDLAND PLANT UNITS 1 & 2
CONSUMERS POWER COMPANY

FEEDWATER ISOLATION
VALVE PIT
PLAN AND SECTION

FIGURE 13-12 DATE: 4/24/79

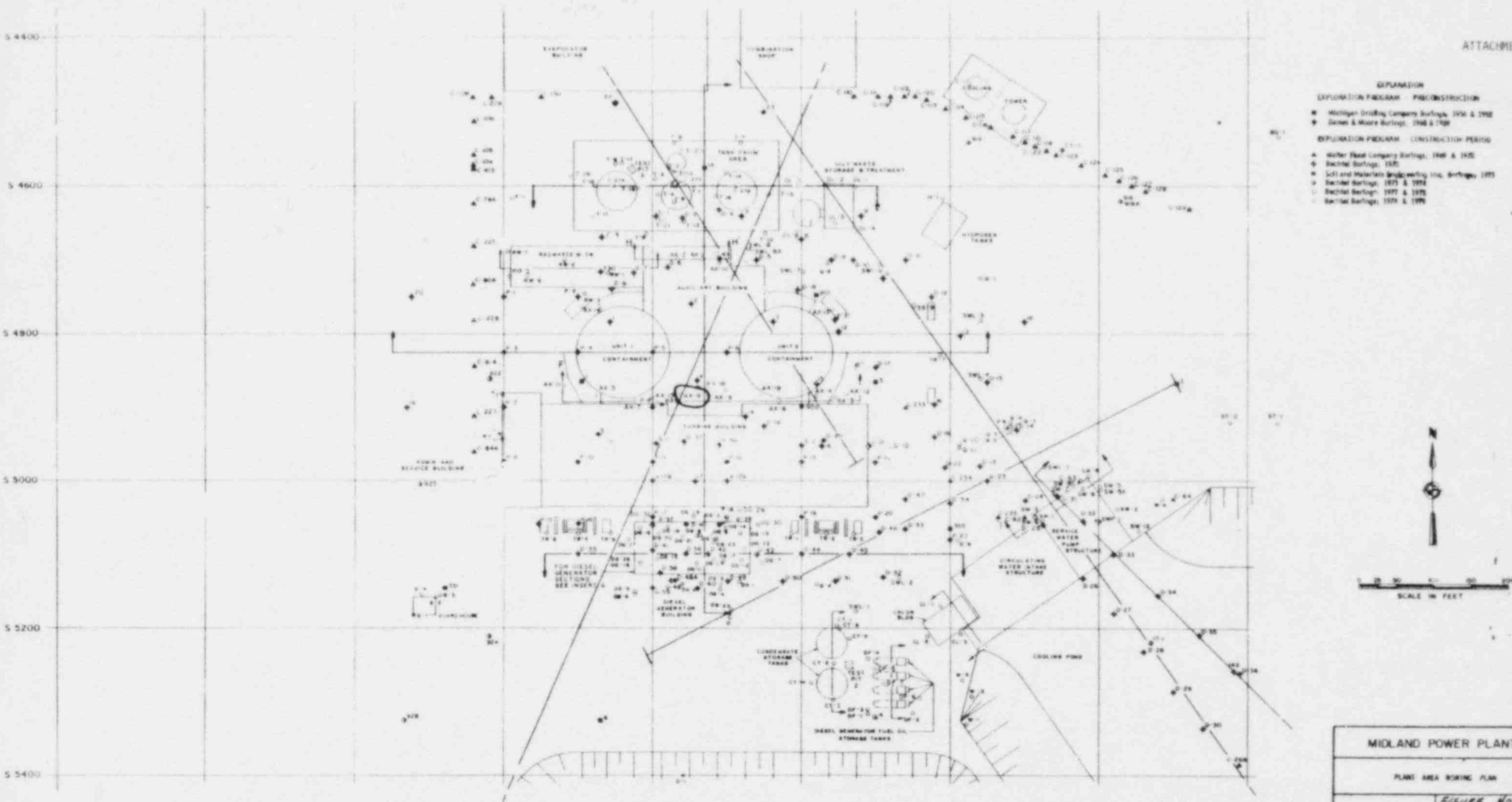


CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

Feedwater and Condensate System Unit 1 (Reactor Bldg. Main Feedwater Header)

FSAR Figure 3.6-12

ATTACHMENT 19



MIDLAND POWER PLANT	
PLANT AREA BORING PLAN	
FIGURE NO. 67	

ATTACHMENT 20

Consumers Power Company

James W Cook
Vice President - Projects, Engineering and Construction

General Offices: 1945 West Parnall Road, Jackson, MI 49201 • (517) 788-0453

September 30, 1981

RECEIVED OCT 22 1981 U.S. NUCLEAR REGULATORY COMMISSION

Harold R Denton, Director
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Washington, DC 20555

MIDLAND PROJECT
DOCKET NOS 50-329, 50-330
SUBMITTAL OF THE AUXILIARY BUILDING
DYNAMIC MODEL, SERVICE WATER PUMP
STRUCTURE DYNAMIC MODEL AND DESCRIPTION OF SOILS
SETTLEMENT REMEDIAL FIX FOR THE AUXILIARY BUILDING
FILE 0485.16, B3.0.1 SERIAL 14110
REFERENCES: (1) JWCook Letter to HRDenton, Serial 11625 Dated March 23, 1981
(2) JWCook Letter to HRDenton, Serial 13738 Dated August 26, 1981
ENCLOSURES: (1) Service Water Pump Structure Seismic Model
(2) Auxiliary Building Seismic Model
(3) Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits

In our previous correspondence of August 26, 1981 (Reference 2) construction permit level design information relating to the remedial actions for the service water pump structure was provided to the staff. Enclosed are twenty-five (25) copies of the report (Enclosure 1) entitled "Service Water Pump Structure Seismic Model" which is based upon the design information already forwarded to the NRC. In addition, we are providing twenty-five (25) copies each of two reports, Enclosures 2 and 3. Enclosure 2 describes the seismic model for the auxiliary building for computing the building response to seismic loading as well as to generate instructure response spectra. Enclosure 3 represents the construction permit level of design information for the auxiliary building remedial actions. All three of the enclosed documents are provided to complete commitments contained in the "Statement of Agreement" from the ASLB Prehearing Conference Order of May 5, 1981.

The seismic model reports for the service water pump structure and the auxiliary building include the following information: (1) model description, (2) soil-structure interaction considerations; (3) the dynamic model properties; and (4) fundamental frequencies and mode shapes. The auxiliary building model includes full underpinning of the control tower and electrical penetration areas, integrally tied to the main auxiliary building at Column Line H. The service water pump structure model includes full underpinning of the northern

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S 1/25

portion of the building originally supported by the fill. The models reflect the underpinning currently planned and, therefore, are subject to possible revision after the final building structural analysis and NRC staff review is completed. We believe that the enclosed reports combined with our scheduled meeting with the staff during the week of September 30, 1981 provides sufficient information to permit the NRC to review and provide its concurrence with the proposed remedial actions. Your expeditious review and approval would be most appreciated to support the hearings and construction of the remedial work.

GS Keeley for JW Cook

JWC/RLT/bh

CC Atomic Safety and Licensing Appeal Board, w/o
CBechhoefer, ASLB, w/o
MMCherry, Esq, w/o
RJCook, Midland Resident Inspector, w/o
RSDecker, ASLB, w/o
DHood, NRC, w/a (2)
DFJudd, B&W, w/o
JDKane, NRC, w/a
FJKelley, Esq, w/o
RBLandsman, NRC Region III, w/a
WHMarshall, w/o
WOtto, US Army Corps of Engineers, w/a
WDPaton, Esq, w/o
FRinaldi, NRC, w/a
HSingh, Army Corps of Engineers, w/a
BStamiris, w/o
FPCowan, w/o

BCC RCBauman/TRThiruvengadam, P-14-400, w/a
WRBird, P-14-418A, w/a
JEBrunner, M-1079, w/a
AJBoos, Bechtel, w/a
WJCloutier, P-24-611, w/a
BDhar, Bechtel, w/a
BFHenley, P-14-100, w/a
RWHuston, Washington, w/a
GSKeeley, P-14-113B, w/a
DBMiller, Midland, w/a
MIMiller, IL&B, w/a
KBRazdan, P-13-220, w/a
JARutgers, Bechtel, w/a
SLSobkowski, Bechtel, w/a
TJSullivan/DMBudzik, P-24-517, w/o
NRC Correspondence File, w/a

TECHNICAL REPORT ON UNDERPINNING
THE AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS
FOR
MIDLAND PLANT UNITS 1 AND 2
CONSUMERS POWER COMPANY
DOCKET NUMBERS 50-329 AND 50-330

MIDLAND PLANT UNITS 1 AND 2 TECHNICAL REPORT ON UNDERPINNING THE AUXILIARY BUILDING AND FEEDWATER ISOLATION VALVE PITS

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APPENDIXES

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FIGURES

MIDLAND PLANT UNITS 1 AND 2
TECHNICAL REPORT ON UNDERPINNING
THE AUXILIARY BUILDING AND
FEEDWATER ISOLATION VALVE PITS

1.0 INTRODUCTION

This report describes the design and construction requirements of the remedial action for the auxiliary building and feedwater isolation valve pits (FIVPs) necessitated by the settlement potential of the plant fill underlying the structure.

2.0 PRESENT CONDITION

The auxiliary building is located between the two containment buildings (Figure 1). The auxiliary building contains the control room, access control room, cable spreading rooms, engineered safeguard systems, switchgear equipment, and the facilities for fuel handling, storage, and shipment. The physical characteristics of the building are diverse because of its many functions. Exterior walls, the spent fuel pool, and some of the interior walls are constructed of reinforced concrete. The reinforced concrete floor and roof slabs are supported by walls, steel beams, and columns. (See FSAR Figures 3.8-51 through 3.8-54)

The FIVPs are symmetrically located at the sides of each containment building and are adjacent to the auxiliary building, electrical penetration area, turbine building, and the buttress access shaft. Each pit is C-shaped with the open end in contact with, but structurally separate from, the containment building. Primarily, the pits enclose the Seismic Category I feedwater pipe isolation valves.

Parts of the auxiliary building are founded on plant area fill: the railroad bay on the north side, the eletrical penetration areas for Units 1 and 2, and the control tower on the south side. The rest of the auxiliary building is founded on natural soil. The FIVPs for both Units 1 and 2 are founded on plant fill. Figures 1 and 2 show the layout of the auxiliary building foundation, including the areas of plant fill.

After discovering settlement of the plant fill under the diesel generator building, an investigation of the plant fill revealed some areas of inadequately compacted fill under the electrical penetration areas and FIVPs.

3.0 REMEDIAL ACTION

To adequately support the structure under all design load conditions, a continuous underpinning wall resting on undisturbed natural material is provided under the control tower and the electrical penetration area exterior walls. A similar wall is provided under the FIVPs. The underpinning wall provides the necessary vertical and horizontal support to the affected part of the structure. To ensure adequate load transfer, the existing structure is jacked against the underpinning walls (see Figures 3, 4, and 5).

4.0 DESIGN FEATURES

The details of the underpinning are provided in Figures 3, 4, and 5. The design features with appropriate dimensions are described as follows. The proposed underpinning for the Unit 1 and 2 penetration areas is a 6-foot thick, reinforced concrete wall that is 38 feet high and is belled out to 10 feet thick to limit bearing pressures to the allowable values. The underpinning walls under the control tower are 6 feet thick, 47 feet high, and are belled out to 14 feet thick. Similarly, the underpinning walls under the FIVPs are 4 feet thick, 38 feet high, and are belled out to 6 feet thick (see Figures 3, 4, and 5 for details). The walls are constructed to act as a continuous member under the perimeter of the structures. The entire wall system will be founded on undisturbed natural material. The allowable bearing pressures for the undisturbed natural material are based on safety factors of 2 for dynamic loading and 3 for static loading.

Design jacking force is applied to the existing structure to provide adequate load transfer from the structure to the underpinning. The jacking force is determined so the structure is not unduly stressed under dead load and live load conditions. These jacking forces are transmitted from the structure through the permanent underpinning wall to the bearing stratum. Preliminary details of jacking forces are shown in Figures 3, 4, and 5.

Dowels connect the underpinning walls and the existing structure at the vertical and horizontal interfaces. The dowels are designed to transfer shear and tension forces between the structure and the underpinning wall.

In addition to the conventional lap splices, Fox Howlett mechanical, tapered thread splices may be used for reinforcing the underpinning walls. These tapered thread splices shall conform to the requirements of Section III, Division 2 of the American Society of Mechanical Engineers Boiler and Pressure Vessel Code, 1980 Edition, 1980 and 1981 Summer Addenda.

5.0 CONSTRUCTION

This section addresses construction groundwater control, building post-tensioning, temporary supports, construction details, instrumentation, load transfer, load sensing, and corrective measures.

5.1 CONSTRUCTION GROUNDWATER CONTROL

To construct the underpinning, the FIVPs and electrical penetration and control tower areas are dewatered.

The construction groundwater control consists of three parts:

a. Permanent wells to maintain the groundwater at el 595

b. A frozen earth cutoff membrane to prevent recharge to the underpinning area

c. Supplemental internal predrainage wells to remove the residual water within the cutoff wall

The soil particle monitoring acceptance criteria for the permanent wells have been accepted by the NRC staff. The acceptance criteria for the supplemental internal predrainage wells is that the effluent has less than 10 ppm of soil particles larger than 0.05 millimeters. For further details regarding construction groundwater control, refer to Appendix B.

5.2 BUILDING POST-TENSIONING

Construction site dewatering removes the buyoyancy force under the FIVPs and electrical penetration areas. To compensate for this effect, an existing temporary post-tensioning system applies a compressive force to the upper part of the east-west walls of the electrical penetration areas as shown in Figure 7. The post-tensioning system will be removed after initial jacking loads are applied under the penetration areas.

5.3 TEMPORARY SUPPORTS

Presently, the FIVPs are temporarily supported as shown in Figure 8 by rock bolts and tension rods from the steel grillage beams spanning the buttress access shaft and the turbine building walls. These temporary supports will be removed after the valve pits are underpinned.

5.4 CONSTRUCTION DETAILS

Refer to pendix C for construction details.

5.5 INSTRUMENTATION, LOAD TRANSFER, LOAD SENSING, AND CORRECTIVE MEASURES

Instrumentation is provided to detect building and pier movements during construction, permanent load transfer, and corrective measures. The details are given in Appendix D.

5.6 ACCEPTANCE CRITERIA FOR BEARING STRATUM

The quality of the clay till as bearing material will be initially assessed by using the WES penetrometer (Type CN 973). This device consists of a hand-held, cone penetrometer with a shaft whose area is equal to precisely 1/2 sq in. and whose bottom end is tapered with a 30° cone. A load is applied through a proving ring at the top with a dial gage which reads as pressure intensity in the full cross-section of the shaft. That is, the dial gage reading gives the pressure applied through the shaft to the soil in psi, which is exactly double the force applied through the proving ring. The maximum foce that can be placed through the proving ring is 150 pounds total or 300 psi in the shaft cross-section.

The correlations presented in this appendix relate pressure read by the proving ring in psi and penetration of the pointed cone with bearing capacity for a strip footing. The failure bearing capacity factors utilized are nine times shear strength for the cone, 5.7 for the strip footing, a safety factor of 3, and assume a $\phi = 0$ soil.

The cone will be painted in 1/4-inch wide bands with high visibility colors. Several readings will be taken of the force required to make penetrations of the cone to various depths in the soil. From this, using the calibration curves (see Figure 9), the bearing quality of the subgrade can be evaluated. This helps the resident geotechnical engineer judge by his visual inspection when the underpinning excavation has reached clay till of the bearing quality described in Subsection 7.2.1.

More decisive information is expected to be obtained from the application of jacking loads to the individual piers for the temporary support of the two penetration structures. Settlement of these piers will be measured by a dial gage as the jacking load is applied in stages. From the observed load and settlement values, an equivalent soil modulus will be computed utilizing the conventional relationships in elastic theory. This modulus value will then be applied to a confirming analysis of the permanent underpinning wall to ensure that the combination of final bearing pressure bearing area and embedment will limit settlement to tolerable values for the structure.

6.0 <u>MONITORING REQUIREMENTS</u>

During construction, the underpinning of the existing structure is monitored for settlement and crack propogation, and the existing structure is monitored for differential settlement. The long-term surveillance program of the building after the underpinning is constructed is being evaluated.

6.1 SETTLEMENTS

6.1.1 Monitoring Structures

The elevations of settlement markers attached to the structure are measured and recorded according to a schedule based on construction procedures. Based on these readings, the absolute and differential settlement values are calculated. The acceptance criteria for the absolute settlement values are established based on the requirem .s of various interconnected piping systems and geotechnical considerations. The acceptance criteria for the differential settlement are based on the structure's ability to withstand the loading due to differential settlement in combination with stresses from other loads. The acceptance criteria for the differential and absolute settlement and the frequency of measurement will be established before the start of the underpinning work.

6.1.2 Monitoring Underpinning

During jacking operations, the underpinning will be monitored for movements. The procedure for this monitoring is being developed. An acceptance criteria for this movement will be established before beginning underpinning, based on the expected loads due to jacking and geotechnical properties of the foundation material.

6.2 CRACKS

Existing or new cracks appearing during the underpinning construction will be monitored. Because of the sequence of construction procedures, it is not anticipated that existing cracks will significantly widen or that significant new cracks will appear. However, any new structural cracks or changes in existing structural cracks exceeding 0.01 inch will be evaluated and if any crack widths reach 0.03 inch, construction in the affected area will be modified or suspended until the reasons for excessive cracking are established and appropriate remedial measures are implemented.

6.3 CONSTRUCTION GROUNDWATER

Groundwater measurements will be taken at the existing observation wells and/or piezometers located outside and within the influence of the freeze wall. Measurements will be taken at least daily after the freeze wall is installed. Instrumentation to monitor ground temperature and the rate of frost penetration will also be installed (such as thermistors; multi-sensor, copper, constantan thermocouples, flouroscene frost penetration markers, etc). These systems will be monitored closely during freezing of the elements and on a routine basis during the underpinning. Each freeze unit and coolant pipe distribution system will be visually inspected periodically during underpinning operations.

7.0 ANALYSIS AND DESIGN

The auxiliary building and FIVPs were originally designed in accordance with Final Safety Analysis Report (FSAR) requirements for Seismic Category I structures. A preliminary analysis of the underpinned structure is in progress. The seismic loads used in this analysis were obtained from a preliminary seismic analysis of the auxiliary building. These loads incorporate the effects of the underpinning.

7.1 DESCRIPTION OF ANALYTICAL MODELS

A description of the mathematical seismic model is being submitted separately to the NRC. A description of the static, three-dimensional, finite element model of the structure is given in Appendix A.

7.2 SOIL PARAMETERS

7.2.1 Bearing Capacity

The present approach to selecting allowable bearing pressures on the various underpinning elements is based on FSAR shear strength data as modified by the recent exploration and testing by Woodward-Clyde and Consultants (WCC). Strength test data (Reference 1) are summarized in the Consumers Power Company letter (Reference 2) of September 22, 1981, to the NRC.

The allowable bearing capacity commitment in FSAR Subsection 2.5.4.10.1 for the foundation design requires a safety factor of 3 against dead load plus sustained live load and a safety factor of 2 for the above loads plus the seismic load. For the strip footing, which represents the mode of load application for the continuous underpinning piers, the bearing capacity factor (Nc) in $\emptyset = 0$ foundation soil ranges between 6 and 7 depending on the depth of embedment in the natural soils. Choosing a bearing capacity factor (Nc) of 6 and the average shear strength (Su) of 6.6 ksf for a typical condition where embedment equals half the width of the underpinning piers, an ultimate bearing capacity can be calculated. This ultimate bearing capacity equals Nc x Su, which is approximately 40 ksf. The allowable bearing pressure under sustained loads thus equals 40/3 or 13.3 ksf. The ultimate bearing capacity under sustained loads using an undrained shear strength of 6 ksf from FSAR Figure 2.5-33 is 36 ksf resulting in an allowable bearing capacity of 36/3 or 12 ksf. Similarly, the corresponding allowable bearing capacity values for the seismic condition are 20 ksf and 18 ksf using the average shear strength based on WCC tests and the design values presented in FSAR Figure 2.5-33. It is evident from the foregoing discussion that the allowable bearing capacity based on the average shear strength obtained from the tests are higher than the conservative values based on FSAR shear strength data.

Support of both temporary and permanent underpinning elements will be in undisturbed, unweathered, clay till whose suitability will be judged by the procedures outlined in Section 5.6. The bearing characteristics of the underpinning units can be summarized as follows based on currently available loading information.

Unit	Applied Bearing Pressure (ksf)	Safety Factor	Approximate Elevation of Bearing
Temporary piers for penetration building	20	2.0	562 to 568
Permanent wall for penetration building	6.8	5 to 6	571
Permanent wall for control tower	8.8	4 to 4-1/2	562

Note: Bearing pressure is for dead load, sustained live load, and net weight of the underpinning elements. In case of dead, live, and seismic loads, the allowable bearing capacity will be 20 ksf, giving a safety factor of 2 based on the recent WCC investigation.

Distinctly conservative bearing pressures have been chosen for the permanent underpinning design because of the necessity of providing essentially unyielding support after the lock-off of the jacking load. The test jacking of the individual piers for temporary underpinning is expected to demonstrate the suitability of the load condition of the permanent units. The higher bearing pressures applied to the temporary piers will be judged satisfactory if the settlement rate reaches a straight line trend on a semi-log time plot approximately 2 days after the jacking load reaches a peak value, and the final rate of settlement does not exceed the criteria given in Appendix D.

7.2.2 Soil Deformation Modulus

Undrained E values were studied by WCC, employing controlled rebound-reload cycles in the undrained triaxial tests. Four such tests yielded a median value of secant modulus of elasticity of about 3,500 ksf, roughly 500 times the median undrained shear strength. This corresponds closely to the equivalent undrained E value interpreted from the Dames and Moore tests between el 560 to 580. The E value which can be computed from the actual observed settlement in the latter stages of the reactor construction are somewhat larger than the laboratory test values and range from about 5,000 to 6,000 ksf.

This is expected because the large size of the reactor foundation stresses the clay till to a great depth, generally in excess of 100 feet below the reactor bearing level. This engages till with high residual, horizontal stress whose effective E value probably increases with depth in the same manner.

For the underpinning units whose narrow dimension in plan is only 10 feet, the seat of settlement is much shallower than that of the reactor structures. Consequently, the E value selected should be closer to the laboratory test values than to the interpreted E value for the reactor. We have conservatively selected an E value of 3,000 ksf for the analysis presented in Subsection 7.2.2 to compute the anticipated settlement of the underpinning units. The E value used in the dynamic analysis is presented in Auxiliary Building Seismic Model (Reference 4).

7.2.3 Settlement of Underpinning Units

Employing an E value of 3,000 ksf, preliminary estimates of settlement were made for each of the three underpinning units described in Subsection 7.2.1. The methods of analysis are based on simple theory of elasticity and summarized in NAVFAC Design Manual DM-7 (Reference 3), which include the influence of size and shape of the loaded area and depth of embedment of the piers or walls within the clay till bearing stratum. The settlement values tabulated below include time-delayed movements, which occur slowly in the form of secondary compression. These latter values are expected to be about one-fifth of the total computed settlement.

		Preliminary Estimated Settlement		
Unit	Applied Bearing Pressure (ksf)	Total (in.)	During Jacking (in.)	After Jacking (Long Term) (in.)
Temporary piers for penetration building	20	1.0	0.8	0.2 (1 yr ±)
Load transfer points for temporary load to reactor footing	2,300 k and 3,000 k	0.3	0.2	0.1 (1 yr ±)

Unit	Applied Bearing Pressure (ksf)	Preliminary Estimated Settlement: Total (in.)	During Jacking (in.)	After Jacking (Long Term) (in.)
Permanent wall for penetration building	6.8	0.6	0.4	0.2 (40 yr)
Permanent wall for control tower	8.8	0.9	0.6	0.3 (40 yr)

In general, it is anticipated that the jacking load for permanent underpinning would be applied in stages over several days and the peak value maintained for 90 days after reaching the peak. Satisfactory completion of the jacking is expected to be indicated by a final straight line trend on a semi-log plot of settlement versus time. We have conservatively selected an E value of 3,000 ksf for the analysis presented in Subsection 7.2.2 to compute the anticipated settlement of the underpinning. The settlement increment in the last 30 days of sustained load should not exceed 0.05 inch, and not more than 0.01 inch in the final 10 days of the jacking period.

7.2.4 Differential Settlement Between Existing Walls and Underpinning

Of the estimated settlements listed in Subsection 7.2.2, only those occurring after lock-off of the jacking loads will be involved in creating differential settlements within the auxiliary building structural frame. The purpose of the final jacking operation is to ensure that all but the very gradually occurring time-delayed settlement is completed while the structure is maintained in an undeflected position. Thus, the settlement of particular interest is the long-term value for the south underpinning wall of the control tower, which is estimated as somewhat less than 1/4 inch. Differential settlement between the control tower area and the penetration areas will be less than 1/4 inch. This will be compared to the predicted settlement of the nonunderpinned block of the auxiliary building. The respective values will be employed in the analysis of the effects of the predicted settlement pattern.

7.3 DESIGN CRITERIA AND APPLICABLE CODES

The underpinning structure is designed as a Seismic Category I structure in accordance with FSAR Section 3.8.

7.4 CONSTRUCTION CONDITION

7.4.1 Structural Behavior

During construction, the control tower and the penetration areas will be supported by a combination of jacking loads and the existing material. As construction proceeds, the soil support will be replaced by the jacking loads. This will be simulated by appropriate boundary conditions in the finite element model.

7.4.2 Boundary Conditions

For boundary conditions, refer to Appendix A.

7.4.3 Loads and Load Combinations

Dead load, live load, external hydropressures, soil pressures, wind loads, and jacking loads will be investigated. The structure will also be checked for sliding and overturning.

7.4.4 Acceptance Criteria

The acceptance criteria for the temporary condition will be in accordance with American Concrete Institute (ACI) 318-71 except that a stress increase factor of 1.33 for the structure will be used.

7.5 PERMANENT CONDITION

7.5.1 Structure Behavior

The vertical loads of the control tower, electrical penetration areas, and FIVPs are transmitted to the foundation medium through the underpinning wall bearing area. The lateral forces due to seismic and tornado loads in the east-west direction for the control tower and penetration areas will be transferred to the foundation medium by the combined action of the east-west walls of the control tower and penetration areas and the underpinning walls underneath. The lateral forces in the north-south direction will be transferred through the underpinning to the north-south walls in the main auxiliary building. To ensure this action, the underpinning walls are connected to the existing main auxiliary building structure by dowels capable of transferring all direct loads as described in Section 4.0.

The lateral forces for the FIVPs will be carried by the box-shaped underpinning walls.

7.5.2 Boundary Conditions

For the boundary conditions of the finite element model used in the analysis, refer to Appendix A.

7.5.3 Loads and Load Combinations

The underpinning structure rests entirely on undisturbed natural material. The preliminary analysis of the underpinned structure utilizes the same load combinations used in the original design. However, each load combination is modified by adding the jacking load (P_L). For each load combination, the jacking load is evaluated with two load factors: a value of 1.0, and the load factor associated with the dead load for that load combination.

For the design of the underpinning and the connections to the existing structure, the safe shutdown earthquake (SSE) (0.12g) forces were increased by 50% to provide for a possible future increase in this load. The 50% increase was applied to the seismic response of the structure corresponding to the analytical model with the mean soil properties. The existing structure was checked for a 0.12g SSE.

The long-term settlement of the underpinning wall after it is connected to the existing structure will be calculated. The calculation is based on properties of the supporting soil. The long-term settlement effects will be considered in the final analysis of the structure. To provide for these effects, the final analysis is governed by four additional load combinations. These load combinations are discussed in the response to Question 15 of the NRC Requests Regarding Plant Fill (September 1979) and were used in the diesel generator building reanalysis. The load combinations are modified by the additional jacking load.

Table 1 lists 26 load combinations that have been modified for jacking loads. For the preliminary analysis of the underpinned auxiliary building and FIVPs, the following load combination is most critical:

$$U = 1.0D + 1.0L + 1.0E' + 1.0T_0 + 1.25H_0 + 1.0R + P_L$$

where

U = ultimate load capacity

D = dead loads

L = live loads

E' = safe shutdown earthquake

T_0 = thermal effects during normal operating conditions

H_0 = force on structure due to thermal expansion of pipes under operating conditions

R = local force or pressure on structure or penetration cause by rupture of any one pipe

P_L = load on structure due to jacking preload

In addition to this load combination, the underpinned structure was checked for stability using the load combinations specified in FSAR Subsection 3.8.6.3.4.

A complete analysis of the underpinned structure, using all applicable load combinations, will be made when the final seismic loads become available.

7.5.4 Structural Acceptance Criteria

The acceptance criteria for analyzing the underpinned structure are in accordance with FSAR Subsection 3.8.6.5. For factors of safety against sliding and overturning, refer to FSAR Table 3.8-23.

7.5.5 Additional NRC Requirements

For information purposes, an analysis of the critical sections of the underpinned structure will be made conforming to the provisions of ACI 349-76 as supplemented by NRC Regulatory Guide 1.142.

8.0 QUALITY ASSURANCE REQUIREMENT

This project work is a combination of Q-listed and non-Q-listed work. Construction of permanent structures, such as the underpinning wall and the connectors, is Q-listed, as well as any other activity or structure necessary to protect the auxiliary building and FIVPs. Construction of temporary structures such as the access shafts and tunnels is non-Q-listed. A detailed quality plan shall be prepared by Bechtel to identify those specific activities which are required to have a safety "Q" quality program applied with the major quality program elements for these activities.

REFERENCES

1. Woodward-Clyde and Consultants Test Reports

2. Consumers Power Company letter to the NRC, September 2[illegible], 1981

3. NAVFAC Design Manual DM-7

4. Bechtel Power Corporation, Auxiliary Building Seismic Model, Rev 3, September 28, 1981

TABLE 1

LOAD EQUATIONS FOR THE AUXILIARY BUILDING
MODIFIED TO INCLUDE PRELOAD

Responses to NRC Requests Regarding Plant Fill, Question 15

(See Note 2)

a. Normal Operating Condition:

$$U = 1.05D + 1.28L + 1.05T + P_L \quad (1)$$

$$U = 1.4D + 1.4T + P_L \quad (2)$$

b. Severe Environmental Condition:

$$U = 1.0D + 1.0L + 1.0W + 1.0T + P_L \quad (3)$$

$$U = 1.0D + 1.0L + 1.0E + 1.0T + P_L \quad (4)$$

Loading Under Normal Conditions

a. Concrete:

$$U = 1.4D + 1.7L + P_L \quad (5)$$

$$U = 1.25\ (D + L + H_O + E) + 1.0T_O + P_L \quad (6)$$

$$U = 1.25\ (D + L + H_O + W) + 1.0T_O + P_L \quad (7)$$

$$U = 0.9D + 1.25\ (H_O + E) + 1.0T_O + P_L \quad (8)$$

$$U = 0.9D + 1.25\ (H_O + W) + 1.0T_O + P_L \quad (9)$$

For ductile moment resisting concrete frames and for shear walls

$$U = 1.4\ (D + L + E) + 1.0T_O + 1.25H_O + P_L \quad (10)$$

$$U = 0.9D + 1.25E + 1.0T_O + 1.25H_O + P_L \quad (11)$$

Structural Elements Carrying Mainly Earthquake Forces, Such as Equipment Supports

$$U = 1.0D + 1.0L + 1.8E + 1.0T_O + 1.25H_O + P_L \quad (12)$$

TABLE 1 (continued)

b. Structural Steel:

$D + L + D_L$ (stress limit = f_s) (13)

$D + L + T_O + H_O + E + P_L$ (stress limit = $1.25f_s$) (14)

$D + L + T_O + H_O + W + P_L$ (stress limit = $1.33f_s$) (15)

In addition, for structural elements carrying mainly earthquake forces, such as struts and bracing:

$D + L + T_O + H_O + E + P_L$ (stress limit = f_s) (16)

Loading Under Accident Conditions

a. Concrete:

$U = 1.05D + 1.05L + 1.25E + 1.0T_A + 1.0H_A + 1.0R + 1.05P_L$ (17)

$U = 0.95D + 1.25E + 1.0T_A + 1.0H_A + 1.0R + 0.95P_L$ (18)

$U = 1.0D + 1.0L + 1.0E' + 1.0T_O + 1.25H_O + 1.0R + P_L$ (19)

$U = 1.0D + 1.0L + 1.0E' + 1.0T_O + 1.0H_O + 1.0R + P_L$ (20)

$U = 1.0D + 1.0L + 1.0B + 1.0T_O + 1.25H_O + P_L$ (21)

$U = 1.0D + 1.0L + 1.0T_O + 1.25H_O + 1.0W' + P_L$ (22)

b. Structural Steel:

$D + R + T + H_O + E' + P_L$ (stress limit = $1.5f_s$) (23)

$D + L + R + T_A + H_A + E' + P_L$ (stress limit = $1.5f_s$) (24)

$D + L + B + T_O + H_O + P_L$ (stress limit - $1.5f_s$) (25)

$D + L + T_O + H_O + W' + P_L$ (stress limit = $1.5f_s$) (26)

TABLE 1 (continued)

where

U = required strength to resist design loads or their related internal moments and forces

For the ultimate load capacity of a concrete section, U is calculated in accordance with American Concrete Institute (ACI) 318-71.

F = specified minimum yield strength for structural steel

f_s = allowable stress for structural steel; f_s is calculated in accordance with the American Institute of Steel Construction (AISC) Code, 1963 Edition for design calculations initiated prior to February 1, 1973.

f_s is calculated in accordance with the AISC, 1969 Edition, with Supplements 1, 2, and 3 for design calculations initiated after February 1, 1973.

D = dead loads

L = live loads

P_L = load on structure due to jacking preload

R = local force or pressure on structure or penetration caused by rupture of any one pipe

T = effects of differential settlements, creep, shrinkage, and temperature

T_0 = thermal effects during normal operating conditions

H_0 = force on structure due to thermal expansion of pipes under operating conditions

T_A = total thermal effects which may occur during a design accident other than H

H_A = force on structure due to thermal expansion of pipes under accident condition

E = operating basis earthquake (OBE)

E' = safe shutdown earthquake load (SSE)

B = hydrostatic forces due to the postulated maximum flood (PMF) elevation of 635.5 feet

W = design wind load

TABLE 1 (continued)

W' = tornado wind loads, including missile effects and differential pressure

Ø = capacity reduction factor

The capacity redunction factor (Ø) provides for the possibility that small adverse variations in material strengths, workmanship, dimensions, control, and degree of supervision, although individually within required tolerances and the limits of good practice, occasionally may combine to result in undercapacity.

NOTES:

1. In the load equations, the following factors are used:

   Ø = 0.90 for reinforced concrete in flexure

   Ø = 0.75 for spirally reinforced concrete compression members

   Ø = 0.70 for tied compression members

   Ø = 0.90 for fabricated structural steel

   Ø = 0.90 for reinforced steel in direct tension

   Ø = 0.90 for welded or mechanical splices of reinforcing steel

2. Unity load factor is shown for P_L. An alternative load factor to be considered in all load combinations is the load factor associated with dead load (D) in that loading combination.

For load combinations 23-26:

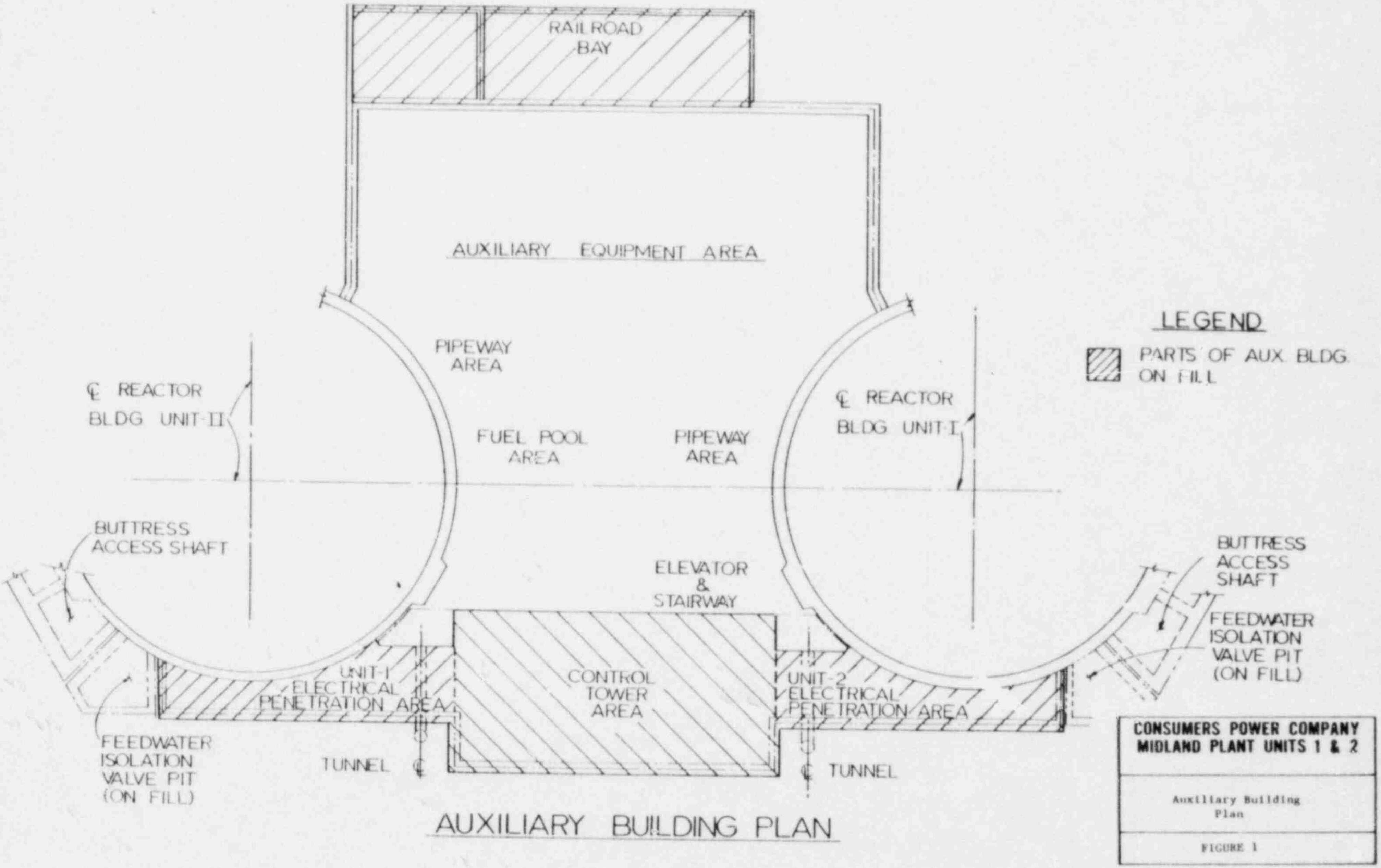
Maximum allowable stress in bending and tension is 0.9 F_y.
Maximum allowable stress in shear is 0.5 F_y.

For these load combinations, the maximum allowable stress except for local areas that do not affect overall stability is limited to 0.9 F_y for bending, bearing, and tension and 0.5 F_y for shear. The time phasing between loads is used where applicable to satisfy the above equations.

TABLE 1 (continued)

Structural components subjected to postulated impulse loads and/or impact effects are designed in accordance with BC-TOP-9-A, Rev 2, using ductility ratios not exceeding 10.

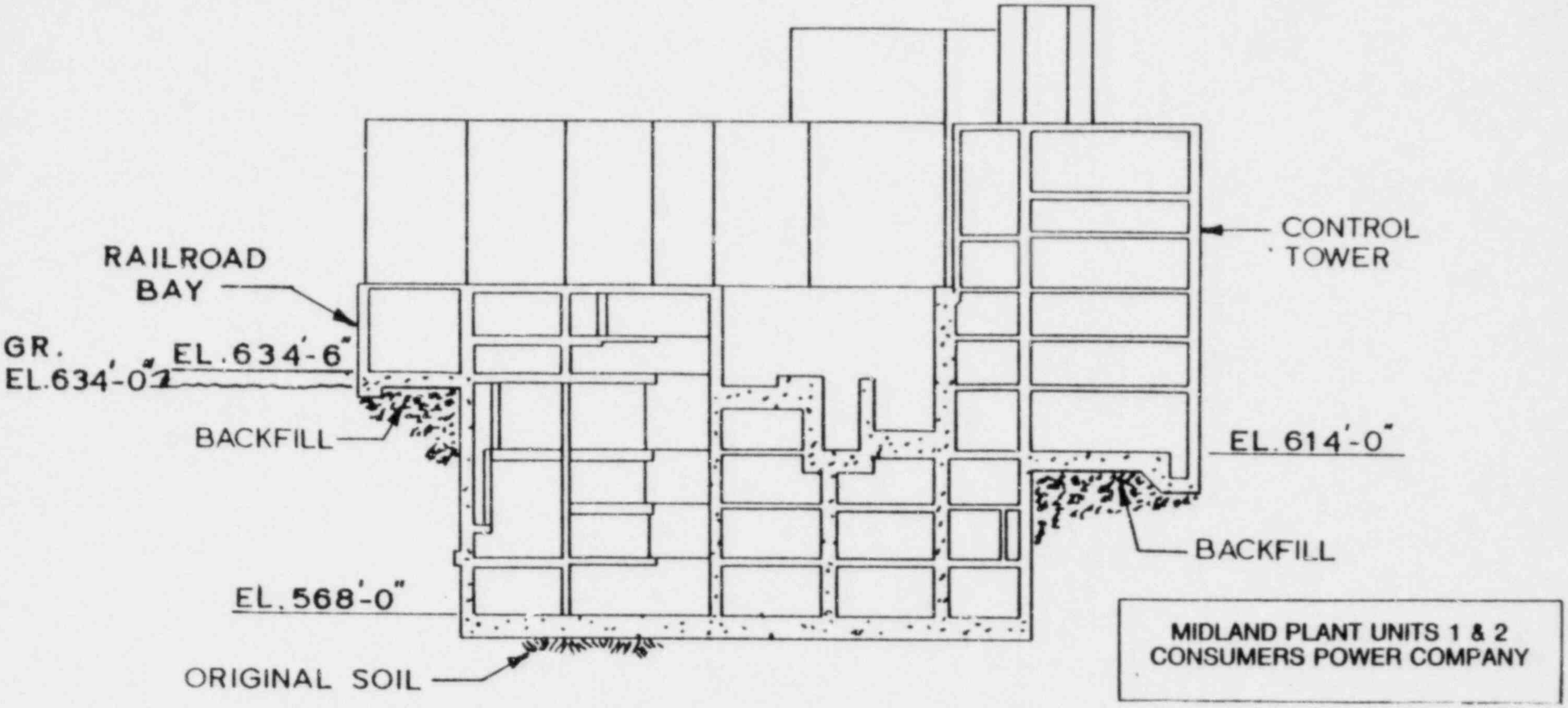
Structural members subjected to missile and pipe break loads are designed in accordance with Bechtel's BC-TOP-9-A, Rev 2, and Bechtel's BN-TOP-2, Rev 2.



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

Auxiliary Building Plan

FIGURE 1



TYPICAL SECTION (LOOKING EAST)

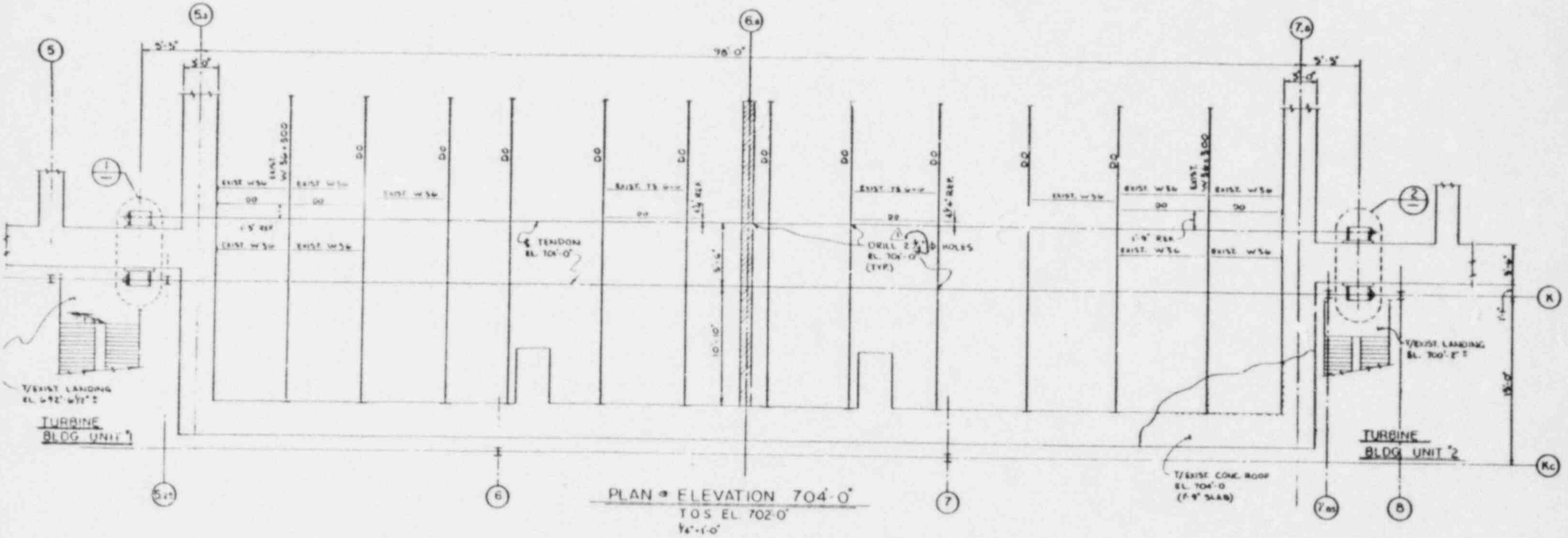
AUXILIARY BUILDING

SECTIONS

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

Auxiliary Building
Sections

FIGURE 2

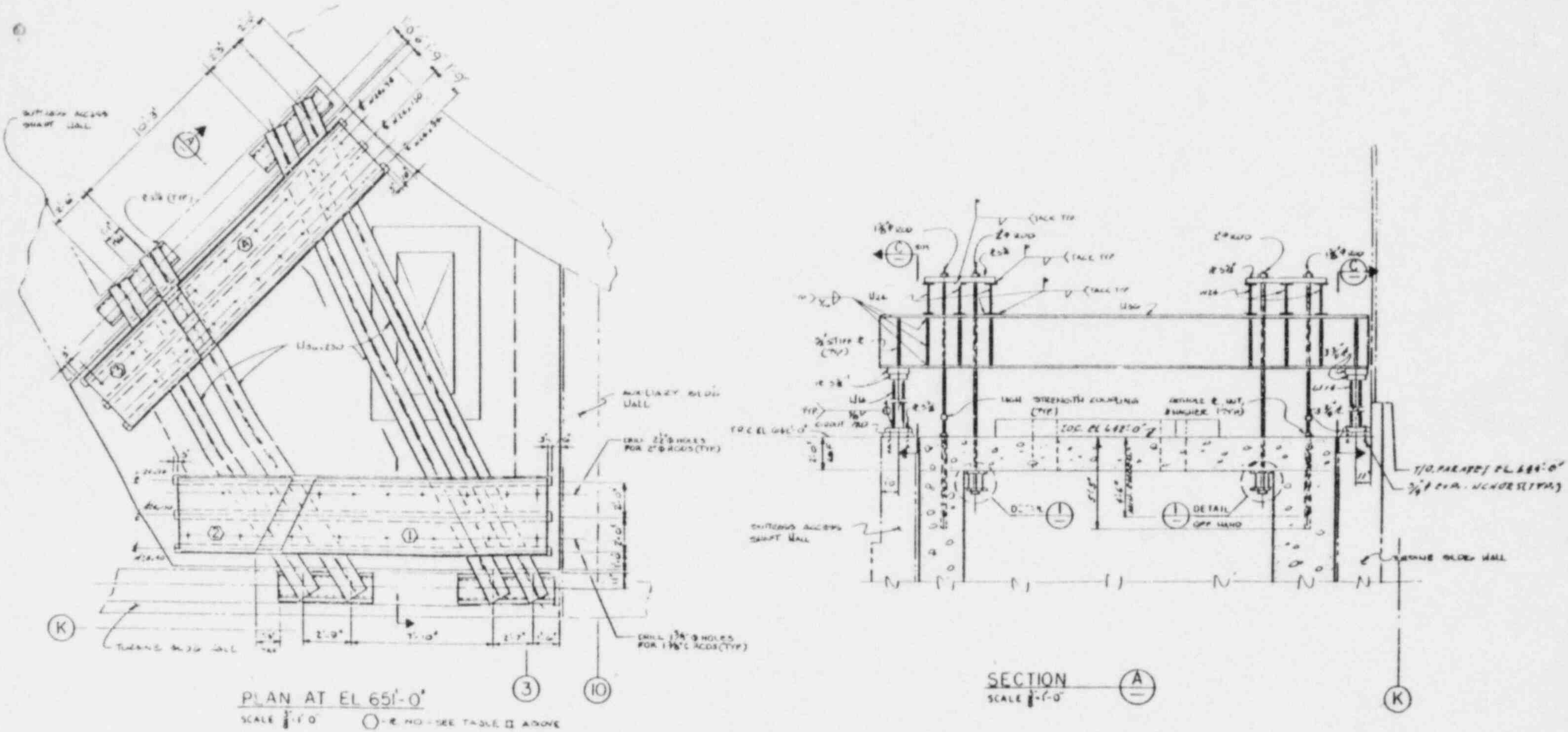


TEMPORARY SUPPORT AT CONTROL TOWER

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

Temporary Support
at Control Tower

FIGURE 7



TEMPORARY SUPPORT FOR FEEDWATER ISOLATION VALVE CHAMBER

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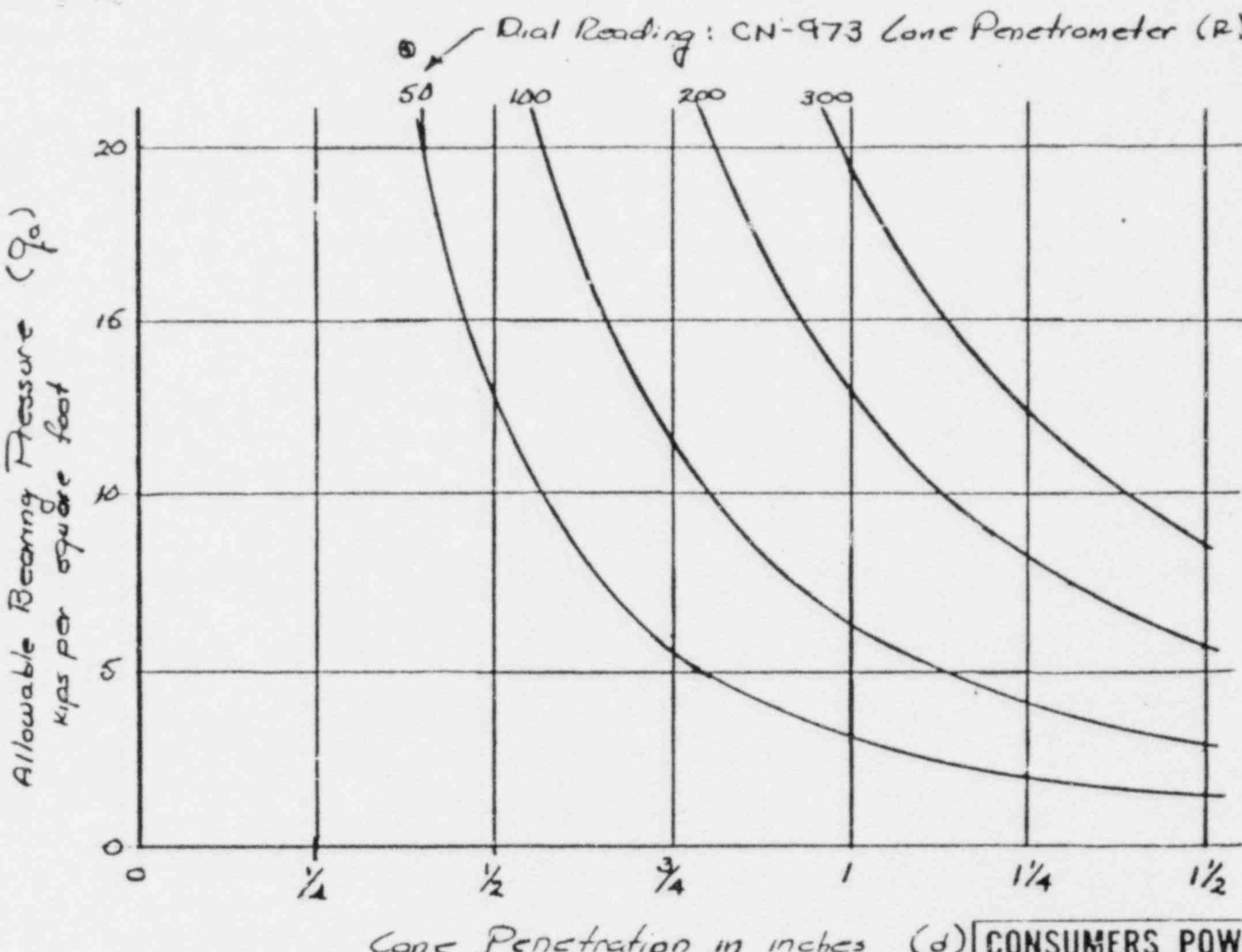
Temporary Support for Feedwater Isolation Valve Chamber

FIGURE 8



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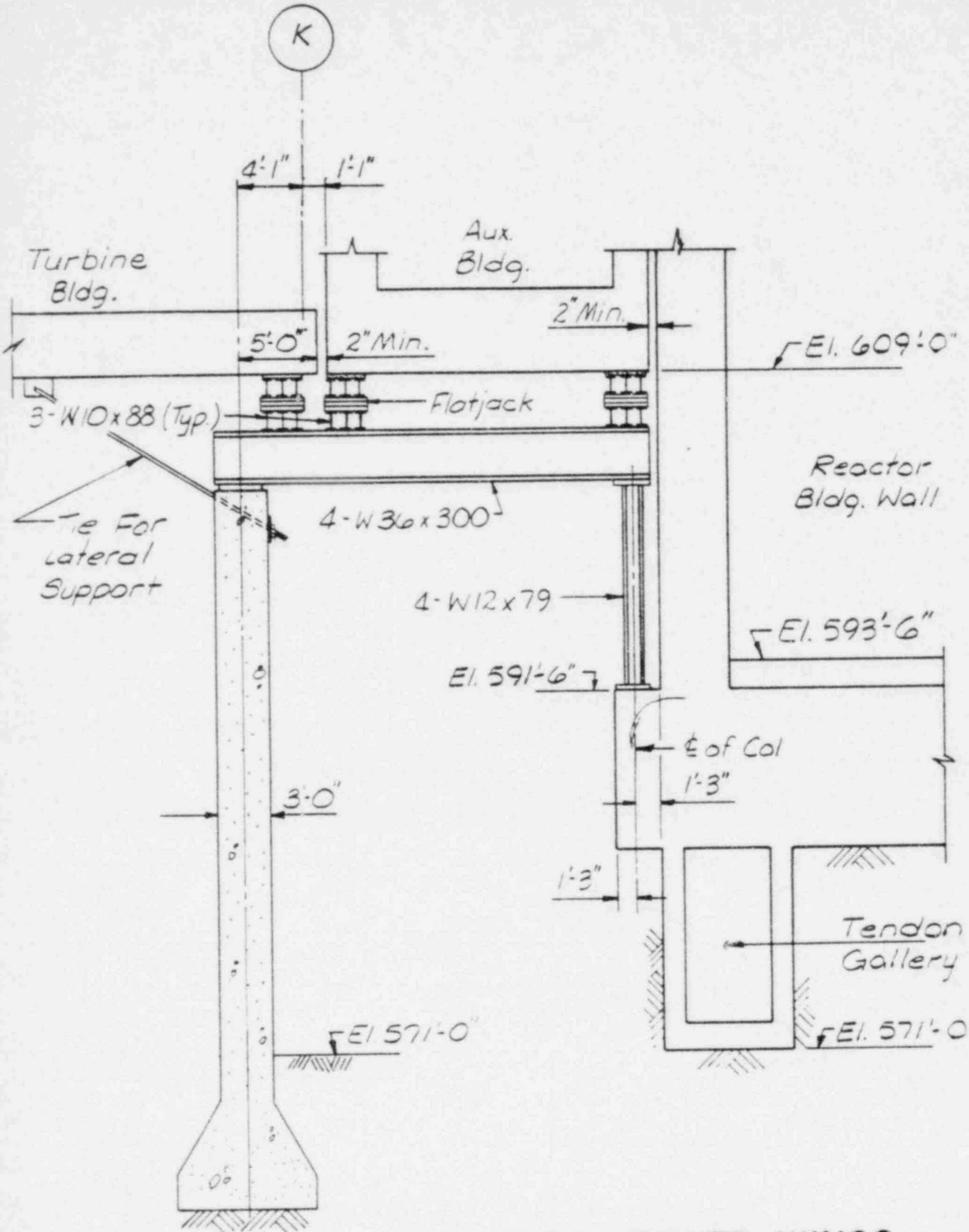
PENETROMETER CALIBRATION CURVES

Figure 9



SECTION AT CONTROL TOWER WINGS

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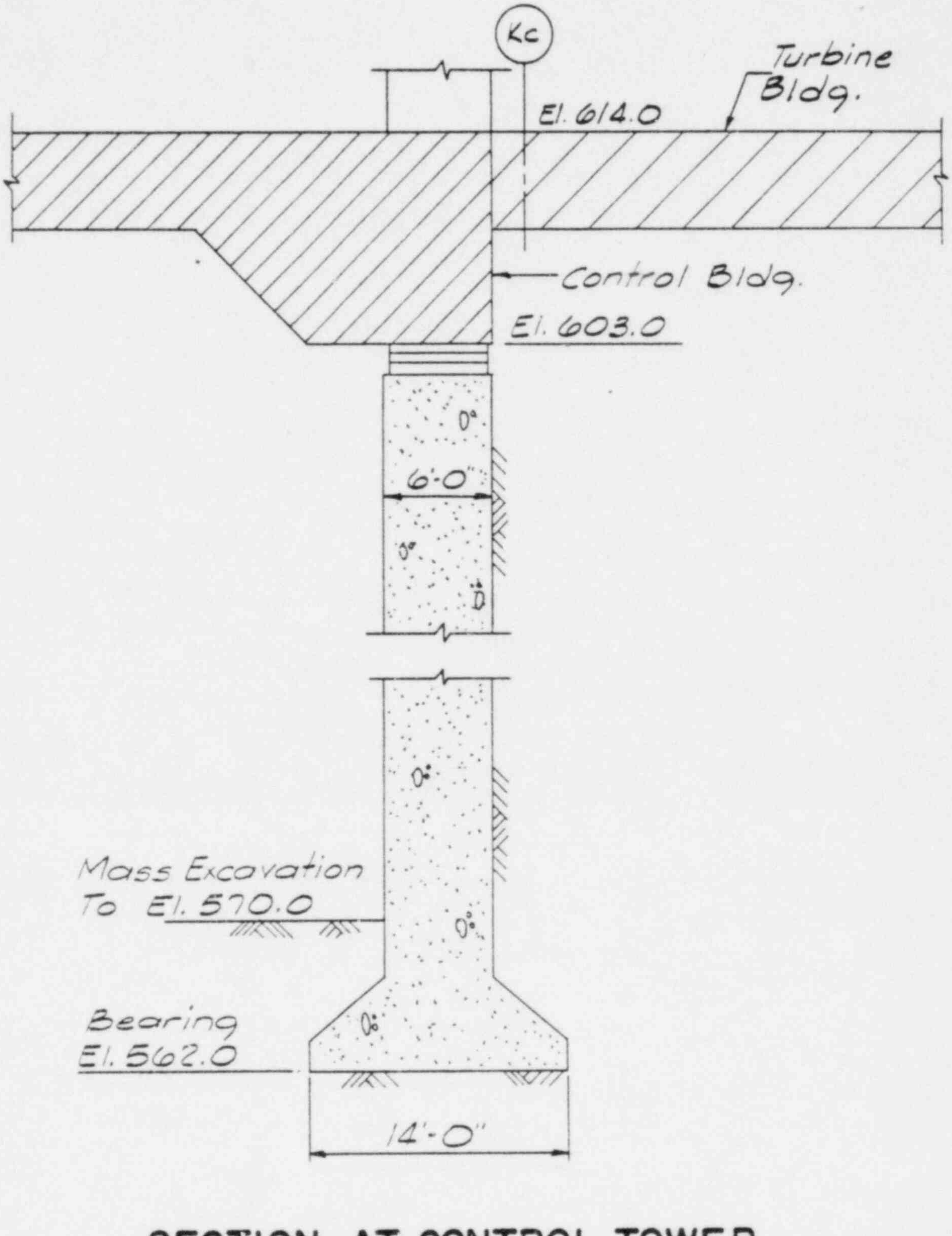
UNDERPINNING AUXILIARY BUILDING

FIGURE 10



SECTION AT CONTROL TOWER

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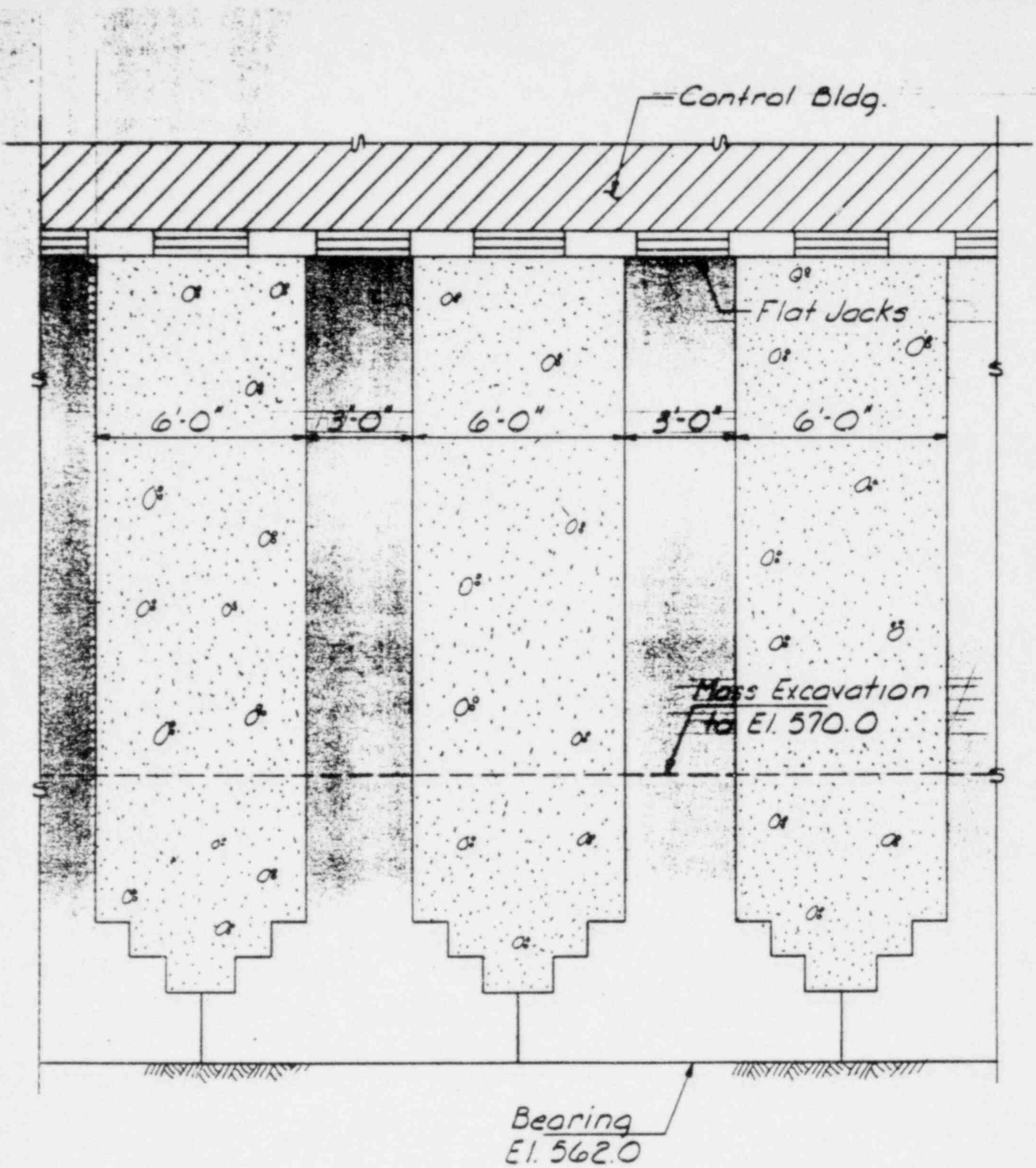
UNDERPINNING AUXILIARY BUILDING

FIGURE 11



ELEV. - UNDERPINNING FOR CONTROL TOWER

CONSUMER POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

UNDERPINNING AUXILIARY BUILDING

FIGURE 12

APPENDIX A

THREE-DIMENSIONAL FINITE ELEMENT MODELS OF THE AUXILIARY BUILDING

1.0 FINITE ELEMENT MODELS

The superstructure and underpinning of the auxiliary building are analyzed by the finite element method using the Bechtel Structural Analysis Program (BSAP). The structure is analyzed for four conditions with four different finite element models. The modeled conditions are: construction sequence of the proposed underpinning, long-term loading without connecting the underpinning to the building, long-term loading with full connection between the underpinning and building, and short-term loading with full connection between the underpinning and building.

The models consist primarily of plate elements. Beam elements are used to represent columns, minor concrete elements, and major steel components of the structure. The nodal mesh is intensified in the areas south of column line G to better represent the detail of the structure in the area significantly affected by underpinning (see Figures A-1 through A-6). The soil subbase is represented by boundary springs placed under the foundation areas. The spring constants are based on appropriate soil response predictions as dictated by the load duration.

The underpinning is modeled as a continuation of the main shear walls in the control tower and the auxiliary building wings and extends the full length under these areas.

The unique characteristics of each model are briefly described below.

1.1 CONSTRUCTION MODEL

The construction sequence model is used to investigate the construction sequence as the existing soil support of the structure is sequentially replaced by jacking loads. Several variations of this model are utilized. The only difference between variations is the total number of boundary springs which are replaced by jacking loads.

The underpinning structure is not present on this model.

The spring constants for the boundary springs reflect the soil properties prior to underpinning.

The load cases applied to the model are: dead load, live load, jacking loads, external hydropressures, soil pressures, and wind loads.

1.2 MODELS FOR LONG-TERM LOADS

1.2.1 Underpinning and Structure Disconnected

This model is used to investigate the effects of long-term loads with the underpinning disconnected from the superstructure. This model represents the construction stage when the superstructure and underpinning are separated by a series of hydraulic jacks with the jacks totally supporting the underpinned areas. Structural interaction is produced by placing upward jacking loads on the superstructure and placing equal and opposite loads on the underpinning.

The boundary springs have spring constants based on the predicted soil response to long-term loads.

The load cases applied to the model are: dead load, live load, external hydropressures, soil pressures, settlement, jacking loads, and wind loads.

1.2.2 Underpinning and Structure Connected

This model is used to investigate the effects of long-term loads with the underpinning fully connected to the superstructure.

The boundary springs have spring constants based on the predicted soil response to long-term loads.

The load cases applied to the model are differential settlement loads.

1.3 MODEL FOR SHORT-TERM LOADS

This model is used to investigate the effects of short-term loads with the underpinning fully attached to the superstructure.

The spring constants for the boundary springs are based on the predicted soil response to short-term loads.

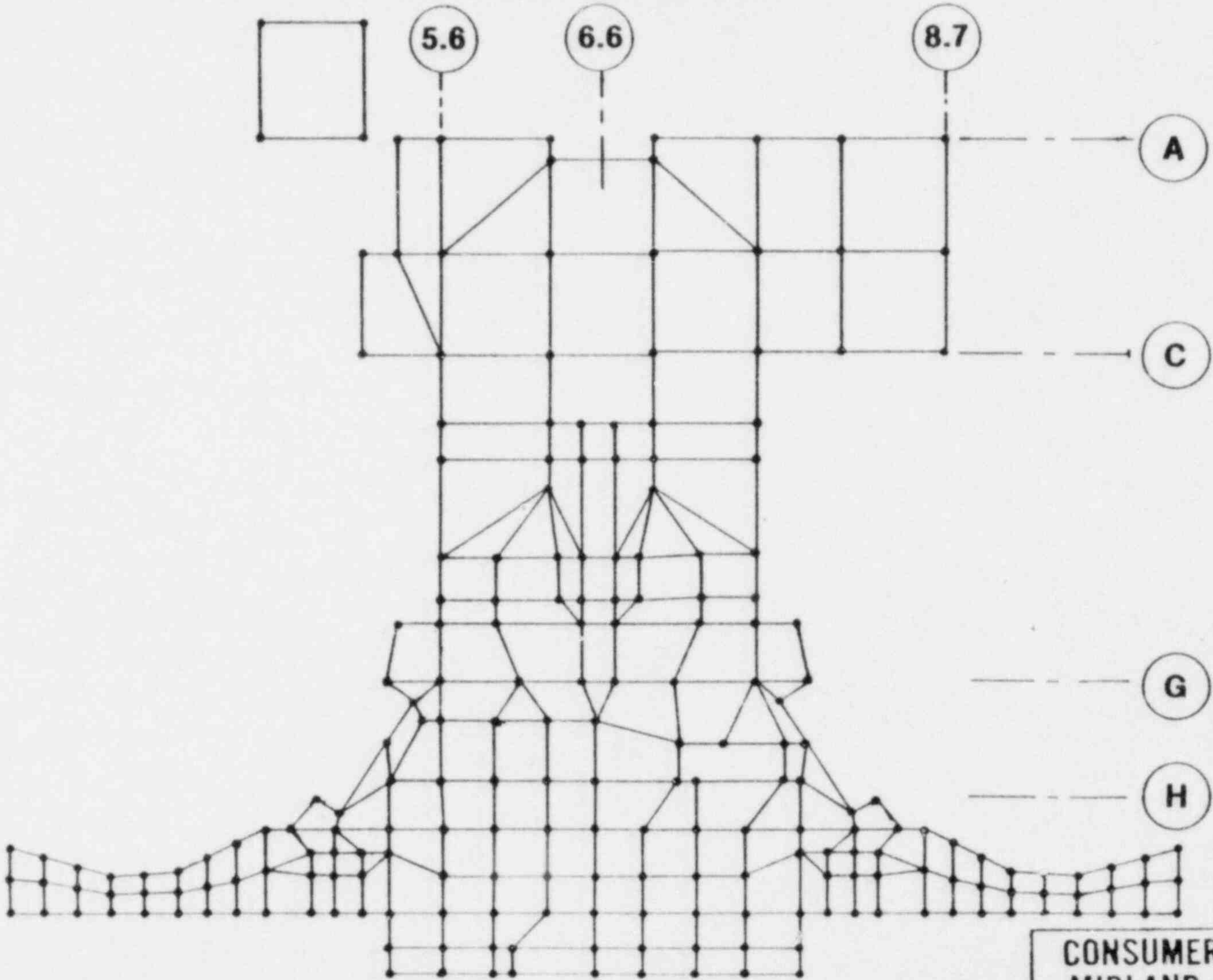
The load cases applied to the model are: east-west earthquake, north-south earthquake, vertical earthquake, tornado, and pipe rupture loads.

2.0 ANALYSIS

The results of these analyses are then factored and added in specific combinations in order to investigate the load combinations listed in Table 1 of the Technical Report on Underpinning the Auxiliary Building and Feedwater Isolation Valve Pits. These combinations are then used to evaluate structural adequacy of the structure and underpinning.

AUXILIARY BUILDING UNDERPINNING

NODAL MESH AT ELEVATION 614'
PLAN VIEW



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

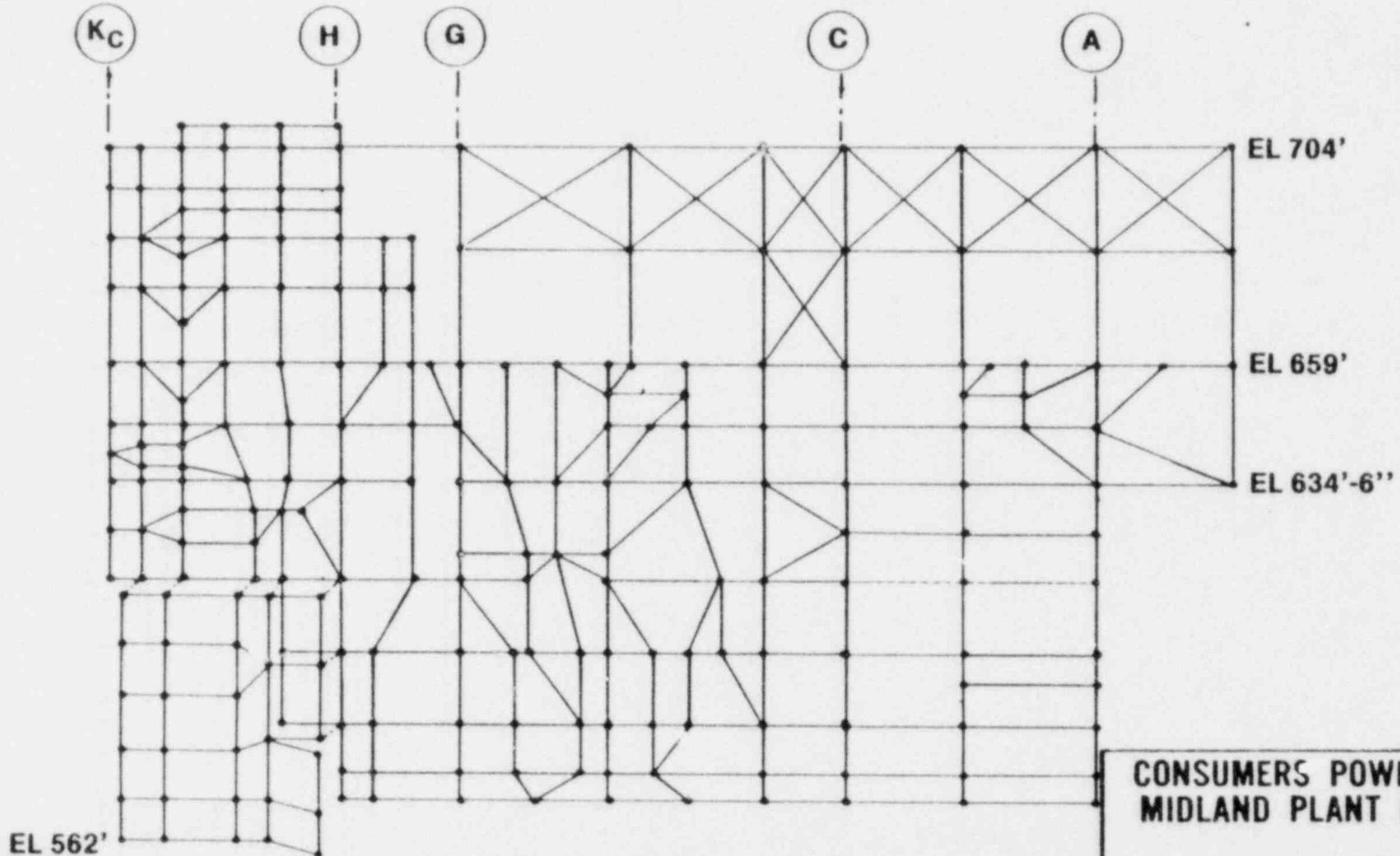
NODAL MESH AT EL. 614'

Fig. A-1

MIDLAND UNITS 1 AND 2
AUXILIARY BLDG 9/22/81

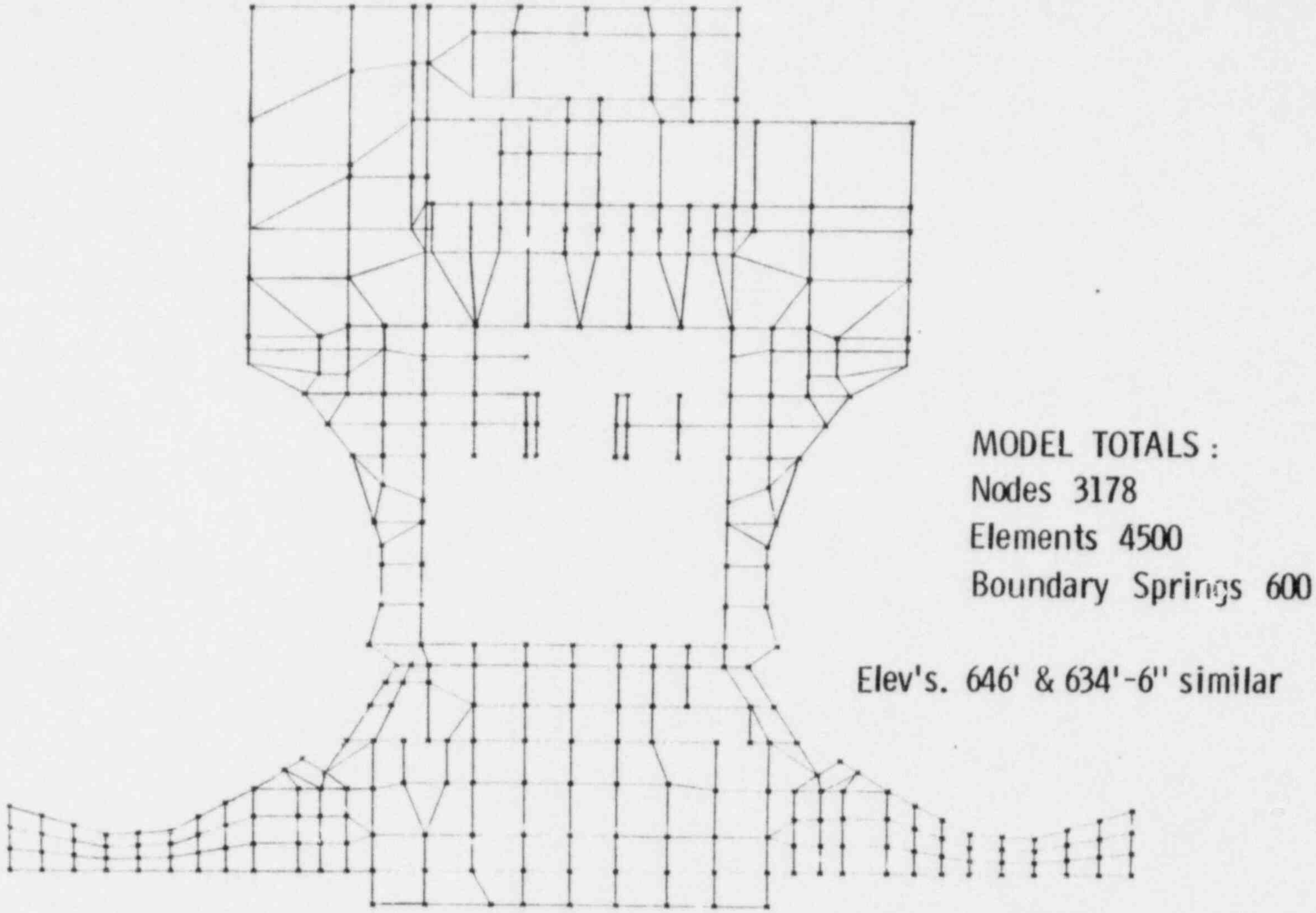
AUXILIARY BUILDING UNDERPINNING

NODAL MESH AT COLUMN LINE 5.6 ELEVATION VIEW



MIDLAND UNITS 1 AND 2
AUXILIARY BLDG 9/22/81

CONSUMERS POWER COMPANY MIDLAND PLANT UNITS 1 & 2
NODAL MESH AT COL. LINE 5.6
Fig. A-2



AUXILIARY BUILDING UNDERPINNING MODEL
NODAL MESH AT EL. 659'

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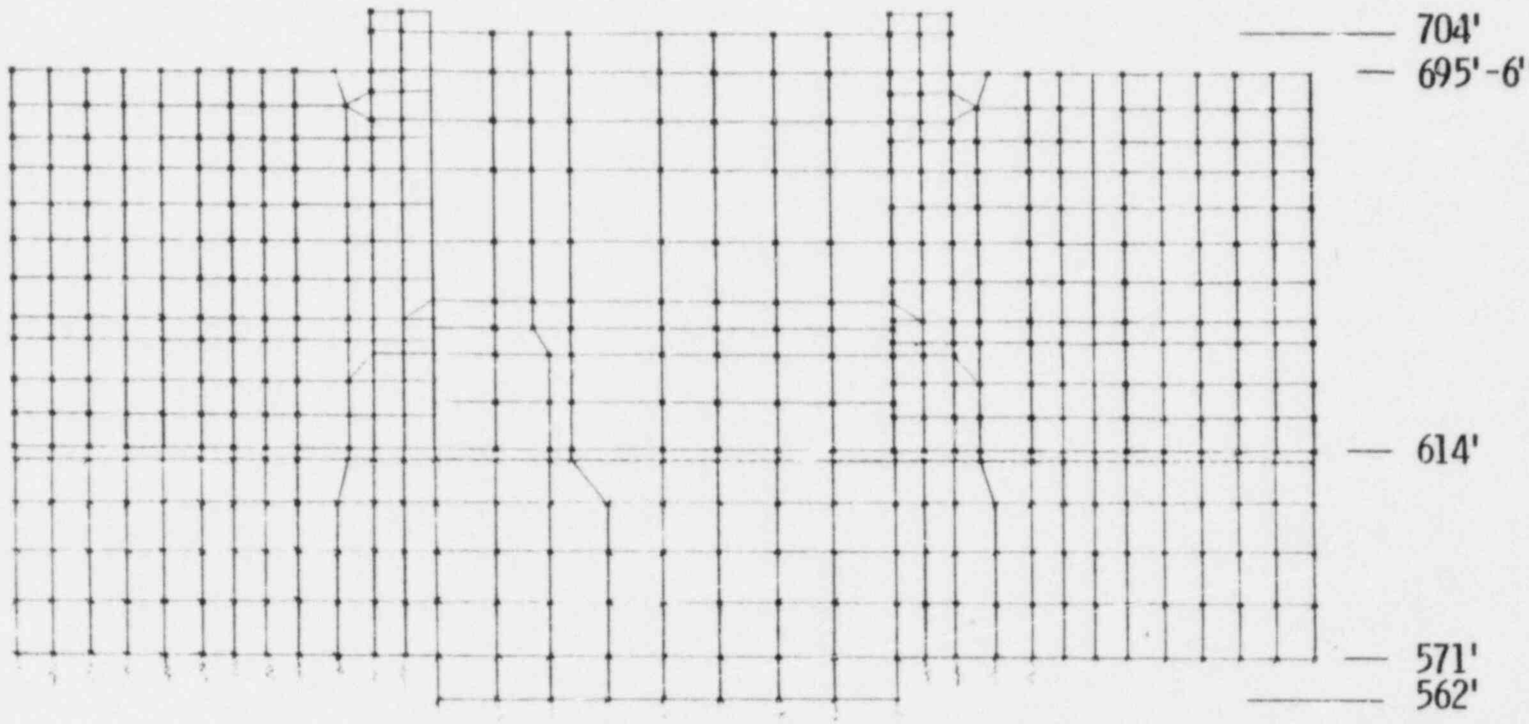
MIDLAND PLANT UNITS 1 & 2

NODAL MESH AT EL. 659'

Figure A-3

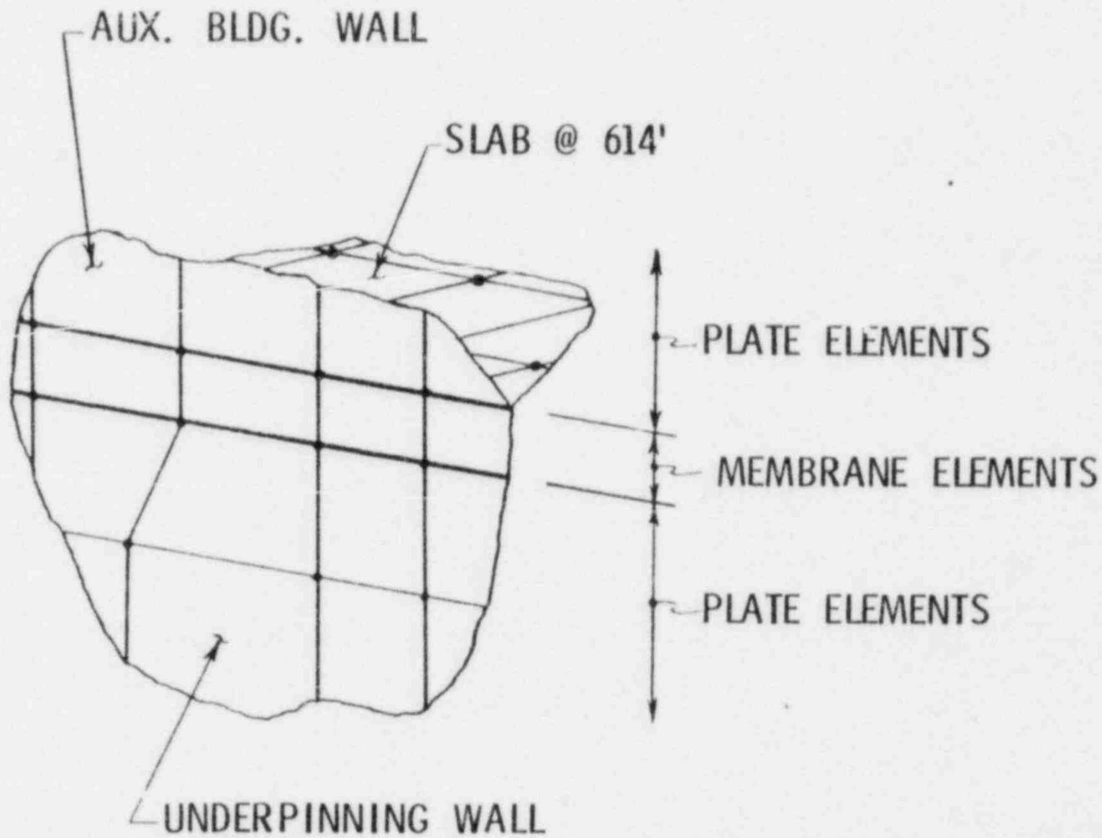


ELEVATION VIEW OF NODAL MESH
FOR COL. LINES K & K_C

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

ELEV. VIEW AT COL. LINES
K & K_C

Figure A-4

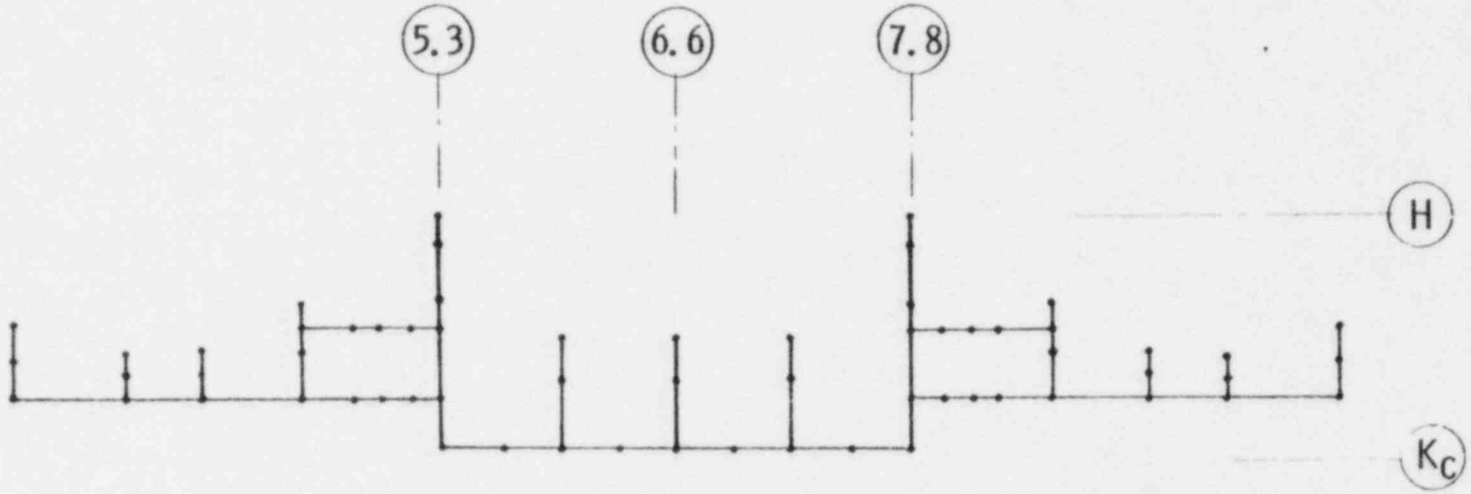


UNDERPINNING CONNECTION DETAIL

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

UNDERPINNING CONNECTION TO STRUCTURE

Figure A-5



UNDERPINNING MODEL PLAN VIEW

CONSUMERS POWER COMPANY MIDLAND PLANT UNITS 1 & 2
PLAN VIEW OF UNDERPINNING MODEL
Figure A-6

APPENDIX B

GROUNDWATER CONTROL

Underpinning the auxiliary building requires excavating approximately 9,000 cubic yards of material from underneath the structure and replacing that material with structural concrete and backfill.

The geohydrological environment in which this excavation is to be performed consists of approximately 40% undisturbed natural clay, 10% unreinforced concrete, and 50% heterogeneous fill. The heterogeneous fill contains free water. To perform the underpinning work safely and within a reasonably predictable period of time, the free water must be controlled so that it does not produce significant settlement of the immediate and surrounding structures or significantly impede progress of the underpinning work.

One well known groundwater control technique is predrainage. Presently, 20 permanent plant dewatering wells are being installed to intercept the seepage from the cooling pond. A predrainage system is also in place at the site of the planned excavation. This predrainage system has a demonstrated capability for lowering the existing groundwater from approximately el 627' to el 595. El 595 is a maximum of 10 feet below the bottom of the structures where the underpinning excavation is to be performed. Ultimately, the underpinning excavation will reach el 562, or 33 feet below the level dewatered by the existing predrainage system.

The existing predrainage system is currently recharged at a rate of approximately 60 gpm, which is expected to decrease with the installation and operation of the permanent dewatering wells. The recharge is occurring at unknown locations around 380 feet of the 700-foot perimeter of the planned excavation. The perimeter of this excavation is not accessible from the existing ground surface except for a distance of approximately 40 linear feet in an area located between the isolation valve pits and the turbine building. The majority of the existing predrainage eductor system is located in this area. There are several other predrainage points which are located in the immediate vicinity of the planned excavation. These predrainage points are located in presently inaccessible areas in the basement of the turbine building.

Such a predrainage system will permit access shafts to be excavated from the ground surface to approximately el 600 and will permit approximately 7 feet of material to be excavated underneath the existing structures. Thus, this predrainage system will provide safe access to the area to be excavated for underpinning.

The present plans for excavating approximately 9,000 cubic yards of material for underpinning requires that shafts be sunk from el 600 to 571 or 562, depending on the shaft location. The present plan for sinking the shafts is to hand excavate the material and place lagging to support the surrounding material as the shaft is progressed vertically. Such a construction method can only be safely performed with very minor water seepage into the shaft excavation. Thus, it is essential that groundwater inflows be substantially reduced to a predictable volume and location.

The configuration and location of the geohydrological environment is defined by the initial excavation for the site as shown in Figure B-1, which shows a large open cut excavation from the original ground surface elevations of approximately 605 to 615 to an ultimate depth at el 562. This original excavation has been filled with structures and backfill. The reactors and portions of the auxiliary building are founded on natural clay and form an essentially impervious barrier over their length. The rest of the structures and the manmade fill do not cut off groundwater movement. The cooling pond water elevation is approximately 627 and constitutes the major charging source of the fills and natural sands located below all the structures except the reactors and a portion of the auxiliary building, which are founded on clay.

The proposed underpinning is located at the deepest level of the original general site excavation. It is bounded by the impermeable reactor and auxiliary building on one side, and by the steep side slopes on the general site excavation on the other.

The location of the groundwater recharging conduits is unknown, although it is known that the conduits do not pass through or under the reactor and a portion of the auxiliary building. Because of this, a groundwater control plan that intercepts or cuts off recharge water must be of a nonspecific nature in that it must function over the entire potential recharge zone.

The frozen earth membrane method has been selected as a groundwater cutoff (interception) plan for the following reasons.

a. Proof of its continuity is easily monitored.

b. Its extent can be discriminately controlled at specific locations by the input of coolant.

c. Spacing of the freeze pipes can be readily adjusted to deal with interferences and yet continuity of the membrane can be ensured.

d. Its vertical extent can be varied after intial installation.

e. Membrane formation times are predictable.

f. When its function is completed, it is totally degradable and will permit groundwaters to behave as if the system has never been used. Thus it would not interfere with or jeopardize the expected performance of the permanent dewatering system.

The preliminary plan location for installing the frozen earth membrane is shown in Figure B-2. This location was chosen for the following reasons.

a. It does not interfere with the planned groundwater recharge program particularly as it relates to the diesel generator building.

b. It is the most efficient method of intercepting, or cutting off, recharge water in open and accessible areas.

c. It removes the coolant circulating lines from trafficked areas.

d. It is close to the deep well system and will benefit from the deep well dewatering.

El 610 has been preliminarily selected as the membrane crest height (as shown in Figure B-3) for the following reasons.

a. It forms a barrier in the granular (SP class) soil materials, which are suspected of being a major recharge aquifer.

b. It minimizes operational energy costs.

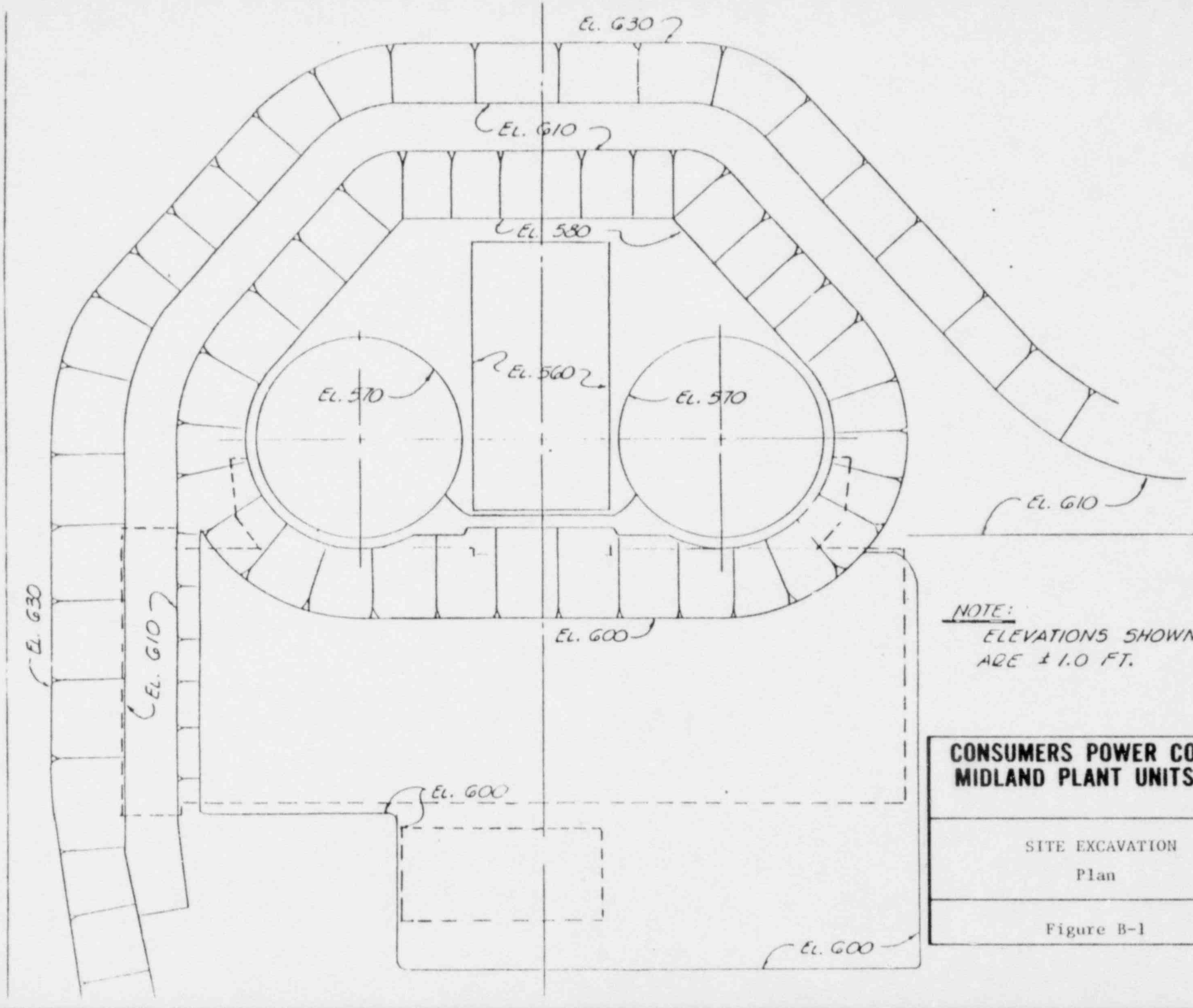
c. It allows for maximum benefit from the drawdown effect of the planned predrainage system when it becomes operational.

Attached Figures:

B-1 Site Excavation - Plan

B-2 Frozen Earth Membrane - Proposed Location

B-3 Frozen Earth Membrane - Typical Section



NOTE:
ELEVATIONS SHOWN ARE ± 1.0 FT.

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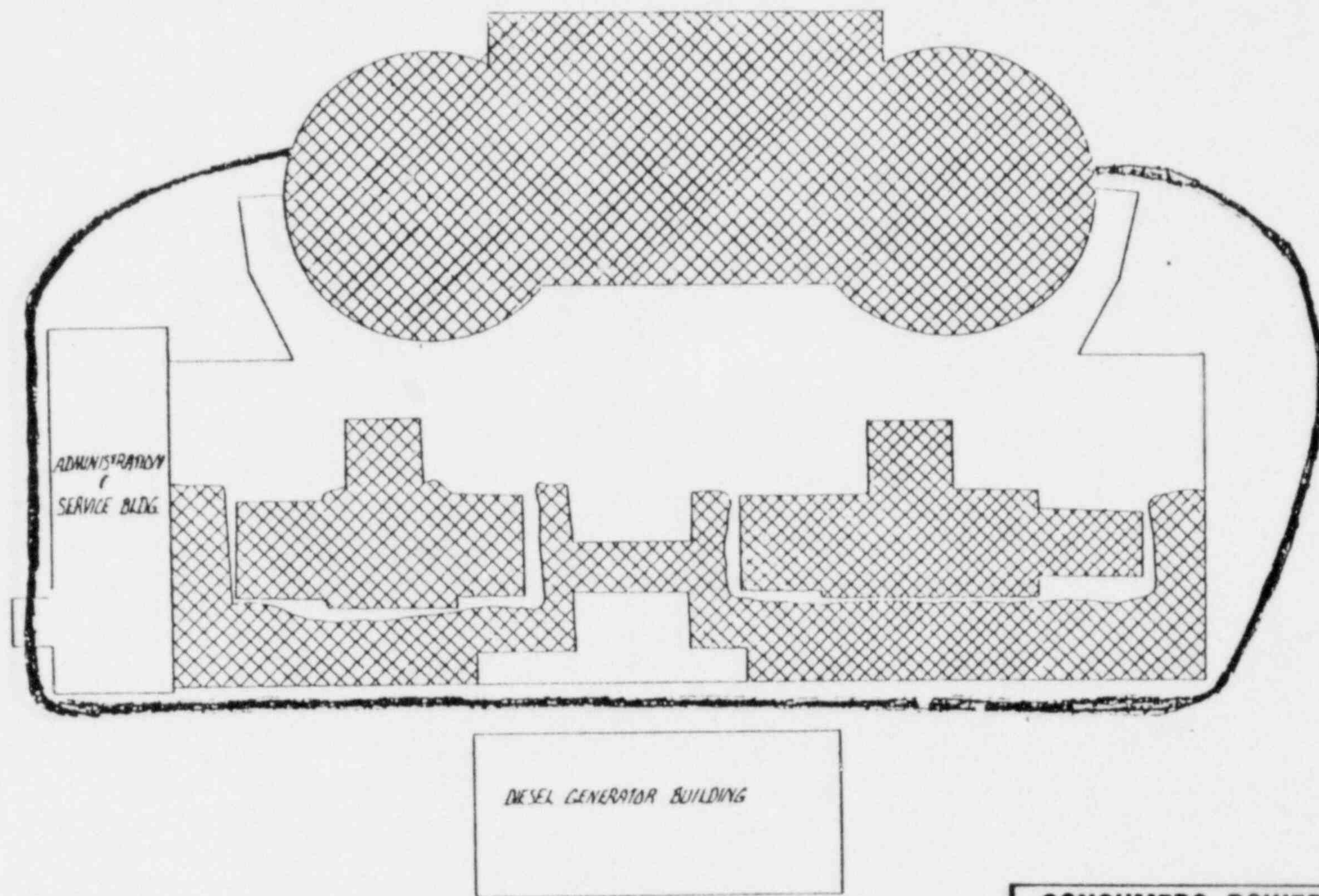
SITE EXCAVATION

Plan

Figure B-1



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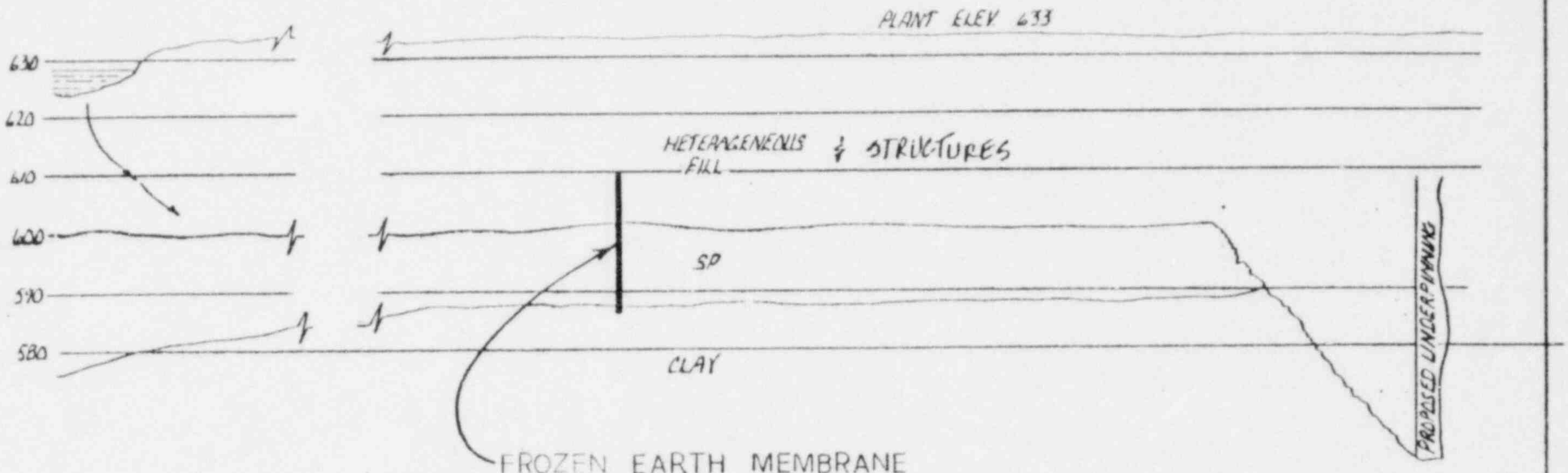
FROZEN EARTH MEMBRANE

Proposed Location

Figure B-2



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FROZEN EARTH MEMBRANE

Typical Section

Figure B-3

APPENDIX C

CONSTRUCTION DETAILS

The material presented herein is based on preliminary analysis and concepts, and must be checked in detail and adjustments made accordingly.

The objective of the underpinning construction plan is to complete the underpinning work in such a manner that there will be no intolerable stresses or strains imposed on the existing Auxiliary Building or Feedwater Isolation Valve Pits.

The strategy which will be employed to serve this objective is to reduce potentially high levels of existing stress or strain prior to removing any significant portion of the existing subgrade support for the structure.

The tactics which will be employed are as follows:

a. Use temporary support to reduce potentially high levels of existing stress or strain.

b. Install initial support at locations which require the minimum disturbance at the existing subgrade support.

c. Activate currently unused existing structure strength characteristics to effect reduction of potentially high levels of existing stress or strain.

The areas in the existing structure which have the potential for having the highest level of stress or strain that may be approaching an intolerable limit are, in order of priority:

a. The junction between the control tower and the wings in the general area of Column Lines 5.3 or 7.8.

b. The junction between the control tower and the main Auxiliary Building, in the general area of Column Row H.

With respect to Item (a) above, the potentially high stress is tension in the upper portion of the junction.

With respect to Item (b) above, the potentially high stress is either tension in the upper portion of the junction or shear across the junction.

Both junctions have not exhibited intolerable stresses or strains when buoyant support of approximately 1.3 ksf was lost due to temporary dewatering. Both the wings and control tower structures behaved as cantilevers when the buoyant support was removed. The settlement measurements indicate that the control tower behaved more as a rigid body than the wing when the buoyant support was removed.

In addition to the aforementioned structural considerations, the construction plan must address the existing subsurface conditions. The most noteworthy subsurface conditions are as follows:

a. The soil immediately under the wings is in an indeterminate state of compactness.

b. Underlying the soil immediately under the wing is a layer of unreinforced concrete which averages six feet in thickness and bears on natural clay.

c. The soil immediately under the control tower is an adequate state of compactness. It is underlain by layers of concrete varying in thickness from 1 to 2 feet, except in the vicinity of the utility tunnels where the concrete which bears on natural clay is as much as 15 feet thick.

The permanent underpinning must be founded on undisturbed natural clay. This means that unless it can be proven that the clay in contact with the concrete is in an undisturbed state, approximately 1,000 cubic yards of concrete must be demolished and excavated.

The detailed preliminary underpinning construction plan which best copes with the existing conditions and meets the objective is developed below and shown in graphic form in Appendix C, Figures 1 through 13. The construction plan incorporates protective construction for the Turbine Building and Buttress Access Shafts, which is shown in the graphic exhibits and explained at the end of this section.

A. Auxiliary Building and Feedwater Isolation Valve Pits Underpinning

1. Install temporary support at the open end of both wings

   a. Locate the bearing support for the temporary support so that it minimizes the amount of concrete to be removed.

   b. Provide sufficient bearing capacity to develop that maximum structural capacity of the existing structure when the existing subgrade support is neglected.

   c. Load test large temporary support pier founded in undisturbed clay.

   d. Preload temporary support to an amount yet to be determined, which will reduce potentially high levels of tension stresses near the top of the wings in the vicinity of Column Lines 5.3 and 7.8. The preload will also have the effect of establishing the wing structure as a propped cantilever and thus remove any doubt as to the structural behavior of the wings.

   e. Perform all the aforementioned work prior to the removal of any subgrade support from under the control tower or the wings except for the outermost 8 feet.

   f. If the subgrade under the wing was supporting the structure to some degree prior to preloading of the temporary support, some amount of the structure load supported by the subgrade may be transferred to the control tower when the temporary support is preloaded.

      Since none of the subgrade under the control tower has been disturbed, this is the best time to transfer load to that area. (The control tower subgrade is not only undisturbed, but it is also in its original state of confinement.)

Since the preload on the temporary support at the end of the wings will be introduced incrementally, the behavior of the control tower structure can be monitored. If the control tower cannot tolerate part or all of the load imposed by the wing preload, then the preload will be stopped at that point and pits will be installed under the control tower to supplement the control tower capacity so that the full preload from the wings can be distributed. If this situation were to arise, no further excavation from under the wings would occur until the full wing preload were in place.

2. Start installation of permanent support at control tower using pit method.

   a. Preload to an amount yet to be determined which will reduce potentially high levels of stress in the vicinity of Column Row H.

   b. Adjust preload at wings to bring temporary support load to predetermined amount.

3. Install temporary support at middle of wings and continue installing permanent support under the control tower.

   a. Provide sufficient bearing capacity so that the temporary support is capable of supporting the wing as a propped cantilever without failing and without structure support from the previously installed temporary support.

   b. Load test large temporary support founded in undisturbed clay.

   c. Preload temporary support to an amount yet to be determined, but which results in it assuming a major portion of any load previously shifted from the wing to the control tower.

4. Install last temporary support under wing and continue installing permanent support under the control tower.

5. Start mass excavation and lagging for support of excavation under wings.

6. Complete permanent support under control tower.

7. Excavate to electric ducts under control tower and temporarily support same.

8. Excavate and install lagging for support of excavation under control tower.

9. Install bracing for support of excavation as required.

10. Perform concrete demolition as required under wings and develop tunnel under existing pipeway.

11. Complete demolition and excavation under control tower.

12. Construct permanent underpinning under valve pits, wings and control tower at Column Lines 5.3 and 7.8, and complete permanent underpinning under control tower along Column Row K.C.

13. Install permanent load transfer and long term load test jacking equipment and start backfill of mass excavation and removal of support of excavation under wings.

14. Perform long term load test.

15. Complete long term load test, transfer structure load to permanent underpinning.

16. Grout dowels between permanent underpinning and existing structure.

17. Remove temporary support upon completion of load transfer.

13. Complete backfill as required.

19. Backfill access shaft.

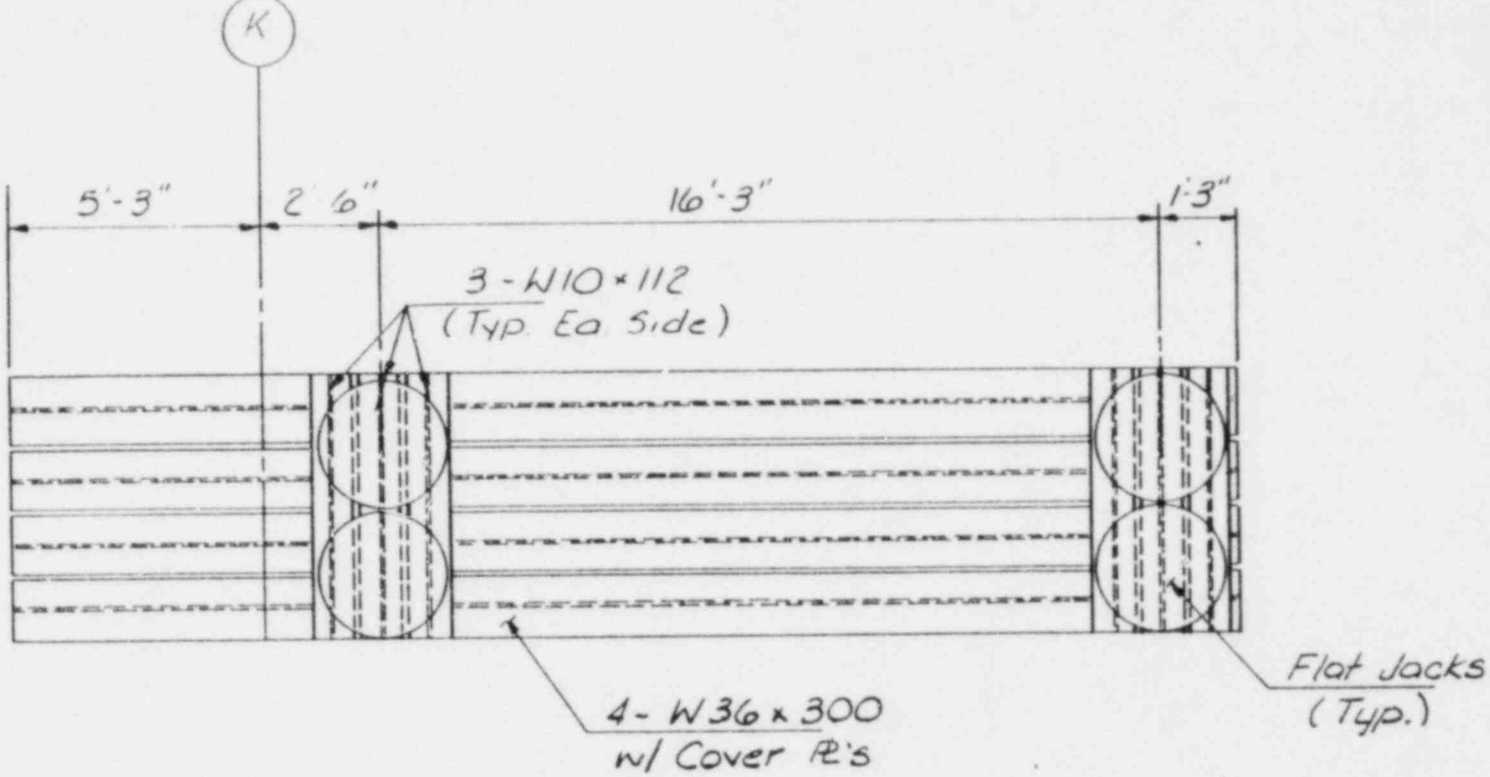
B. Turbine Building and Buttress Access Shaft Protective Construction

1. Underpin Turbine Building where it is loaded from the feedwater isolation valve pit temporary support reaction, in the vicinity of Column Row K and Column Lines 2.5-3.0 and 10.0-10.5.

2. Underpin Buttress Access Shafts.

3. Underpin Turbine Building as shown in Exhibit 1, but Auxiliary Building sequence of work always controls priority and sequence of work.

LIST OF FIGURES



PLAN VIEW - TRANSFER BEAMS & JACKS

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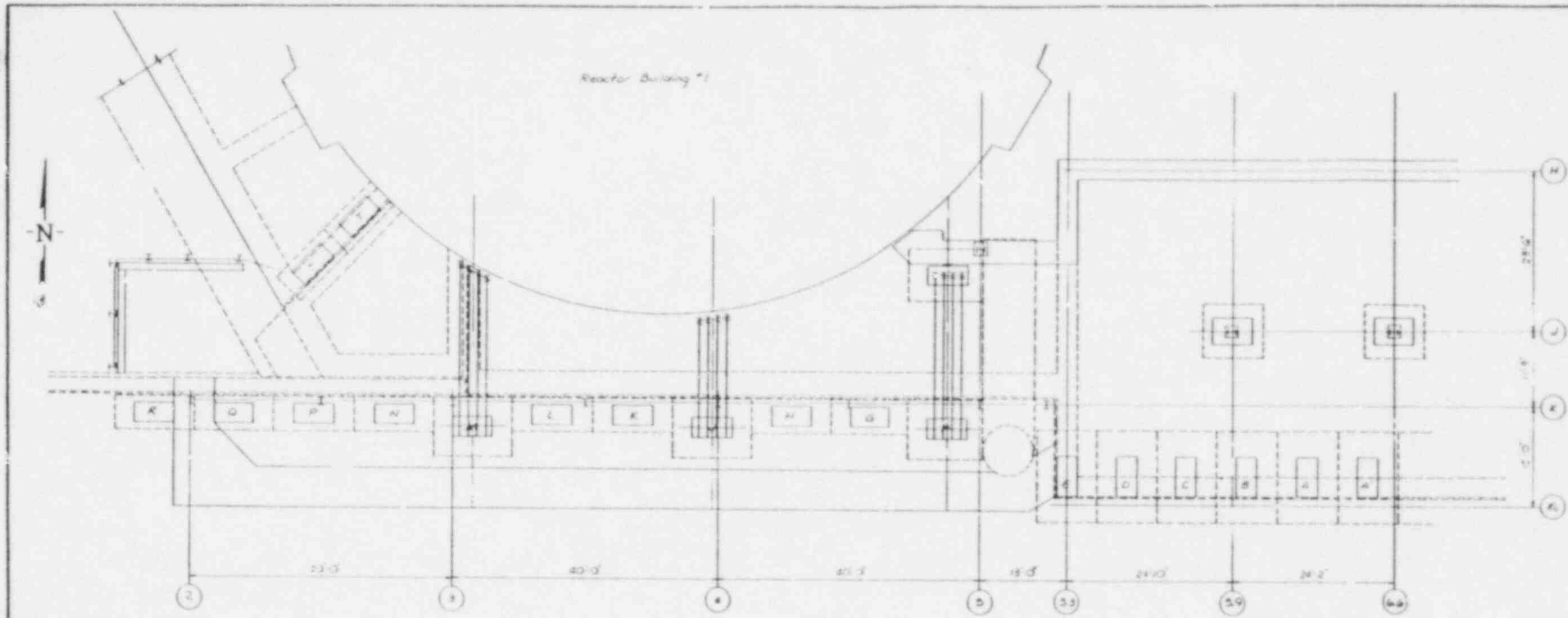
MIDLAND PLANT UNITS 1 & 2

UNDERPINNING AUXILIARY BUILDING

FIGURE 13



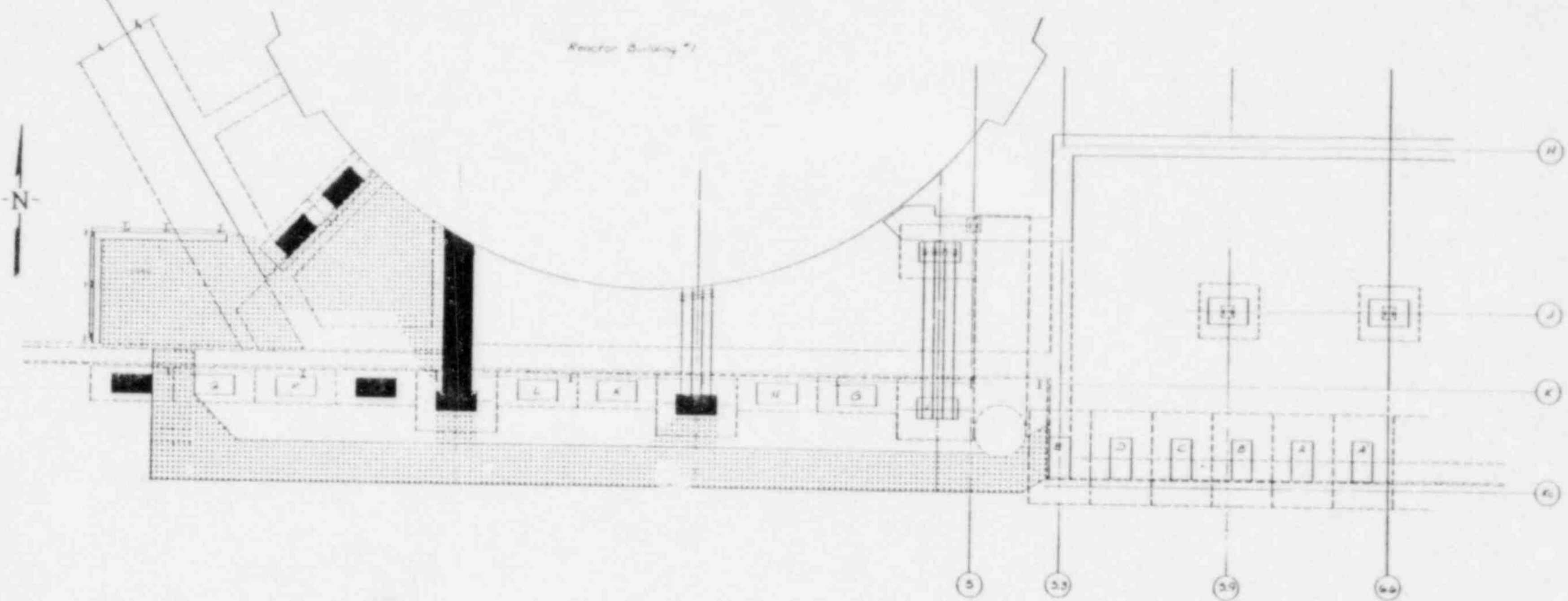
GENERAL PLAN
(WEST SIDE)

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

CONCEPT DRAWING
UNDERPINNING AUXILIARY BUILDING

GENERAL PLAN

APPENDIX C FIGURE 1



GENERAL PLAN
(WEST SIDE)

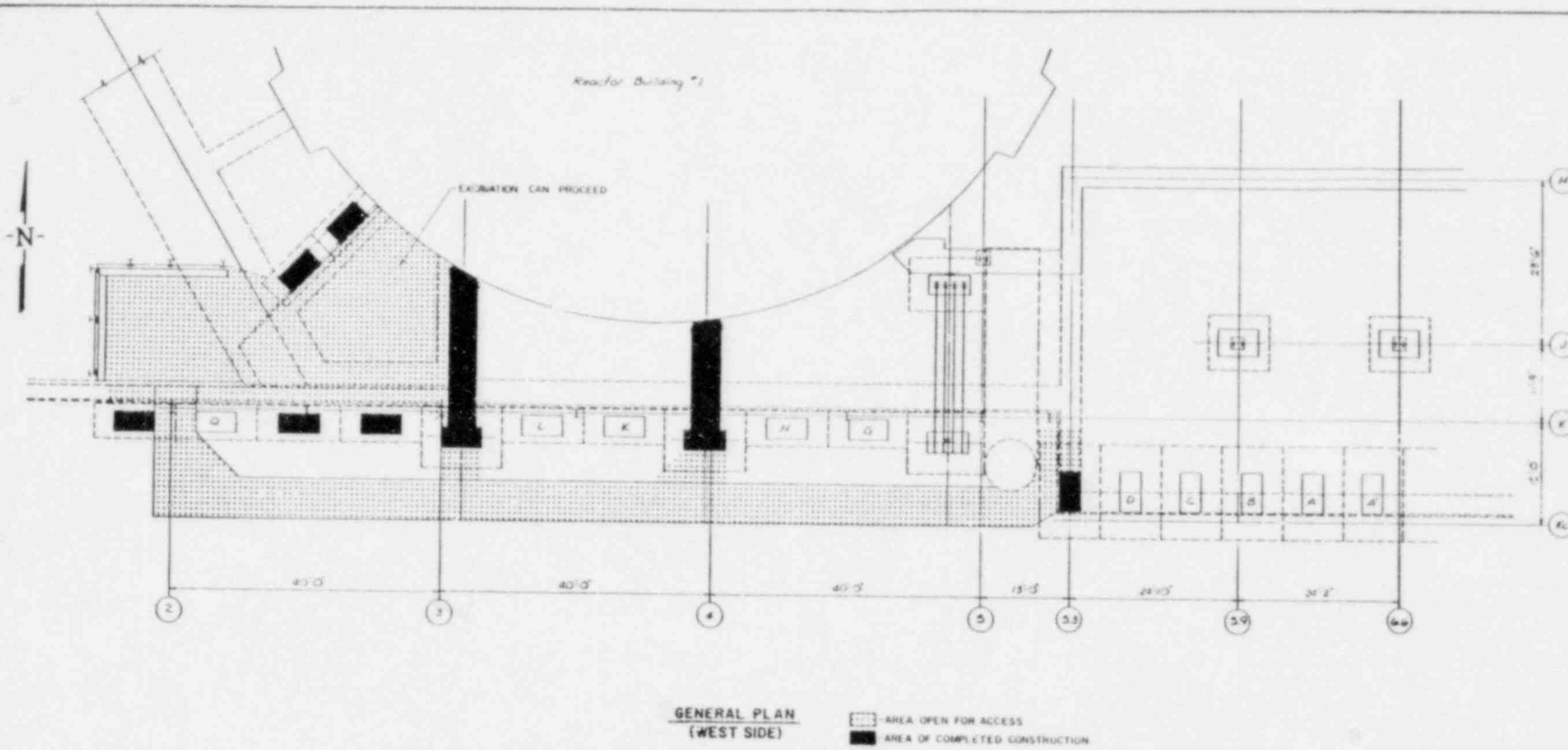
AREA OPEN FOR ACCESS
AREA OF COMPLETED CONSTRUCTION

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

CONCEPT DRAWING
UNDERPINNING AUXILIARY BUILDING

CONSTRUCTION AFTER 13 WEEKS

APPENDIX C FIGURE 2

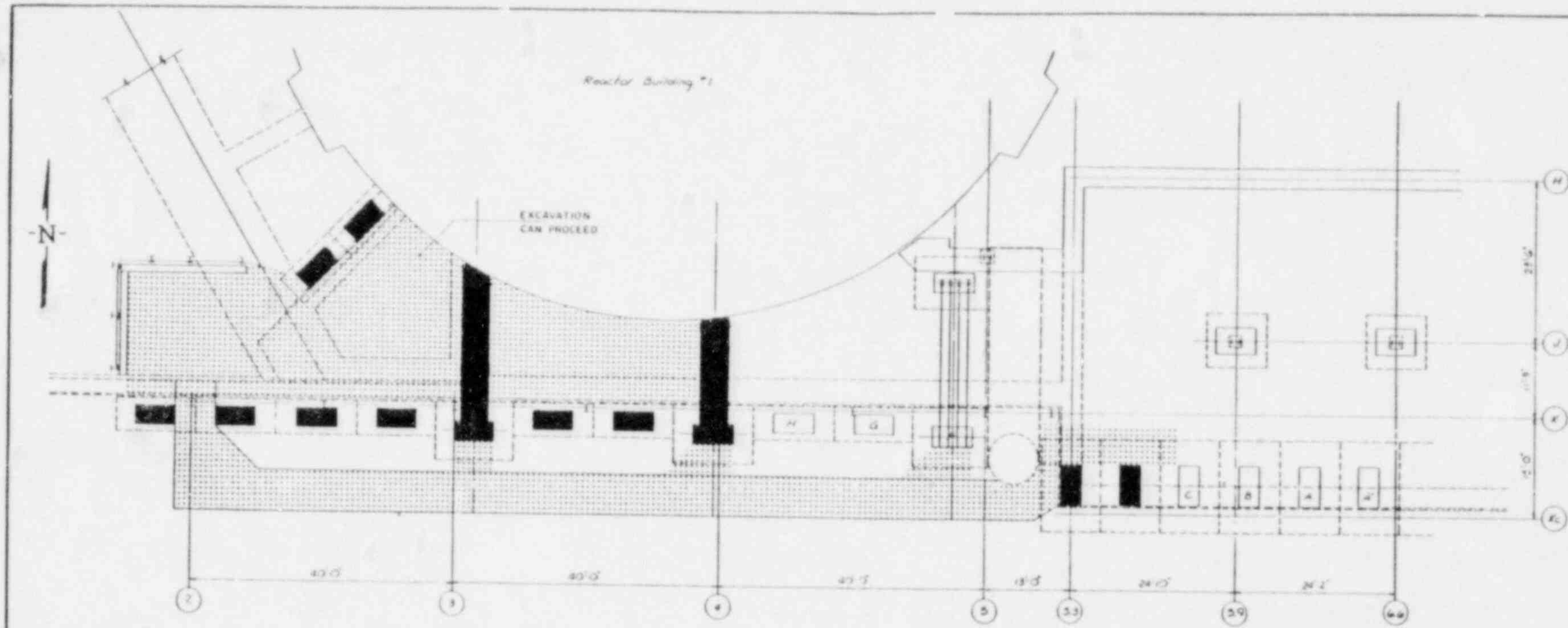


CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

CONCEPT DRAWING
UNDERPINNING AUXILIARY BUILDING

CONSTRUCTION AFTER 16 WEEKS

APPENDIX C FIGURE 3



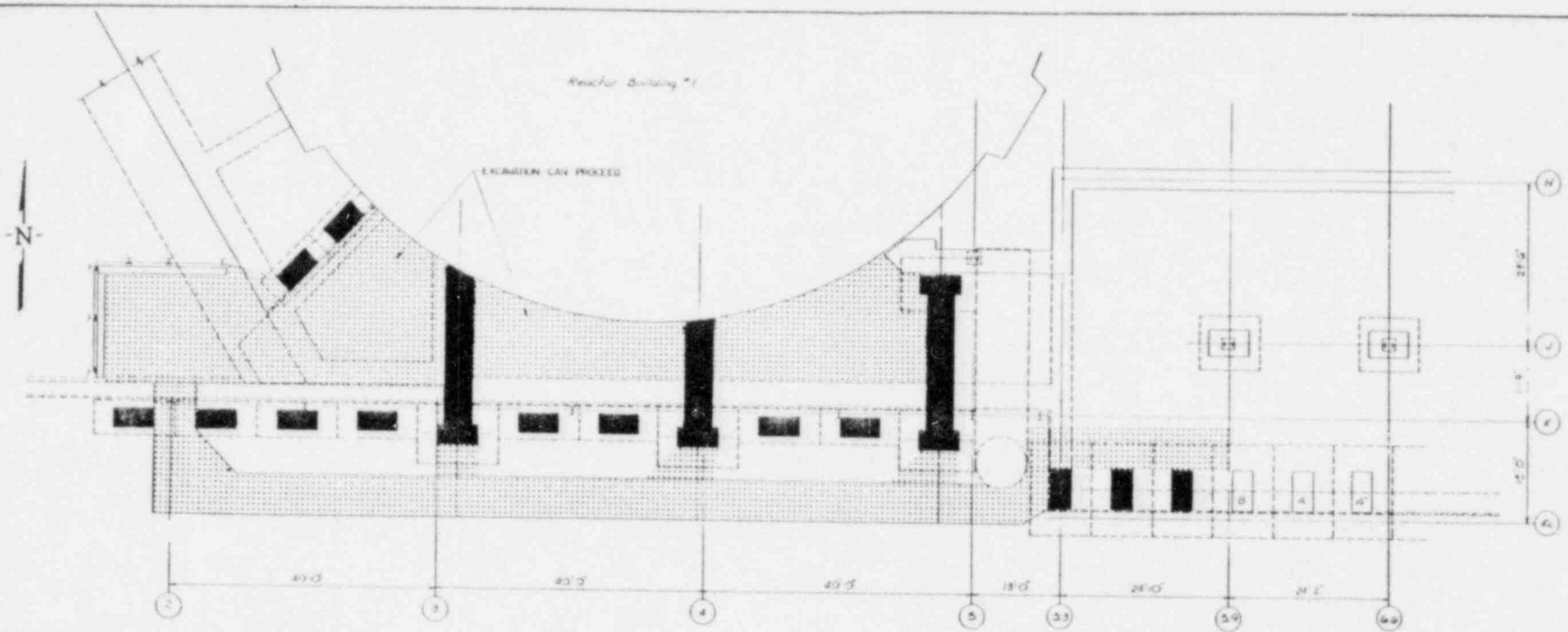
GENERAL PLAN
(WEST SIDE)

AREA OPEN FOR ACCESS
AREA OF COMPLETED CONSTRUCTION

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

CONCEPT DRAWING
UNDERPINNING AUXILIARY BUILDING
CONSTRUCTION AFTER 19 WEEKS

APPENDIX C FIGURE 4



GENERAL PLAN
(WEST SIDE)

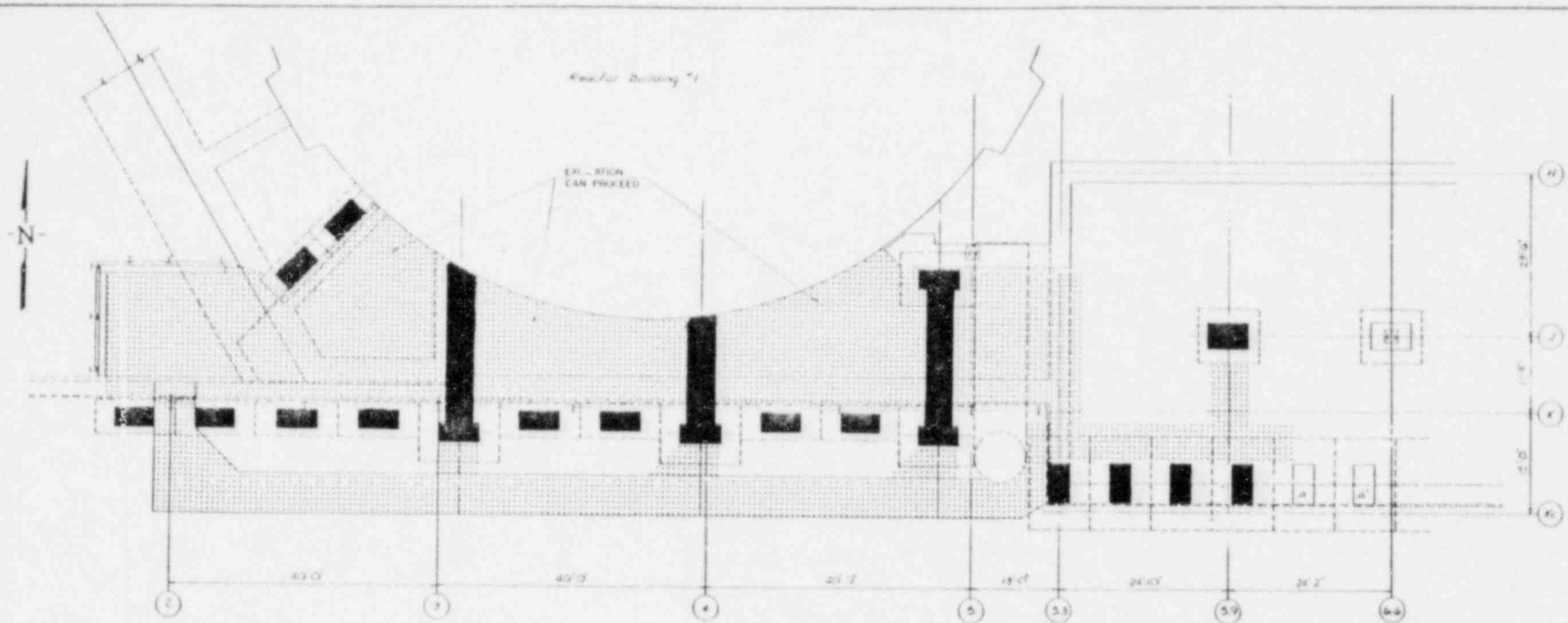
AREA OPEN FOR ACCESS
AREA OF COMPLETED CONSTRUCTION

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

CONCEPT DRAWING
UNDERPINNING AUXILIARY BUILDING

CONSTRUCTION AFTER 22 WEEKS

APPENDIX C FIGURE 5



GENERAL PLAN
(WEST SIDE)

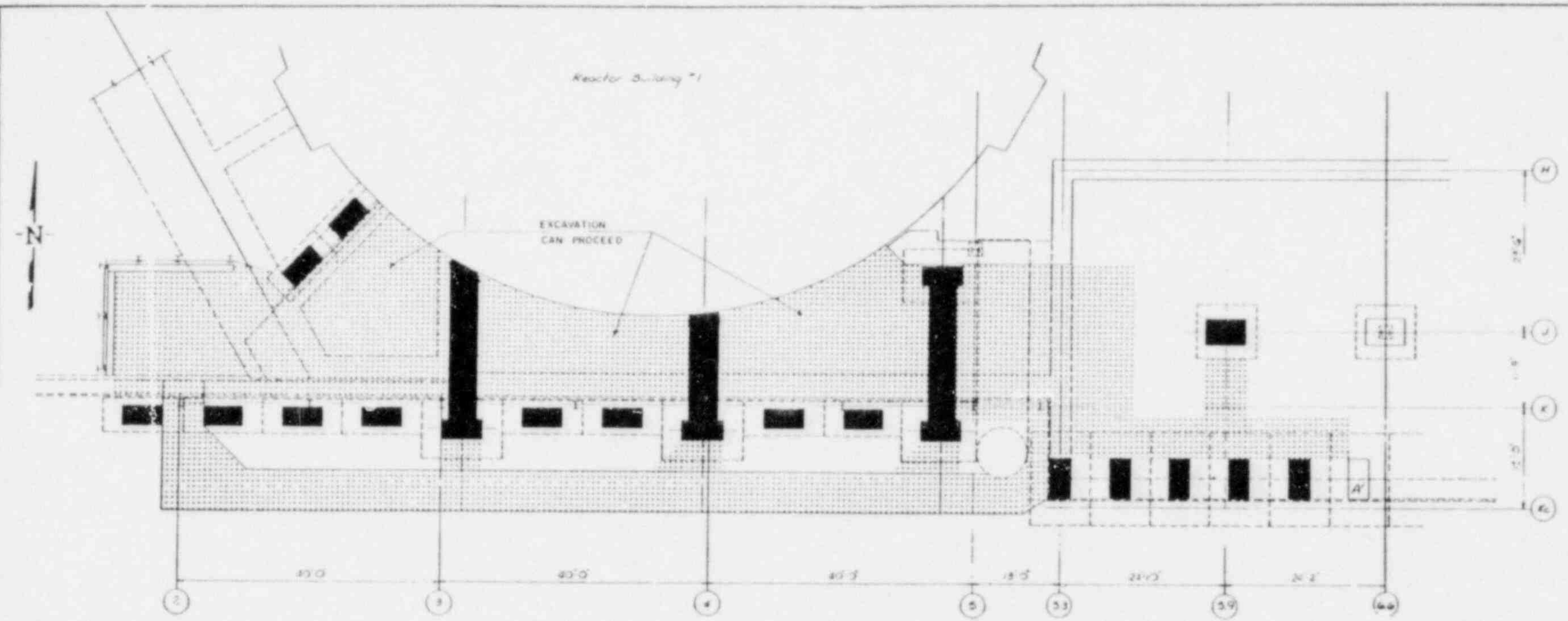
AREA OPEN FOR ACCESS
AREA OF COMPLETED CONSTRUCTION

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

CONCEPT DRAWING
UNDERPINNING AUXILIARY BUILDING

CONSTRUCTION AFTER 25 WEEKS

APPENDIX C FIGURE 6



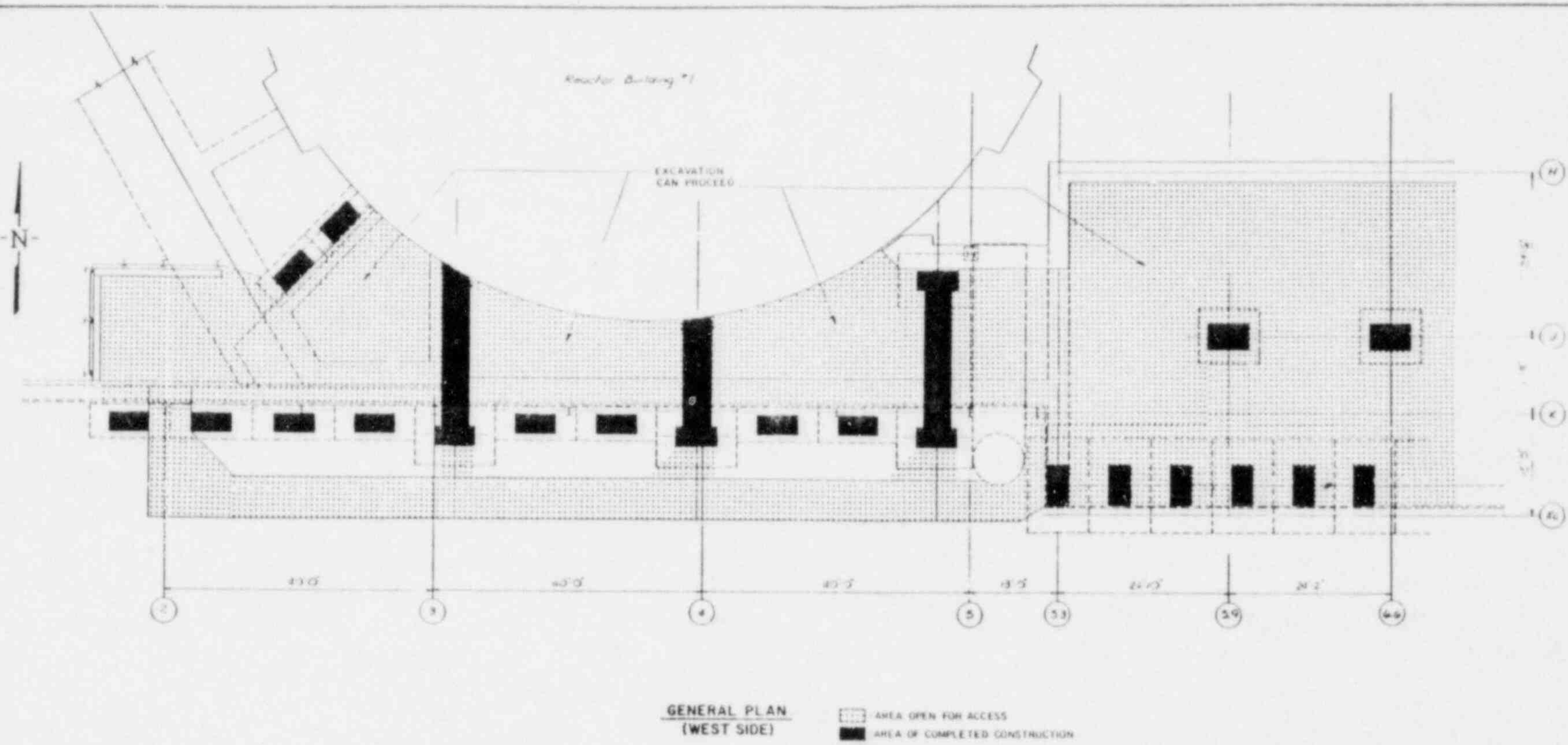
GENERAL PLAN
(WEST SIDE)

AREA OPEN FOR ACCESS
AREA OF COMPLETED CONSTRUCTION

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

CONCEPT DRAWING
UNDERPINNING AUXILIARY BUILDING
CONSTRUCTION AFTER 28 WEEKS

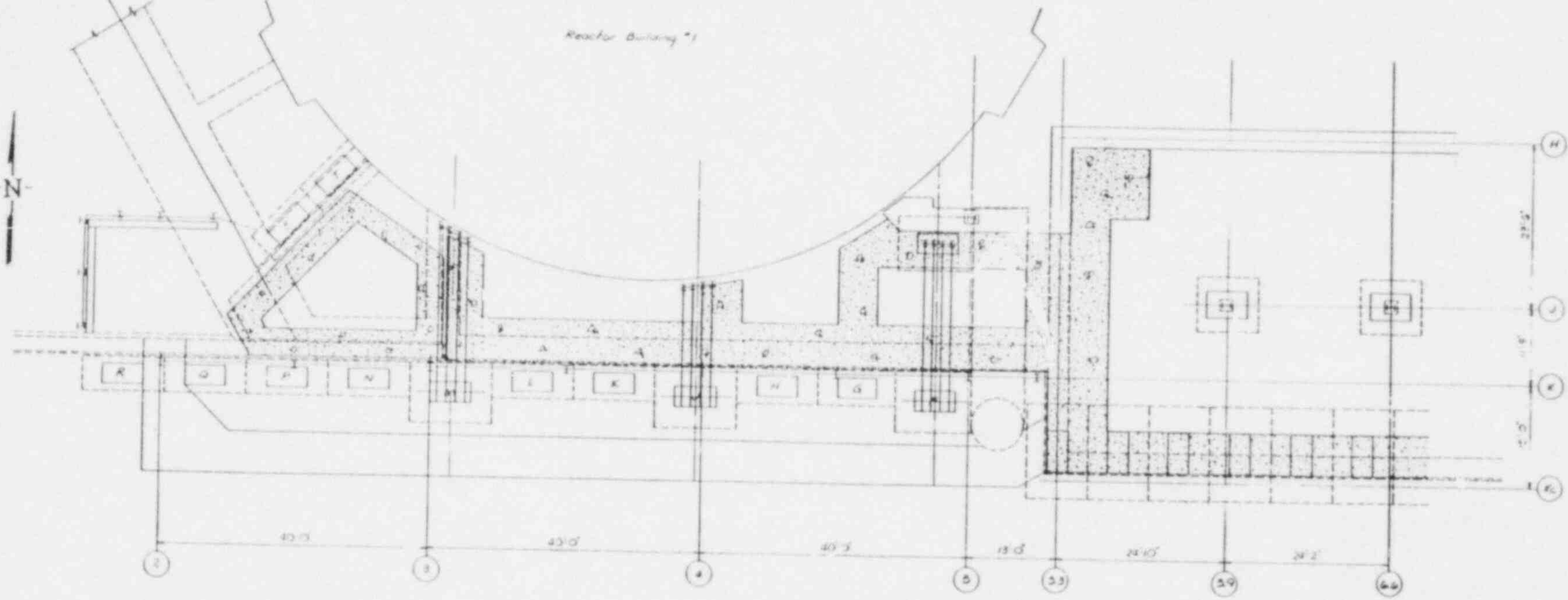
APPENDIX C FIGURE 7



CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

CONCEPT DRAWING
UNDERPINNING AUXILIARY BUILDING
CONSTRUCTION AFTER 31 WEEKS

APPENDIX C FIGURE 8



GENERAL PLAN
(WEST SIDE)

PERMANENT UNDERPINNING TO BE CONSTRUCTED

CONSUMERS POWER COMPANY
MIDLAND PLANT UNITS 1 & 2

CONCEPT DRAWING
UNDERPINNING AUXILIARY BUILDING

PERMANENT UNDERPINNING

APPENDIX C FIGURE 9

APPENDIX D

INSTRUMENTATION, LOAD TRANSFER, LOAD SENSING, AND CORRECTIVE MEASURES

1.0 INSTRUMENTATION

Underpinning the auxiliary building requires excavating approximately 9,000 cubic yards of material from underneath the structure and replacing that material with structural concrete.

The structural integrity of the auxiliary building or adjacent structures must be maintained while performing the underpinning work. To maintain the structural integrity of the structure, it is necessary to control structural stresses below predetermined levels. Allowable stresses can be correlated to allowable strains which are manifested in structure movement.

The underpinning methods that are planned require that the structure be undermined in small, discrete units and that these units be replaced with load bearing units of greater capacity than the unit that was removed. Discrete units are removed and replaced progressively, according to a predetermined plan, in a manner that will maintain the stresses in the structure below allowable limits.

Nonetheless, the existing backfill may not assume the subgrade reaction exactly according to the predetermined plan. Consequently, there is a possibility that the removal/replacement method could induce structure movement. Because of the size of any removal/replacement relative to the total structure, any single removal/replacement will not induce significant structure movement. However, the progressive nature of the work could cause an accumulated movement, which if unchecked or undetected, could lead to overstressing beyond tolerable limits.

The key to preventing intolerable stress is threefold:

a. A systematic and accurate method for detecting structure movement

b. A plan for arresting structure movement before these movements reach intolerable levels

c. A method for monitoring and assessing structure movement and load data, which results in placing a protective plan into effect. This method should constrain unwarranted concern or unnecessary remedial work.

This instrumentation plan provides the first key to preventing intolerable stress: a systematic and accurate method for detecting structure movement.

Because the structure is relatively rigid, measurable differential structure movement could produce high stress growth. Therefore, the measurement system must be highly sensitive to real movements. Similarly, measurement points must be located where they will have the highest degree of magnification proportional to real stress (i.e., the free end of the cantilever).

Thus, two systems for detecting vertical and horizontal movement have been designed to meet this objective. Both systems are based on two layers of measurements. The first layer is for detecting movement of the reactor's auxiliary building and turbine building with respect to a fixed datum or vertical plane. The second layer is for detecting relative movement of each of the structures with respect to each other. A two-layer system is required because of the precision of instrumentation hardware.

The only exception to this is that a fixed base datum (deep seated bench mark), high-precision system will be used to measure movement of the free end of the wings and control tower. The location of these measurement points is shown in Figure D-1. The precision of the instrumentation is ±0.002 inch; accuracy is ±0.005 inch.

All other vertical detection of movement points rely on a two-layer system. The precision of the instrumentation of the first layer (relative base) is ±0.005. The precision of the second layer (fixed base reference) is ±0.06 inch. Because the first-layer system depends on the second level, the absolute accuracy of the first-level system is approximately ±0.07 inch. Because of direct reading and high precision, the benefit of the second-level system is that data from the system is readily available for sensing differential movement, for developing trends, and for triggering nonroutine readings of the absolute system. The location of the relative base and absolute base reference points is shown in Figure D-1.

Relative horizontal movement will be detected at all vertical measurement locations at all three levels. Relative horizontal movement instrumentation will consist of Ames dials and mechanical micrometers.

The absolute horizontal movement detection system will be installed at three locations on top of the auxiliary building. One point will be on the top of the elevator shaft, just north of column row H along column line 6.6. The other two points will be at column lines 3 and 10, approximately 9 feet north of column row K. The reference points for these points will be the base of the elevator shaft and targets near existing ground level at the exterior walls of the auxiliary building at column lines 3 and 10. The exterior targets will be referenced to a monumented double center horizontal alignment system which projects east and west at existing ground level. The reference points at the roof level will be projected to the base reference points by plumbing. The absolute system will have a precision of ±0.1 inch.

2.0 LOAD TRANSFER AND LOAD SENSING

Section 1.0 listed three key features of a successful program to prevent distress in the structures being underpinned. The second feature of that program is compensation for structure movements and their consequent stresses. This section addresses that issue.

The structures being underpinned are not continually supported on hydraulically actuated jacks. Therefore, in practice, hydraulic pressure data (translated into force) cannot be used for measuring load over a period of time. Hydraulic pressure (translated into force) is used only for transferring load from the structure to the underpinning. The amount of hydraulic force is based on the calculated amount of support the underpinning element must provide at the time of the load transfer. Hydraulic pressure is used because the load transfer is dynamic and requires elongating the jack until equilibrium is established between the structure and the underpinning element.

Once the dynamic aspect of load transfer is completed, a rigid structural element is inserted between the structure and the underpinning element. The rigid element is closed by driving matched pairs of steel wedges. However, prior to driving the wedges and deactivating the jacks, it is necessary to ensure that short-term, time-dependent effects are not occurring. Therefore, the jack/hydraulic system is maintained for 1 hour. If the relative position of the structure with respect to the underpinning element changes by more than 0.01 inch, or the hydraulic pressure dissipates by more than 10%, the process is repeated until the aforementioned criteria are satisfied.

As the underpinning work progresses, the load previously transferred to the underpinning elements will most likely change.

Changes in loads on either the newly installed underpinning or the existing and remaining subgrade can cause total or differential settlement of the structure. Differential settlements induce stresses in the structure. These stresses may or may not be significant, depending on their magnitude, algebraic sign, and location.

The causes of load change that are easiest to deal with are related to the newly installed underpinning, because there is access to the point of applied load. This problem will be addressed later in this discussion.

The more difficult cause relates to the existing and remaining subgrade reaction, because the subgrade is not accessible.

The transfer of structure loads to and from this area is accomplished from remote locations. The exercise of remote load control primarily depends on three construction procedures:

a. The installation sequence, location, and capacity of the underpinning

b. The amount of calendar time consumed performing the sequential work

c. The method and rate of dewatering

Any or all of these construction procedures can b_ varied in the planning stage or during construction. In both the planning stage and construction, these procedures can be modeled so critical structural stresses can be determined and an underpinning construction procedure can be developed which prevents intolera)le structural stresses.

The construction plan for the underpinning work is to initially install the maximum allowable support at the end of the wings and transform the wing structure to a propped cantilever with a fixed end at the control tower. This procedure immediately eliminates the need for remote load sensing in the wing areas and the variability in modeling the least known subgrade reaction. It also significantly reduces the area in which remote load sensing is required, while simultaneously simplifying the structure modeling for determining critical stresses.

The sensing of load in the control tower area, while and after the propped cantilever configuration is working, is based on precise measurements of control tower displacements (movements). Deep seated bench marks at column line/row H x 7.2 and K x 7.2 in combination with a deep plumb line at H x 7.2 will be capable of measuring increments of structure movement well below critical strain levels.

By sensing loads at the underpinning support points at the ends of the wings and structure displacements at the control tower, fewer variables will be used to develop the structural model.

If the model data indicate that the boundary conditions and resultant critical stresses for the second stage of underpinning are close to the preplanned conditions, underpinning will proceed as planned. If not, it will be necessary to modify or change the underpinning plan.

Load sensing at the location of the underpinning will be measured by concrete stress meters installed in each underpinning element and verified by hydraulic jack lift-off tests as necessary. Relative movement of the underpinning element base relative to the structure will also be measured by tell tale devices.

Predetermined points which provide the greatest allowable vertical movement (the free ends of the wing and control tower cantilevers) will be measured directly by mechanical devices (Ames dials and mechanical micrometers) attached to deep seated bench marks. All other data for detecting vertical movement will be obtained.

3.0 CORRECTIVE MEASURES

Section 1.0 outlined a three-fold program for preventing intolerable stresses. This section address the third step of the program: A method for monitoring and assessing structure movement and load data, which results in placing a protective plan into effect.

A. Frequency of Monitoring

1. Detection of Movement - Routine

a) High-precision system, cantilever free ends and relative base system

1) Read and plot each point once every 8 hours

2) Evaluation of data by onsite geotechnical engineer once daily

b. Fixed datum reference points

1) Read and plot each point once every week

2) Evaluation of data by onsite geotechnical engineer once every week

3) Evaluation by engineering once every month

c) Underpinning load data

1) Read and plot each point once every 8 hours

2) Evaluation of data by onsite geotechnical engineer once daily

2. Horizontal Detection of Movement - Routine

a) High-precision system

1) Read and plot each point once a week

2) Evaluation of data by onsite geotechnical engineer once each week

b) Fixed base system

1) Read and plot each point once a month

2) Evaluation of data by onsite geotechnical engineer once each month

3. Vertical Detection of Movement - Nonroutine

If evaluation of either the high-precision, fixed datum reference, or underpinning load data reveals movements that exceed predetermined amounts for a particular stage of underpinning, the fixed datum reference points which reflect activity shall be read once every 24 hours and the high-precision points shall be read 9 times a day.

B. Plan of Action

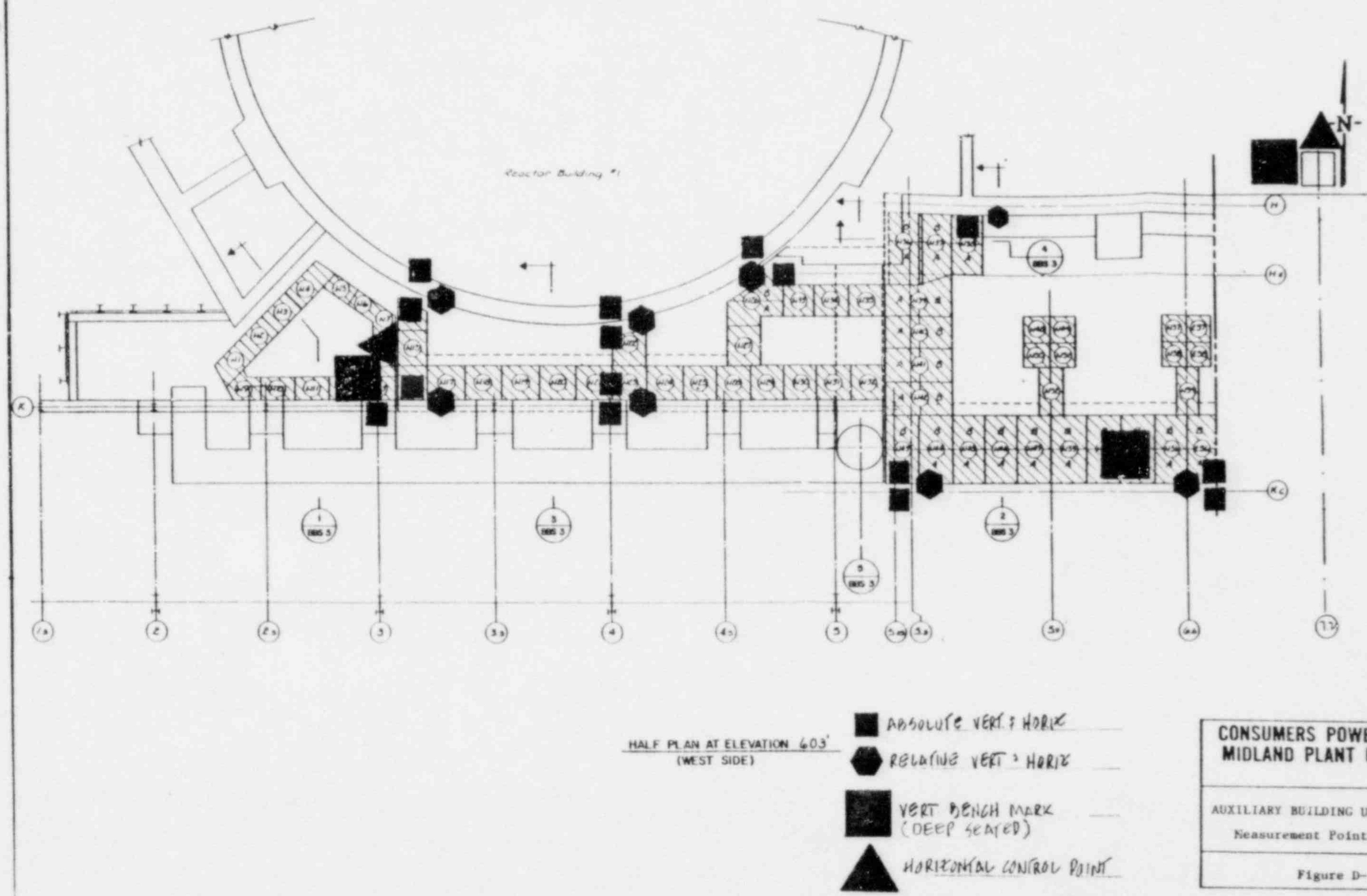
1. If the evaluation of the underpinning load data and the high-precision vertical movement system indicates trends which would lead to an intolerable level, the onsite geotechnical engineer will institute nonroutine procedures cited in Section A.3 above and with the subcontractor will make adjustments to the underpinning construction plan to arrest the movements.

2. If the evaluation of the fixed datum reference data gathered under Section A.1.b or B.1 eveals movements in excess of a predetermined amount, the onsite geotechnical engineer with the subcontractor will make adjustments to the underpinning construction plan to arrest the movements and report the findings, data, and corrective actions to engineering. Engineering shall evaluate the

information furnished by the onsite geotechnical engineer and determine if other changes in the underpinning construction plan are required, and if construction must be partially or totally stopped.

Attached Figures:

D-1 Auxiliary Building Underpinning - Measurement Points Location



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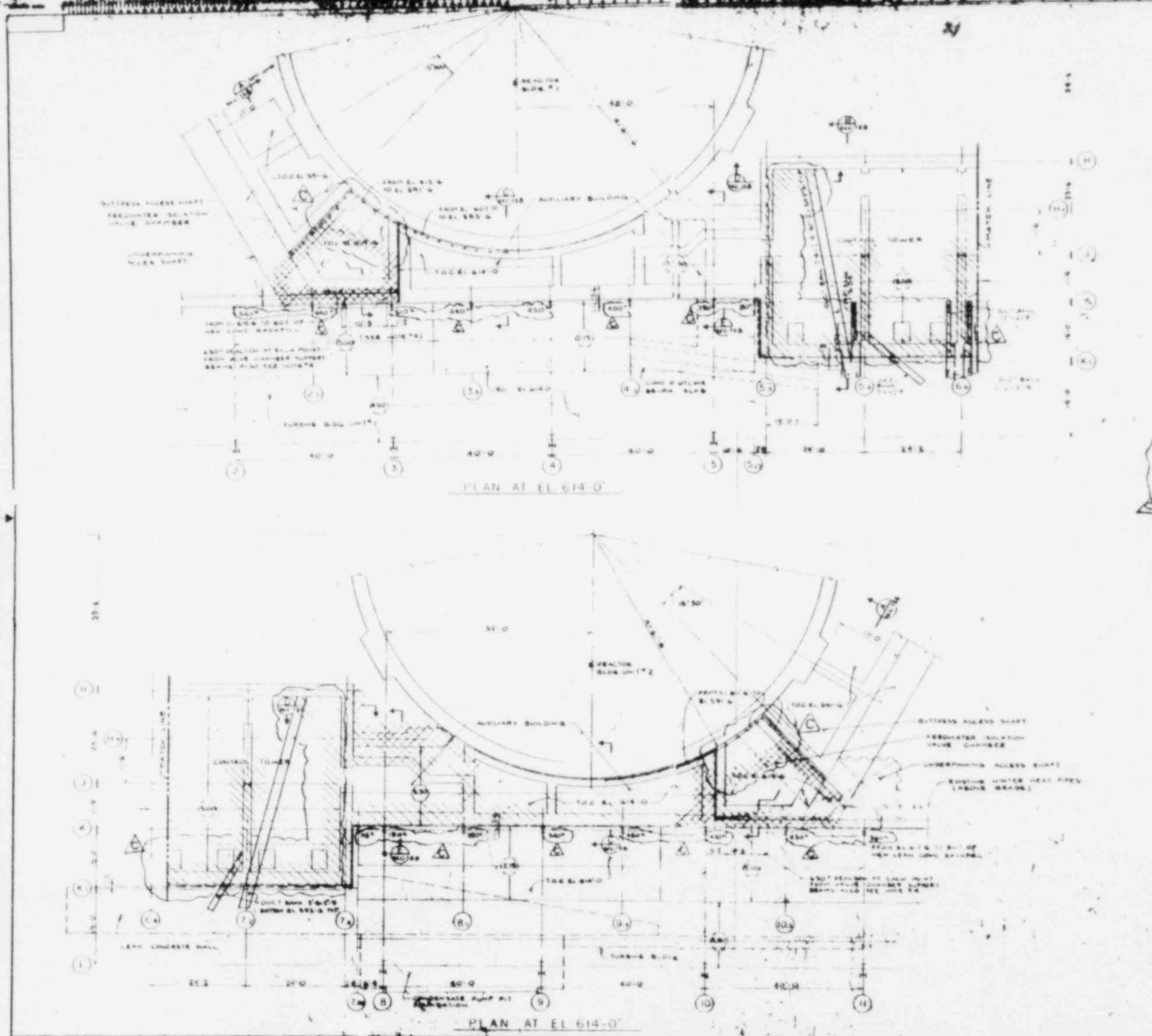
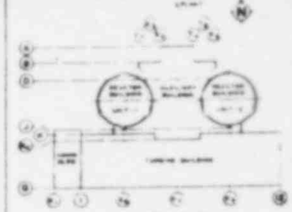
MIDLAND PLANT UNITS 1 & 2

AUXILIARY BUILDING UNDERPINNING
Measurement Points Location

Figure D-1





KEY PLAN

NOTES

1. THIS DRAWING WILL BE USED FOR DEVELOPING A CONSTRUCTION PROCEDURE FOR EXCAVATION & UNDERPINNING WORK
2. THE EXTENT AND THICKNESS OF CONCRETE BACKFILL SHOWN ARE APPROXIMATIONS BASED ON SOIL BORING INFORMATION AND CONSTRUCTION RECORDS
3. UNDERPINNING SUBCONTRACTOR TO PROVIDE ADEQUATE SUPPORT AND BRACING FOR MUDMAT, CONCRETE BACKFILL TO PREVENT ANY DAMAGE, SAGGING OR SEPARATION OF MUDMAT AND WATERPROOF MEMBRANE
4. A REDUCED PRESSURE OF 2.15 KSF MAY BE USED FOR THIS ZONE IF THE FEEDWATER ISOLATION VALVE PITS ARE SUPPORTED INDEPENDENTLY BY THE SUBCONTRACTOR
5. TURBINE BUILDING LOADS ARE TO BE SUPPORTED SEPARATELY
6. FOR OTHER NOTES AS APPLICABLE SEE DRAWING C-2002
7. FOR REFERENCE DRAWINGS SEE DWG. SKC 187
8. FOR DUCT BANK SIZE AND LOCATION SEE DWGS. E 537 SH 1, E 545 SH 1, E 547 SH 1
9. TURBINE BUILDING COLUMNS WILL BE SUPPORTED SEPARATELY FROM AUXILIARY BUILDING UNDERPINNING
10. TEMPORARY TUNNEL MAY BE EXCAVATED UNDERNEATH TURBINE BUILDING FOR ACCESS DURING AUXILIARY BUILDING UNDERPINNING

Q-LIST ITEMS

UNDERPINNING, EXCAVATION AND PLACING OF CONCRETE

LEGEND

PRESSURE IN (KSF) ON SOIL DUE TO DEAD LOAD FROM ABOVE DOES NOT INCLUDE PRESSURE DUE TO HEIGHT OF MUDMAT OR CONCRETE BACKFILL

PRESSURE BOUNDARY LINES

COLUMN LOADS (DL+LL) AT BASE OF COLUMN

AREA WHERE STYROFOAM OR EQUIVALENT TO BE PLACED BETWEEN NEW CONCRETE BACKFILL AND EXISTING STRUCTURES FROM ELEVATIONS AS NOTED

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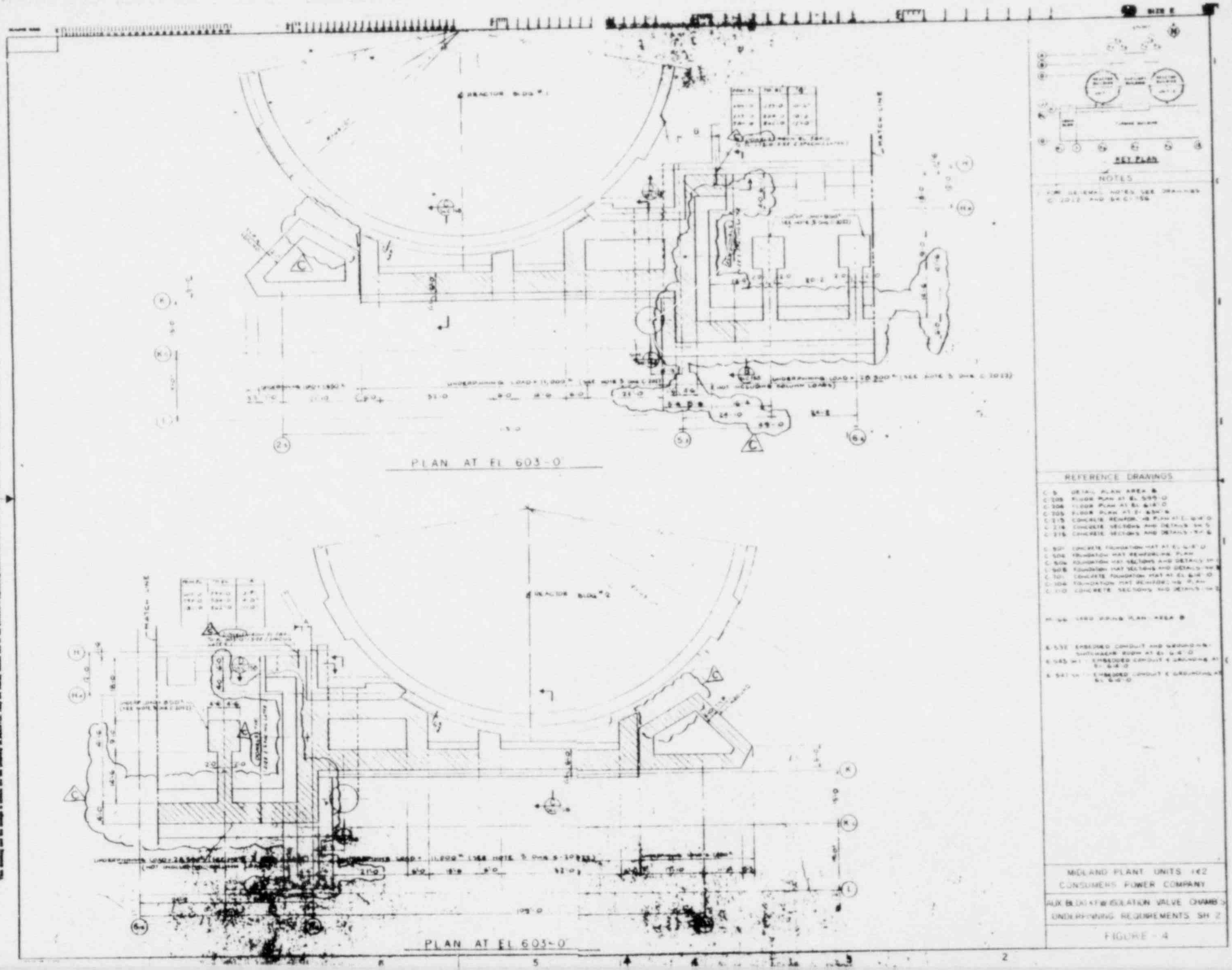
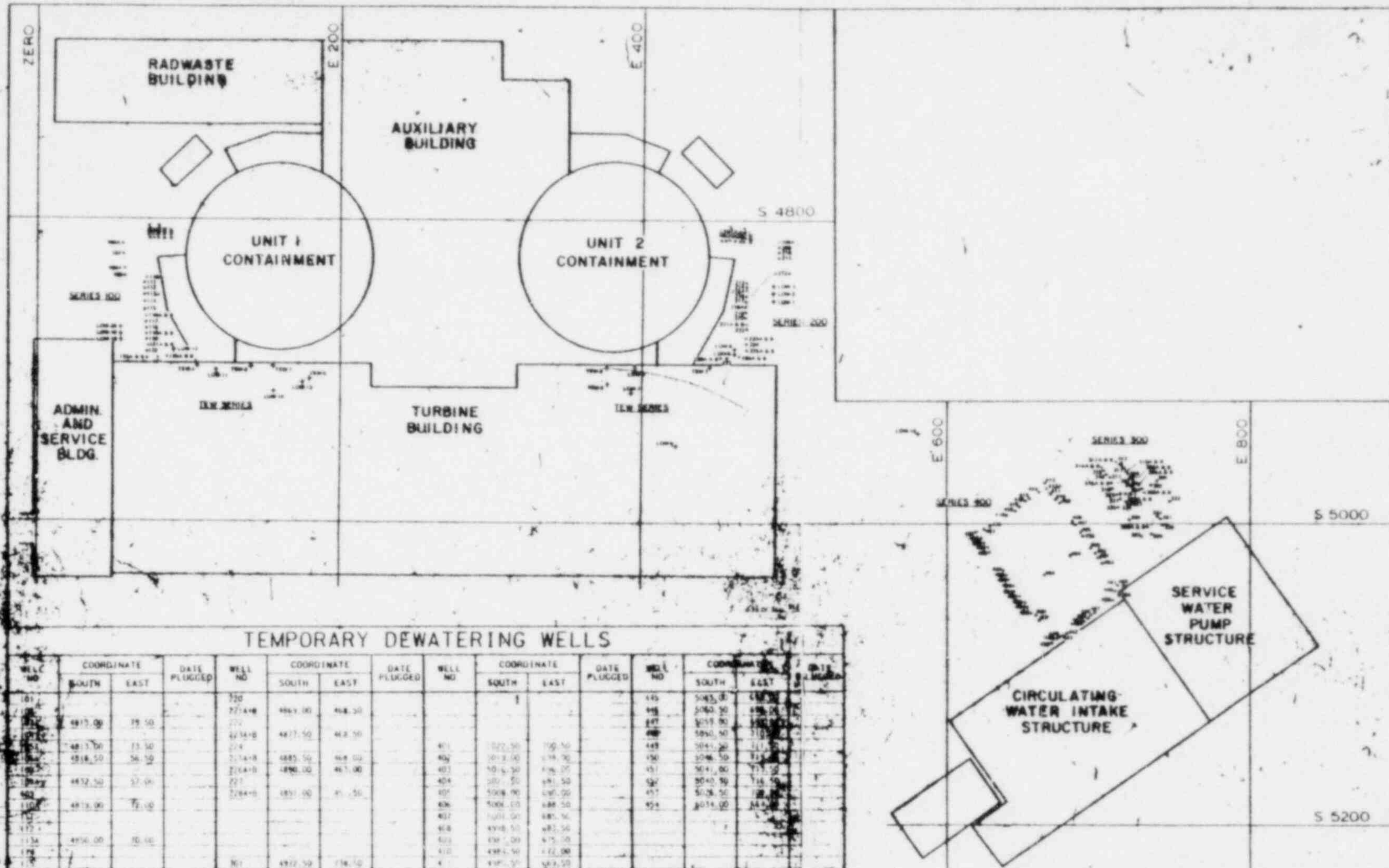
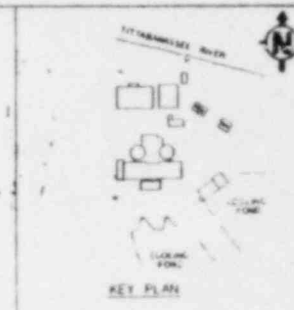
AUX BLDG & FW ISOLATION VALVE CHAMB
UNDERPINNING REQUIREMENTS SH. 1

FIGURE - 7


KEY PLAN
NOTES
FOR GENERAL NOTES SEE DRAWINGS C-1012 AND SK-C-156
REFERENCE DRAWINGS
MATCH LINE
REACTOR BLDG #1
REACTOR BLDG #2
PLAN AT EL 603-0"
PLAN AT EL 603-0"
MIDLAND PLANT UNITS 1&2
CONSUMERS POWER COMPANY
AUX BLDG & FW ISOLATION VALVE CHAMB'S
UNDERPINNING REQUIREMENTS SH 2
FIGURE - 4





NOTES

TEMPORARY DEWATERING WELLS

WELL NO	COORDINATE SOUTH	COORDINATE EAST	DATE PLUGGED
101			
[illegible]			
[illegible]	4913.00	79.50	
[illegible]			
[illegible]	4813.00	73.50	
[illegible]	4818.50	56.50	
[illegible]			
[illegible]	4832.50	57.00	
[illegible]			
[illegible]	4813.00	72.00	
[illegible]			
112			
113A	4956.00	70.00	
[illegible]			
[illegible]			
115A+B	4864.50	71.50	
117			
119A+B	4876.00	71.00	
120			
121A+B	4884.00	72.50	
122			
123A+B	481[illegible]	72.00	
124			
125A+B	4891.00	82.00	
201			
202			
[illegible]	481[illegible].00	457.50	
204			
[illegible]			
[illegible]	[illegible]	468.00	
207			
[illegible]	4817.50	469.[illegible]	
[illegible]			
210			
211			
[illegible]	4837.50	45[illegible]	
213			
214			
215A	[illegible]	453.50	
216			
217			
218A	[illegible]	457.50	
219			

WELL NO	COORDINATE SOUTH	COORDINATE EAST	DATE PLUGGED
220			
221A+B	4969.00	468.50	
222			
223A+B	4877.50	462.50	
224			
225A+B	4885.50	468.00	
226A+B	4890.00	467.00	
227			
228A+B	4851.00	4[illegible].50	
301	4972.50	736.50	
302	4975.00	735.50	
303A+B	4980.00	731.00	
304	4977.50	729.50	
305A+B	4975.50	728.00	
306	4974.00	725.50	
307	4972.00	723.50	
308A+B	4970.00	721.50	
309	4968.50	720.00	
310A+B	4966.00	718.00	
311	4964.00	717.50	
312B	4962.50	716.20	
313	4961.00	715.00	
314A+B	4963.50	703.50	
315	4965.50	704.50	
316	4968.00	706.00	
317	4970.00	708.00	
318A+B	4973.50	709.50	
319	4977.00	710.00	
320	4981.00	713.00	
321	4987.00	722.00	
322A+B	4989.50	714.50	
323	4984.00	715.00	
[illegible]	4981.00	719.50	
325	4985.50	718.50	
326	4997.50	720.50	
327	5009.00	728.00	
328A+B	5012.50	732.50	
329	5001.00	735.00	
330	5061.00	742.50	
331	4997.50	741.50	
332	4994.50	739.50	
333	4995.50	747.00	

WELL NO	COORDINATE SOUTH	COORDINATE EAST	DATE PLUGGED
401	5022.50	709.50	
402	5013.00	69[illegible]	
403	5016.50	696.20	
404	5017.50	691.50	
405	5008.00	690.00	
406	5006.00	688.50	
407	5001.00	685.50	
408	4998.50	683.50	
409	4997.00	675.00	
410	4986.50	672.00	
411	4997.50	669.50	
412	4977.50	667.50	
413	4973.50	666.00	
414	4985.00	661.00	
415	4984.00	654.50	
416	4985.50	652.00	
417	4989.50	648.00	
418	4995.50	645.50	
419	4995.00	635.00	
420	5001.00	678.00	
421	5004.00	626.50	
422	5006.00	619.00	
423	5010.00	622.50	
424	5013.00	624.00	
425	5015.50	627.00	
426	5019.00	628.00	
427	5031.00	637.00	
428	5034.00	639.00	
429	5037.00	641.00	
430	5041.50	643.50	
431	5045.00	644.50	
432	5048.00	648.00	
433	5052.00	650.50	
434	5055.00	653.00	
435	5058.50	655.00	
436	5061.00	656.50	
437	5064.50	660.50	
438	5072.50	659.50	
439	5077.00	672.00	
440	5076.00	674.50	
441	5074.50	676.50	
442	5072.50	678.50	
443	5066.00	680.50	
444	5061.00	[illegible]	

WELL NO	COORDINATE SOUTH	COORDINATE EAST	DATE PLUGGED
445	5063.00	[illegible]	
446	5060.50	[illegible]	
447	5055.00	[illegible]	
448	5050.50	[illegible]	
449	5041.50	[illegible]	
450	5046.50	[illegible]	
451	5041.00	713.50	
452	5040.50	716.50	
453	5026.50	[illegible]	
454	5014.00	[illegible]	

MIDLAND PLANT UNITS 1&2
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LOCATION OF TEMPORARY DEWATERING WELLS

FIGURE - 6

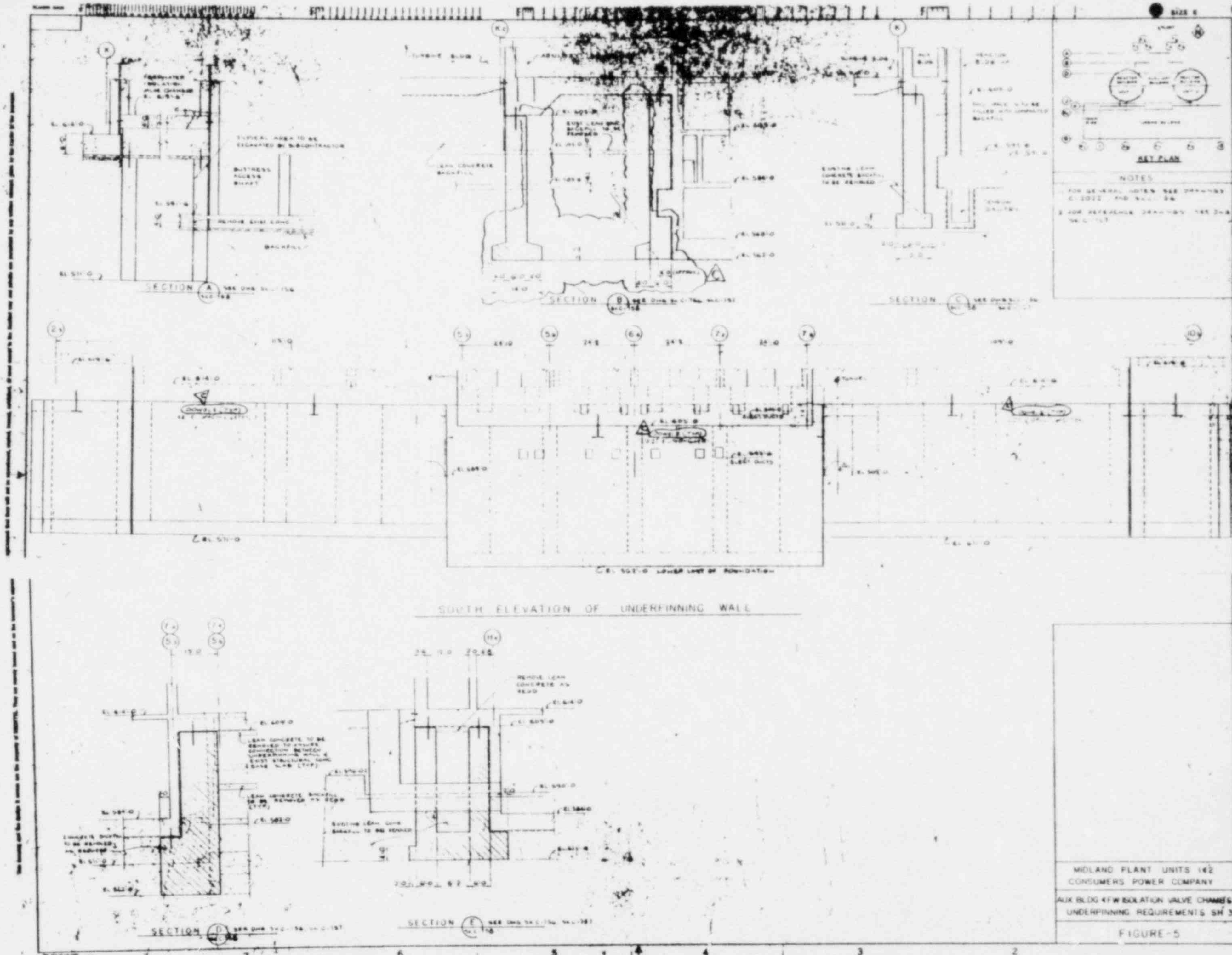
SECTION A
SECTION B
SECTION C
KEY PLAN
NOTES
SOUTH ELEVATION OF UNDERFINNING WALL
SECTION D
SECTION E
MIDLAND PLANT UNITS 1&2
CONSUMERS POWER COMPANY
AUX BLDG 4 FW ISOLATION VALVE CHAMBS
UNDERPINNING REQUIREMENTS SH 3

FIGURE-5