NUREG-0793
Supplement No. 2

Safety Evaluation Report

related to the operation of Midland Plant, Units 1 and 2

Docket Nos. 50-329 and 50-330

Consumers Power Company

U.S. Nuclear Regulatory Commission

Office of Nuclear Reactor Regulation

October 1982

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This report supplements the Safety Evaluation Report, NUREG-0793, issued May 1982 by the Office of Nuclear Reactor Regulation of the U.S. Nuclear Regulatory Commission with respect to the application filed by Consumers Power Company, as applicant and owner, for licenses to operate the Midland Plant, Units 1 and 2 (Docket Nos. 50-329 and 50-330). The facility is located in the city of Midland in Midland County, Michigan. This supplement provides recent information regarding resolution of some of the open items identified in the Safety Evaluation Report. Most of the open items are associated with soils-related problems at the Midland site.

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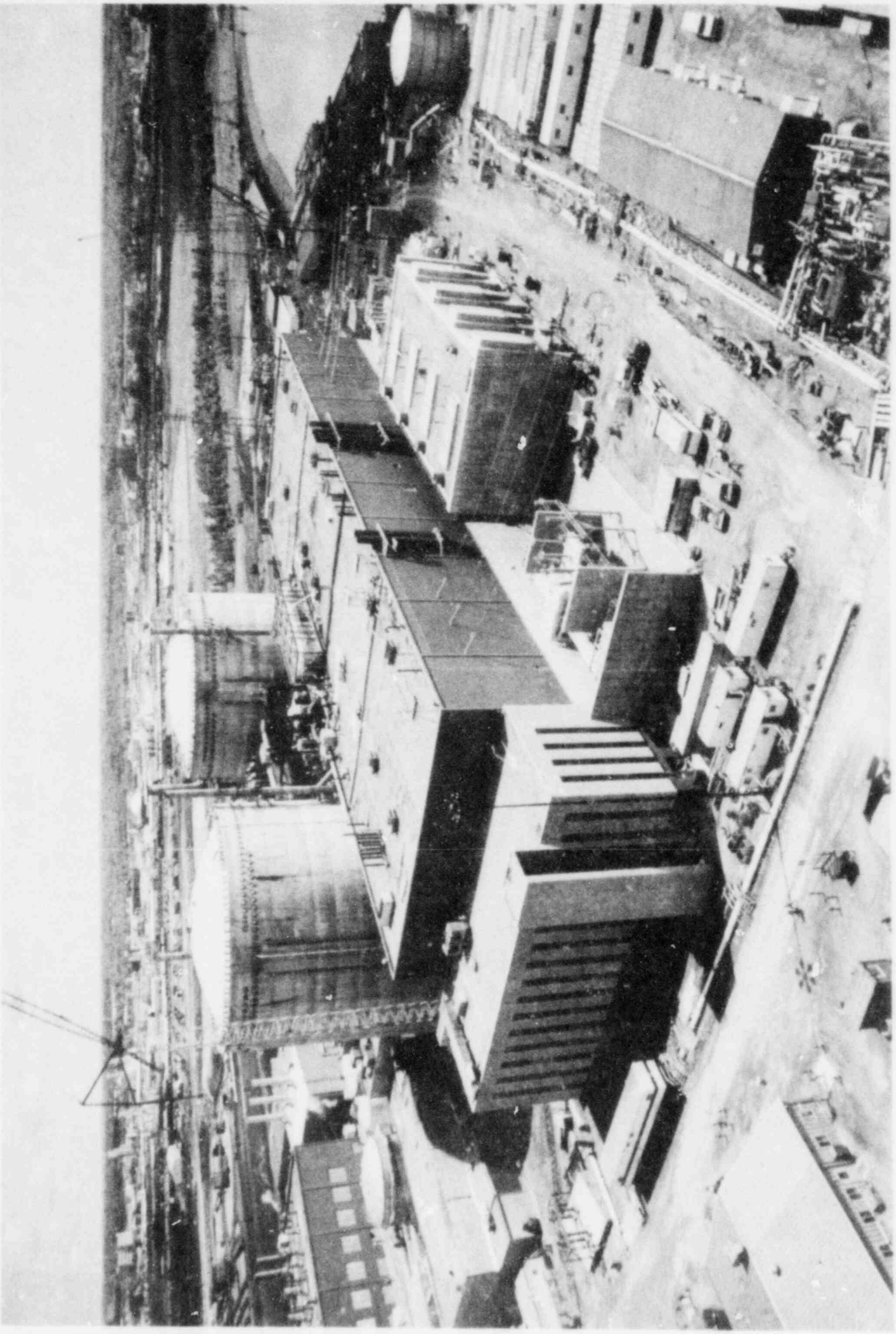
Office o[illegible]clear Reactor Regulation

October 1982

ABSTRACT

This report supplements the "Safety Evaluation Report related to the Operation of Midland Plant, Units 1 and 2" (SER) (NUREG-0793) issued in May 1982 by the Office of Nuclear Reactor Regulation of the U.S. Nuclear Regulatory Commission with respect to the application filed by Consumers Power Company, as applicant and owner, for licenses to operate the Midland Plant, Units 1 and 2 (Docket Nos. 50-329 and 50-330). The facility is located in the city of Midland in Midland County, Michigan. This supplement provides recent information regarding resolution of the soils settlement issue, one of the open items identified in the SER. Certain confirmatory issues identified in the SER also are addressed.



Midland Nuclear Power Plant
Midland, MI — August 1982

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1 INTRODUCTION AND GENERAL DISCUSSION

1.1 Introduction

On May 11, 1982, the Nuclear Regulatory Commission staff (NRC staff, the staff) issued a Safety Evaluation Report (SER) (NUREG-0793) regarding the application by Consumers Power Company (the applicant) for licenses to operate the Midland Plant, Units 1 and 2. The first supplement to that SER (SSER) was issued on June 30, 1982.

This second supplement provides more recent information regarding resolution of some of the open items identified in the SER, primarily those related to soils settlement.

The sections and appendices of this supplement are designated by the same numerals or letter as the section or appendix of the SER that is being updated, and the discussions are supplementary to and not in lieu of the discussion in the SER unless otherwise noted. Accordingly, Appendix A is a continuation of the chronology of the safety review. Appendix B is an updated bibliography.* Appendix D is a list of abbreviations used in this supplement. Appendix E is a list of principal contributors to this supplement. Appendix H is a list of errata for the SER. There is a new appendix, Appendix I, which describes the underpinning construction support program and contains portions of some preliminary drawings used in this review. No changes in SER Appendices C, F, or G have been made by this supplement.

The Project Manager is Mr. Darl S. Hood; he may be contacted by calling (301) 492-8474 or by writing to the Division of Licensing, U.S. Nuclear Regulatory Commission, Washington, DC 20555. Mr. Ronald W. Hernan also serves as Project Manager and may be reached on (301) 492-8395.

1.7 Summary of Outstanding Items

Section 1.7 of the SER noted several outstanding open items for which staff review must be completed prior to a decision on issuance of operating licenses. This supplement addresses one of these open items, item 5, which applies to several sections of the SER that discussed the soils settlement issue at the Midland Plant. The SER sections that identified soils-related open items and their disposition in this supplement are presented below.

SER Section 2.4.4, Flood Protection Requirements

This supplement does not address the Technical Specification that is required to define the conditions under which watertight doors will be

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*Availability of all material cited is described on the inside front cover of this report.

closed or watertight barriers will be put in place when needed for flood protection. This matter will be closed during the staff's detailed review of the Midland Technical Specifications prior to plant operation.

- SER Section 2.4.6, Groundwater

  Sections 2.4.6.2 and 2.4.6.3 of this SSER discuss the staff's further review and acceptance of the dewatering system with respect to increased rates of groundwater rise at the railroad bay area (RBA), and with respect to additional pipe breaks. Section 2.4.6.4 of this SSER discusses the specific site areas to be dewatered and the wells to be used for this purpose. Section 2.4.6.4 of this SSER also indicates that the basis for establishing a suitable Technical Specification for dewatering is provided for by the design and test results.

  The actual Technical Specification for permanent dewatering will be reviewed during the staff's detailed review of the Midland Technical Specifications prior to plant operation. The open items from SER Section 2.4.6 are closed.

- SER Section 2.5.2, Seismology

  Further discussion of the SHAKE calculations is presented in Section 2.5.4.5.6 of this SSER. This portion of SER open item 5 is closed.

- SER Section 2.5.4, Stability of Subsurface Materials and Slopes

  The staff's evaluation and acceptance of the stability of subsurface materials and foundations for seismic Category I structures and components is presented in Section 2.5.4 of this SSER. This section also discusses acceptable resolution of design details to ensure foundation stability and the implementation of adequate construction controls to safely complete remedial work for the various structures and components affected by inadequate soils compaction. Staff conclusions regarding the diesel generator building (DGB) foundation are subject to the results of a structural analysis being performed by an NRC consultant using a settlement profile of actual measured settlement releases. Certain information audited by the staff remains to be docketed and certain Technical Specifications remain to be developed at the appropriate stage of the staff's detailed review prior to plant operation; these previously identified portions of SER open item 5 are otherwise closed. However, a new open item results from a modification to the originally proposed freezewall design at intersections with piping and conduit (Section 2.5.4.4.5 of this SSER).

- SER Section 3.7.1, Seismic Input

  The applicant is currently evaluating all existing seismic Category I structures necessary for safe shutdown and continued heat removal to determine the seismic safety margins resulting from application of the site-specific response spectra. This portion of SER open item 5 is reclassified as a confirmatory issue for the reasons discussed in Section 3.7.1 of this SSER.

- SER Section 3.7.2, Seismic Analysis

  Section 3.7.2 of this SSER discusses the staff's finding that the applicant's use of 1.5 times the FSAR seismic response spectra in lieu of the site-specific response spectra for designing new structures (foundation rings for the borated water storage tanks (BWSTs), underpinning for the auxiliary building and service water pump structure (SWPS)) is conservative. The applicant has also shown that the three peaks in floor response spectra resulting from a variation of ± 30% of the soil stiffness would be enveloped by the design spectra. Acceptable results for potential structure-to-structure interaction are also indicated. All portions of SER open item 5 identified in SER Section 3.7.2 are closed.

- SER Section 3.7.3, Seismic Subsystem Analysis

  Completion of the staff's review of the seismic analyses of buried piping is discussed in Sections 2.5.4 and 3.9.3.1 of this SSER; this portion of the several open items within SER Section 3.7.3 is closed, subject to the conditions noted below for SER Section 3.9.3. The other open items in SER Section 3.7.3, which regard seismic analyses for nuclear steam supply system (NSSS) piping and for the NSSS subsystem, are not addressed in this SSER and remain open.

- SER Section 3.8.1, Concrete Containment

  This portion of SER open item 5 relates to the evaluation of the containment structure for the new site-specific seismic response spectra. The containment structure (which is located on natural soils) is not addressed in this SSER, but is included in the above discussion of open item 5 with respect to SER Section 3.7.1.

- SER Section 3.8.2, Concrete and Structural Steel Internal Structures Inside Containment

  The open item regarding the evaluation of masonry walls is closed, as discussed in Section 3.8.5 of this SSER.

- SER Sections 3.8.3 and 3.8.4, Other Seismic Category I Structures and Foundations

  Section 3.8.3 of this SSER presents staff acceptance of the criteria used in the analysis, design, and construction of structures affected by inadequate soils compaction. The open item on cracks is closed, as indicated in Section 3.8.3.5 of this SSER. Evaluation of seismic safety margins is a confirmatory issue, as discussed above for SER Section 3.7.1. All other portions of open item 5 identified in SER Sections 3.8.3 and 3.8.4 are closed, subject to a suitable Technical Specification review and docketing of as-built and short-term monitoring results at the appropriate stage (Section 3.8.3.7 of this SSER).

SER Section 3.9.3, ASME Code Class 1, 2 and 3 Components, Component Supports, and Core-Support Structures

Section 3.9.3.1 of this SSER presents staff acceptance of remedial actions for underground piping. This item is closed, subject to (1) confirmatory reporting of as-installed conditions of rebedded or replaced piping at the appropriate construction stage, and (2) suitable Technical Specification review prior to plant operation.

1.8 Confirmatory Issues

Section 1.8 of the SER noted that certain confirmatory information had not yet been provided by the applicant for several identified items. This supplement updates some of those items for which the confirmatory information has subsequently been provided by the applicant and for which review has been completed by the staff. These items, and the sections of this SSER discussing the staff review conclusions, are

(3) Program for Masonry Wall Evaluation (3.8.5)

(8) Evaluation of Environment Near DHR Suction Valves (5.4.4.2)

(13) Lowest service metal temperature (6.2.7)

In addition to the confirmatory issues in the SER, this SSER identifies confirmatory information needed by the staff for concluding its review. The sections of this SSER that identify confirmatory issues are

• FSAR Documentation on As-Built Conditions and Short-Term Monitoring	2.5.4.7, 3.8.3.7
• FSAR Documentation on Design Modifications at Freezewall Crossings	2.5.4.7
• Seismic Margin Study	3.7.2.2
• Alignment and Ovality of Rebedded or Replaced Underground Piping	3.9.3.1.6
• Reporting Requirements for Limits on Underground Piping	3.9.3.1.6

1.12 Plant Fill Deficiencies and Remedial Actions

This section of the SER identified the various seismic Category I structures and utilities affected by inadequately compacted fill soils at the Midland site, and summarized the principal topics and structures of the staff's soils-related review. This SER section also noted that the staff would report its conclusions on each topic in an SSER. This SSER provides those conclusions. The topics and structures identified by SER Section 1.12 and the sections of this SSER where the principal discussion of each appears are shown in Table 1.1 of this SSER.

Table 1.1 Cross-reference table for SER Section 1.12

SER Section and Title		Principal SSER Reference Section
1.12.1	Quality Assurance Aspects of the Soils Hearing	17.3.1
1.12.2	Seismic Changes	3.7.2
1.12.3	Cooling Pond Dikes	None (accepted by the staff in SER Section 2.4.4, except for SER Section 1.8(1), which remains open)
1.12.4	Permanent Site Dewatering	2.4.6
1.12.5	Diesel Generator Building	2.4.6, 2.5.4.4.2, 3.8.3.4
1.12.6	Auxiliary Building and Feedwater Isolation Valve Pits	2.5.4.4.1, 3.8.3.1, Appendix I
1.12.7	Service Water Pump Structure and Adjacent Cooling Pond Retaining Wall	2.5.4.4.1, 3.8.3.2, Appendix I, 3.7.2.4
1.12.8	Borated Water Storage Tanks	2.5.4.4.3, 3.8.3.3
1.12.9	Diesel Fuel Oil Storage Tanks	None (accepted by the staff in the SER)
1.12.10	Underground Pipes	3.9.3.1, 3.12

2 SITE CHARACTERISTICS

2.4 Hydrologic Engineering

2.4.6 Groundwater

2.4.6.2 Design of Dewatering System

As described in Section 2.4.6.1 of the SER and Section 2.5.4.5 of this SSER, there are areas of loose sands beneath the diesel generator building (DGB) and the railroad bay area (RBA) of the auxiliary building above el 610 ft mean sea level (MSL) that could liquefy during a safe shutdown earthquake (SSE). To minimize the potential for liquefaction, the applicant proposes to maintain the areas containing loose sands in an unsaturated condition. To do this, the applicant installed a dewatering system designed to lower and maintain groundwater levels about 15 ft below the elevation of the loose sands. A plan of the permanent watering system is shown in Figure 2.1 of this SSER.

As described in Section 2.4.6.2 of the SER, the applicant stated that the dewatering system is not seismic Category I because it is not required to operate during or after an SSE. The applicant estimated that in the unlikely event of a complete failure of the dewatering system, it would take at least 60 days before water levels below the DGB and the RBA would rise to el 610 ft MSL. This would allow sufficient time to repair and/or replace any damaged portion of the dewatering system. To verify the time before water levels reach el 610 ft MSL, the staff requested and the applicant conducted a recharge test. In conducting this test, water levels in wells surrounding the DGB were lowered to el 595 ft MSL or below, except in areas where impervious soil conditions caused perching of the water at levels slightly higher than this elevation. All dewatering well pumps were then shut off and water levels were allowed to rise normally for a period of 60 days. The staff was informed that the highest water level recorded in the vicinity of the DGB was el 608.95 ft MSL, which occurred in Well COE-10. Figure 2.2 of this SSER shows the locations of Well COE-10 and other wells monitored during the recharge test. The applicant reasoned that water levels at the RBA would rise more slowly because the RBA is further away from the source of recharge, the cooling pond. On the basis of this information, the staff agreed in the SER with the applicant's conclusion that it would take at least 60 days before water levels would rise to el 610 ft MSL beneath the DGB. The recharge test results regarding the effects on water levels in the RBA were not available when the SER was published; however, the staff agreed with the applicant's reasoning that water levels at the RBA would rise more slowly than at the DGB. As discussed below, the results of the dewatering test have shown that the groundwater rise to el 610 ft would be achieved in less than the anticipated 60 days, and water levels at the RBA actually rose at a slightly faster rate than at the DGB.

Subsequent to publication of the SER, by a letter from the applicant dated June 7, 1982, the staff received the applicant's report on the recharge test. Enclosure 1 of that letter shows that water levels in most of the wells monitored in the vicinity of the DGB during the recharge test were at elevations below 595 ft MSL

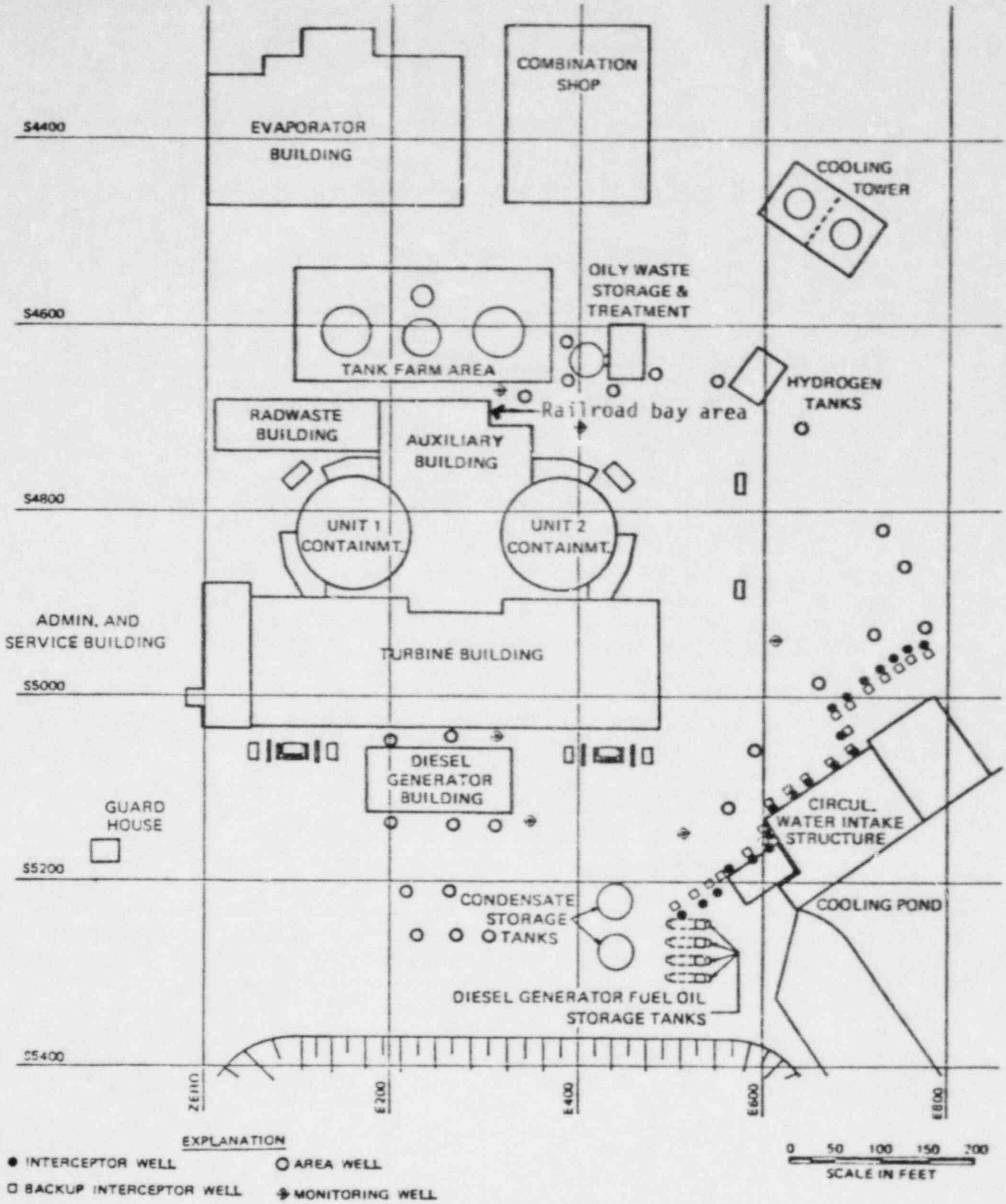


Figure 2.1 Plan of permanent dewatering system
(Source: Applicant's letter of April 19, 1982)

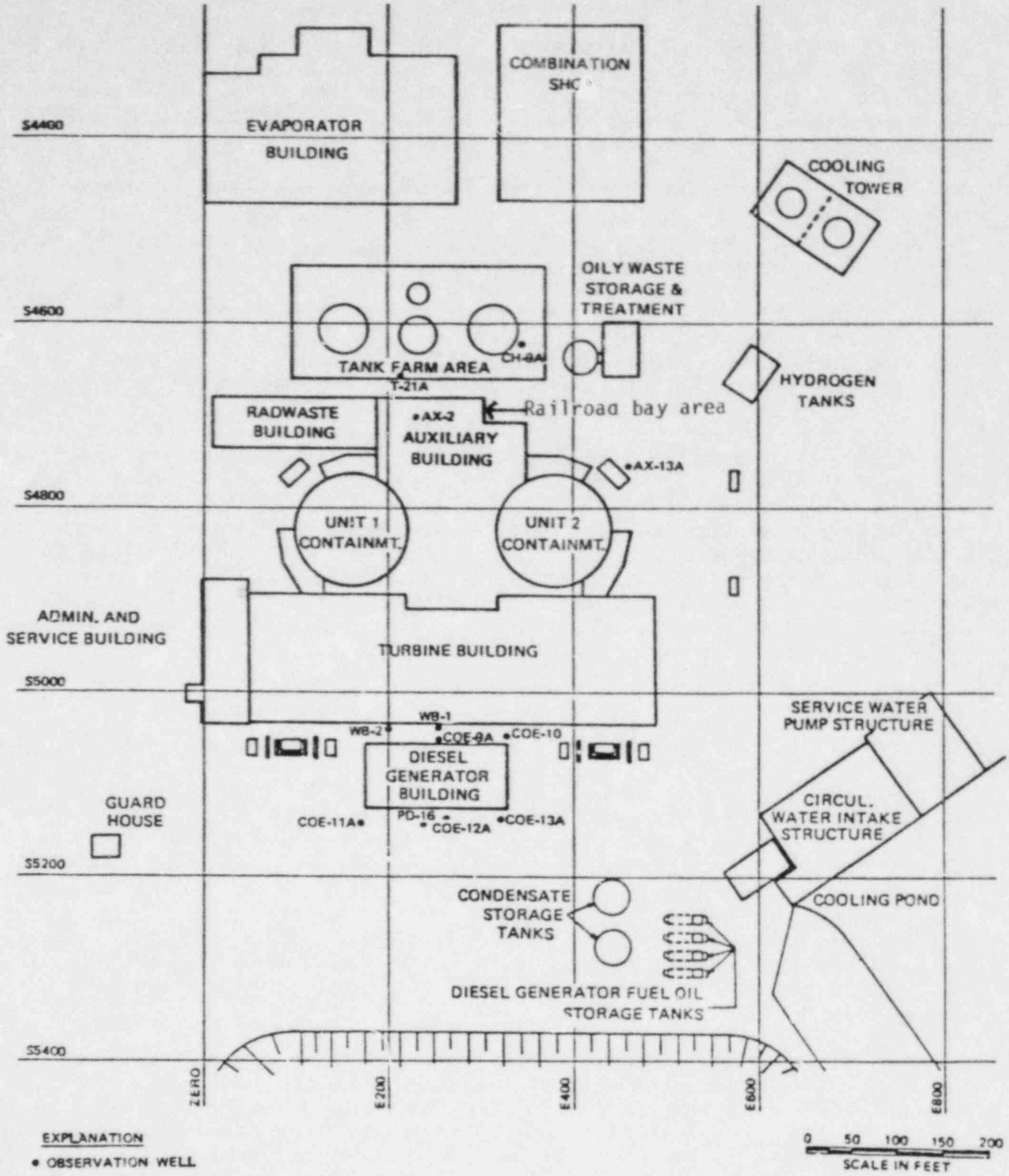


Figure 2.2 Observation wells at critical structures monitored during recharge test (Source: Applicant's letter of April 19, 1982)

when the recharge test was initiated; therefore, the applicant's earlier conclusion that there is a minimum recharge time of 60 days is valid only if groundwater levels are lowered below el 595 ft MSL, as was done before the recharge test. Because the design-basis elevation of the dewatering system is 595 ft MSL, the minimum recharge time at the DGB appears to be only about 52 days.

The applicant's June 7, 1982 letter also shows that monitoring wells in the vicinity of the RBA were dry at the start of the recharge test. This indicates that groundwater levels were at least as low as the bottom of the screened section of the well casings. The well with the lowest screen was one having a 15-ft screened section between el 580 ft MSL and 595 ft MSL. Because this well was dry at the start of the recharge test, groundwater levels had to be at least as low as 580 ft MSL (near the RBA) at that time. This situation is similar to that at the DGB because the applicant's earlier conclusion that there is a 60-day recharge time is valid only if groundwater levels during operation are lowered as was done during the recharge test.

On March 11, 1982, about 35 days after the start of the recharge test, a high-pressure water line below the auxiliary building was ruptured by a boring. Water from this broken water line flowed until March 17, 1982, when the applicant was able to isolate the line break and stop the flowing water. The applicant states that one well located within the RBA (Well AX-2, see Figure 2.2) was flooded by the broken water line; therefore, the water level in this well does not represent a saturated water level within the backfill. The applicant further states that water levels in three other RBA wells also may have been influenced by the broken water line.

The staff has reviewed the results of the recharge test and other data provided by the applicant during an audit held on July 27-30, 1982. The staff notes that Well AX-2, which experienced the rapid water rise, is the RBA well that is nearest the broken water line. The staff agrees that the broken water line increased the rate at which the water level rebounded in Well AX-2 because of the extremely rapid rise in groundwater level as compared with that in other RBA wells. At the start of the recharge test, the groundwater level in Well AX-2 was below el 588.0 ft MSL. By March 15, 1982, 4 days after the water-line break, the groundwater level had risen to el 613.7 ft MSL. By comparison, groundwater levels in the other RBA wells did not rise above el 591 ft during this time. After the groundwater level in Well AX-2 peaked at el 613.7 ft MSL, the level began to drop and continued to drop throughout the remainder of the recharge test. Water levels in other RBA wells did not drop but continued to rise for the remainder of the recharge test.

The staff also concludes that the broken water line may have resulted in an increased rate of groundwater rise in some other RBA wells, although the increase was not as apparent as it was in Well AX-2. To determine what the recharge rate would have been at the RBA had there not been a water-line break, the staff compared the RBA well hydrographs with other hydrographs for wells located just east of the RBA that did not appear to have been affected by the broken water line. These well hydrographs showed a rate of rise of about 0.41 ft/day during the recharge test. A comparison of this rate with that in RBA wells showed that after the water-line break was repaired, water levels in the RBA wells also rose at a rate of about 0.41 ft/day. This rate is slightly higher than the groundwater rate of rise for the DGB, where groundwater levels rose at a rate of about 0.35 ft/day during the last 2 weeks of the recharge test.

Because groundwater levels at the RBA rose at a slightly faster rate than at the DGB during the recharge test, the staff does not agree with the applicant's earlier reasoning that groundwater levels would rise more slowly at the RBA than at the DGB in the unlikely event of a complete failure of the dewatering system. The reason groundwater moves at a faster rate toward the RBA can be seen by referring to Figure 2.3 of this SSER. As explained in Section 2.4.6.2 of the SER, the source of the groundwater in the plant fill area is seepage from the cooling pond near the service water pump structure (SWPS) and the circulating water intake structure (CWIS). Figure 2.3 shows that the plant fill between the seepage source and the RBA consists of a thick layer of sand that has a bottom elevation ranging from el 540 ft MSL to 570 ft MSL. By comparison, the plant fill between the seepage source and the DGB is much shallower, with a bottom elevation between 580 ft and 590 ft MSL. The lateral extent of the sand toward the RBA is also much wider than toward the DGB.

Because water in the plant fill area would rise faster at the RBA than at the DGB, the staff now concludes that in the event of a complete failure of the dewatering system that could not be repaired, groundwater levels could rise from el 595 ft to 610 ft MSL at the RBA in about 40 days. However, on the basis of the applicant's discussion of possible dewatering system malfunctions and estimated repair times, as described in Section 2.4.6.4 of this SSER, the staff concludes that 40 days is a sufficient period to both repair dewatering system malfunctions and, if necessary, shut down the plant.

2.4.6.3 Effects of Pipe Breaks

In Section 2.4.6.3 of the SER, the staff concluded that failure of either a circulating water discharge line or a condensate storage line would not result in groundwater levels above the groundwater design level of 610 ft MSL. However, the staff required verification from the applicant that there are no other pipes whose failure could affect the dewatering system's ability to maintain the required groundwater levels. Since publication of the SER, the applicant has evaluated additional postulated breaks in the dewatering system header line, cooling pond blowdown line, and the cooling tower line.

A break in the dewatering system header line could result in return flow to the dewatering wells in the vicinity of the broken line. In that event, inflow of water could exceed the capacity of the affected pumps, producing a rise in groundwater levels in the immediate vicinity of the affected wells. To remedy this interaction, the dewatering system has been designed to permit a flexible hose to be attached to the affected wells so that the flow can be diverted temporarily until the header line can be repaired. A header-line break at the interceptor wells, which are located adjacent to the CWIS and SWPS, would cause activation of one or more of the backup wells, which are located on a separate header line. This would prevent any additional flow from a header-line break at the interceptor wells from exceeding the capacity of the dewatering well pumps. (As explained in Section 2.4.6.2 of the SER, the applicant determined that 20

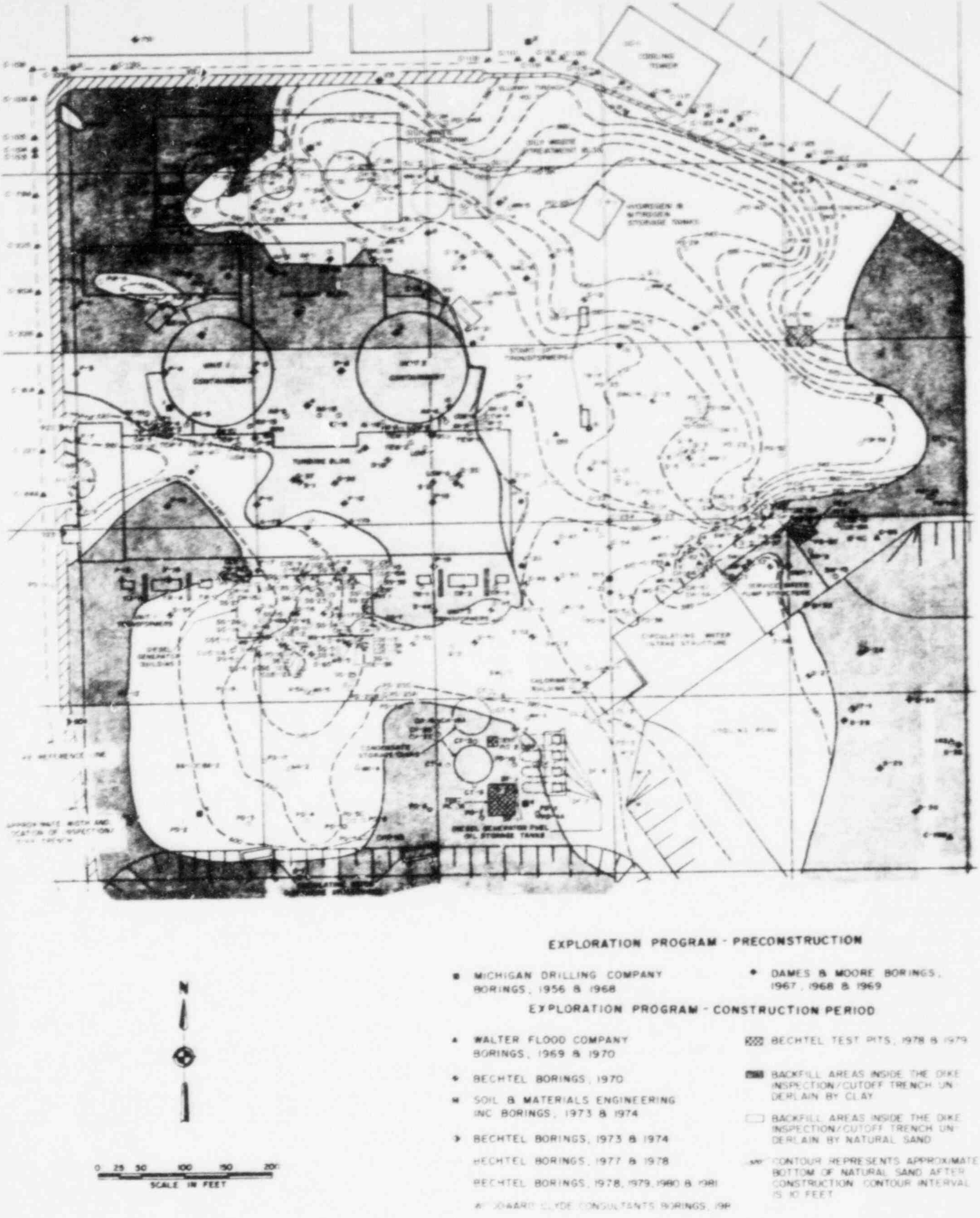


Figure 2.3 Contours on bottom of natural sand after construction, with shaded portions representing areas underlain by clay and unshaded portions representing areas underlain by natural sand (Source FSAR Figure 2.4-49)

interceptor wells are required. However, to provide for uninterrupted service, a second line of 20 backup wells has been installed.)

Breaks in either the cooling tower line or the cooling pond blowdown line would have minimal impact on the dewatering system because these are low-pressure lines and the volume that would be introduced into the groundwater system by a line break would automatically be removed by the dewatering system, which has sufficient capacity to remove all of the released water from such a line break.

The staff has reviewed the applicant's analysis and concludes that failure of an underground pipe line in the plant fill area would not result in groundwater levels above the design level of 610 ft MSL and, therefore, would not cause a liquefaction concern.

2.4.6.4 Dewatering Monitoring Program

In Section 2.4.6.4 of the SER, the staff concluded that it was unable to determine whether the monitoring system proposed by the applicant for the permanent dewatering system was adequate to detect any unexpected rise in groundwater levels because the extent of the areas to be dewatered had not been identified. Thus, the applicant was requested to provide a dewatering control plan that would identify the specific areas to be dewatered and the monitoring wells and/or piezometers that will be used to ensure that the dewatered levels are maintained.

The applicant subsequently submitted a delineation of the areas that will be dewatered (Figure 2.4). The staff reviewed this submittal and concluded that the delineated areas adequately encompass the areas of loose sands beneath the DGB and the RBA that have to be dewatered to preclude liquefaction during a seismic event.

The applicant has not identified the monitoring wells and/or piezometers that will be used to ensure that dewatered levels within the delineated areas are maintained except for two permanent monitoring wells in the DGB area and two in the RBA (see Figure 2.1). These permanent monitoring wells will send continuous water-level data to recorders located in the evaporator building. In addition, these wells will have alarms that will be activated when a significant water-level rise occurs in any of the wells.

The staff agrees that two permanent wells (with continuous recorders) in each delineated area are a sufficient number of recording wells. However, the staff requires that other nonrecording observation wells also be monitored in the delineated areas. Figure 2.4 shows that a sufficient number of additional observation wells are already located within the delineated areas. The staff will require the applicant to monitor some of these wells to supplement the recording wells. This requirement and details of the selected monitoring wells will be included as part of the permanent dewatering system Technical Specification.

In a June 14, 1982 submittal, the applicant also provided additional information regarding the permanent dewatering system Technical Specification. The

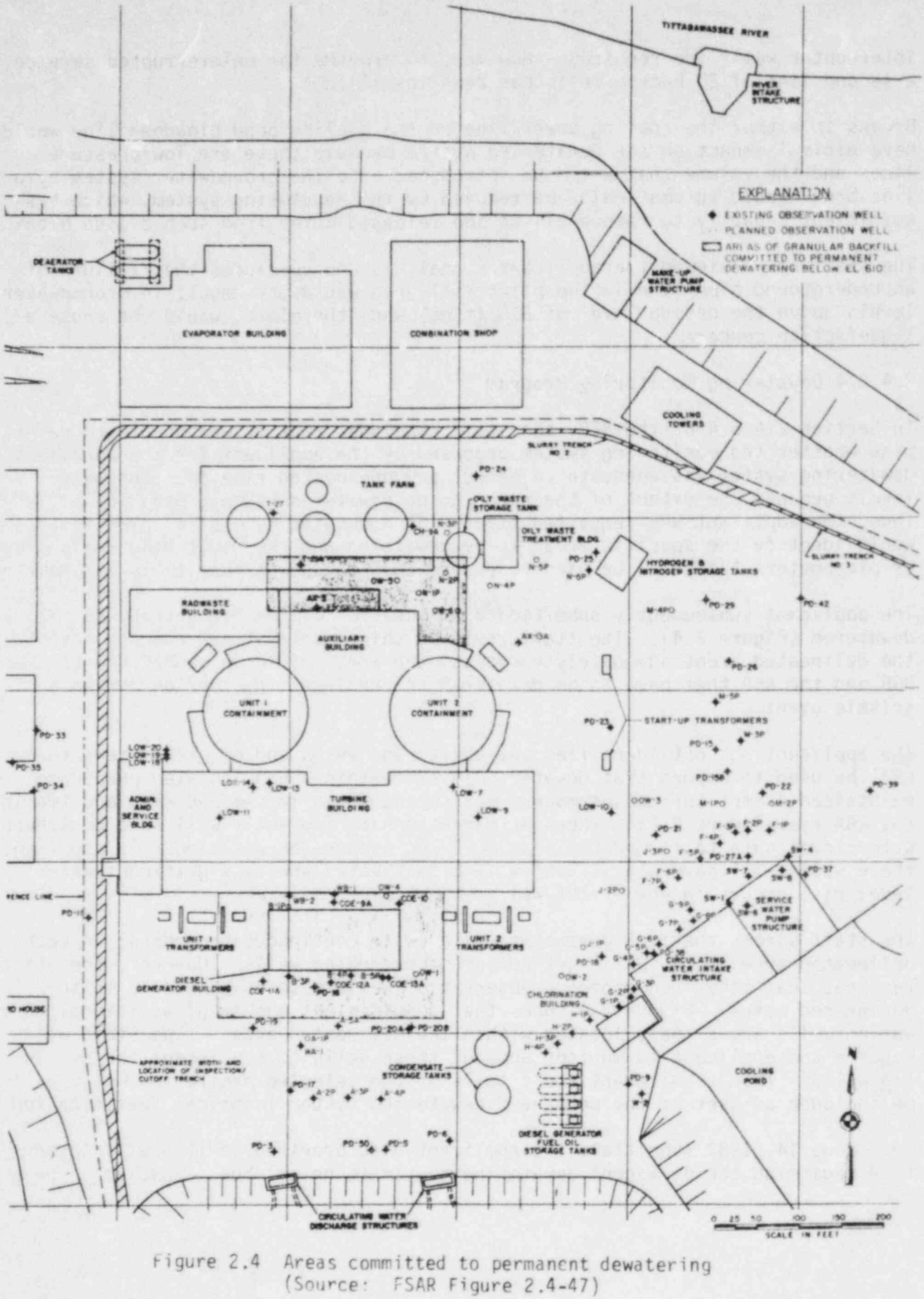


Figure 2.4 Areas committed to permanent dewatering
(Source: FSAR Figure 2.4-47)

Table 2.1 Dewatering well failure mechanisms and responses

Cause of failure	Repair time
Electrical failure	
Single well (wired in parallel)	Less than 1 day to determine cause of failure and to repair and restore well.
Multiple wells (because of power outage)	1 day to initiate operation of backup diesel power to interceptor wells. Operation to continue until normal power can be restored. Backup interceptor wells automatically begin pumping if water levels exceed el 595 ft.
Timers/pumps/ check valves	Less than 1 day to replace part; replacement parts are stored on site.
Header pipe break	1 day to attach flexible hose to each well affected and pump water to storm drains. In case of interceptor well header failure, backup wells (on separate header system) to be initiated.
Well screen encrustation	2 days to acidize well.
Complete loss of well	4 days to replace one well using cable tool rig; 1 day if other drilling method used. If well or wells need to be replaced, there is enough redundancy and pumping capacity to prevent water levels from rising in plant fill while the replacement wells are being installed.

applicant stated that a Technical Specification will be prepared detailing the coordination of plant shutdown should groundwater levels rise excessively. Should the water level in any observation well rise above el 595 ft MSL, the plant operator will verify whether it is a correct reading and, if it is, will identify the cause and implement the repair measures given in Table 2.1. If the repair measures do not effectively limit the rise in groundwater levels, the plant will be shut down several days before el 610 ft MSL is reached. The applicant estimated that 1.5 days would be required to bring the plant to a cold shutdown condition and proposed a margin of several days, which will be added to the 1.5 days for the actual technical specification. The Technical Specification for the DGB will be based upon a groundwater level rise of 0.35 ft/day, as determined from the recharge test.

As stated in Section 2.4.6.2 of this SSER, the staff finds that groundwater levels at the RBA will rise at a rate of about 0.41 ft/day, which is faster than the 0.35-ft/day rise determined by the applicant for the DGB. This rate will be used for the Technical Specification for the RBA to determine the groundwater level at which the plant will be shut down.

The staff has reviewed the applicant's proposed dewatering system design and test results and concludes that there is a high degree of assurance that the system will maintain the loose sands beneath the DGB and the RBA in an unsaturated condition, and thereby preclude potential liquefaction during a seismic event. In arriving at this conclusion, the staff considered the following:

(1) Although 20 wells are required to intercept seepage from the cooling pond, a total of 40 interceptor wells has been provided.

(2) Another 24 wells are located in the plant fill area to remove any water that is not collected by the 40 interceptor wells.

(3) Electrical wiring of the wells will be designed so that the temporary outage of one or more wells will not affect the remaining wells.

(4) A backup diesel generator will be available for temporary operation of the interceptor wells if the overall power supply is disrupted.

(5) It is unlikely that the entire dewatering system will fail. It is more likely that there will be partial failures. The applicant has indicated that partial failures can be repaired in a few days.

(6) Although a total failure of the dewatering system is unlikely, the applicant has also shown by an actual field test that it would take at least 40 days for water levels to rise from el 595 ft to the level of the loose sands (elevation 610 ft) if the dewatering system failed completely. This provides both a reasonable period to effect repairs and, if necessary, sufficient time to shut down the plant. Therefore, the basis for establishing a suitable Technical Specification for dewatering has been provided.

The staff concludes that the proposed dewatering system meets the criteria of Branch Technical Position (BTP) HGEB-1 of Standard Review Plan (SRP) Section 2.4.12 (NUREG-0800), except for the details of the dewatering Technical Specification that will be developed at the appropriate stage of staff review prior to plant operation and is, therefore, acceptable.

2.5 Geology and Seismology

2.5.4 Stability of Subsurface Materials and Foundations

In Section 2.5.4 of the SER, the status of the staff's geotechnical engineering review of the Midland Plant was provided, and it was indicated that a more detailed evaluation of the stability of subsurface materials and foundations for seismic Category I safety-related structures and components would be presented in a supplement. Since issuance of the SER, the applicant has submitted several technical reports addressing previously identified staff review concerns. These reports, dated through September 3, 1982, along with the documents identified in Section 2.5.4 of the SER and information received during a July 27-30, 1982 design audit, have been reviewed by the staff and serve as the basis for the following sections.

In addition to identifying the applicable criteria under which the FSAR Section 2.5.4 review has been conducted, the SER also discussed the following topics related to the plant fill settlement problems:

(1) discovery of the plant fill deficiencies (SER Section 1.12)

(2) affected safety-related structures and utilities (SER Section 1.12 and SER Table 2.2)

(3) NRC issuance of the Order Modifying Construction Permits and a related Licensing Board Order (SER Section 1.12)

Staff review of geotechnical engineering areas, including underpinning specialties, has been performed with technical assistance of consultants from the U.S. Army Corps of Engineers and Geotechnical Engineers, Inc.

2.5.4.1 Site Conditions

2.5.4.1.1 General

The proposed Midland Plant is located in central Michigan on the southwest bank of the Tittabawassee River. Topographic relief is slight in the site area, with elevations ranging between 594 ft (National Geodetic Datum) along the Tittabawassee flood plain to 630 ft in the southwest portion of the site area. To reach the plant-grade elevation, 634 ft, and to be above the flood plain, 30 to 35 ft of fill had to be placed and compacted above the natural ground surface after removal of organic and topsoil materials. The borrow source of soil materials for the plant fill was the 880-acre cooling pond area located south of the plant area, as shown in FSAR Figure 2.5-46. The average original ground surface that existed before the placement of the plant fill was slightly above 600 ft, and it is this surface below which future references in this supplement to natural soils are intended. Plant fill placement activities were conducted mainly from 1975 to 1977.

Subsurface explorations in the natural soils in the main plant area reveal highly variable soil materials and layering conditions that are typical of a glaciated plain. A loose to very dense, brown fine sand (SP) is found beneath the thin topsoil layer. The bottom of the surface sand layer varies in the main plant area from el 575 ft to 600 ft but has been located as deep as el 552 ft in site explorations. Underlying the fine sandy soils is a preconsolidated, very stiff to hard, gray, silty clay (CL) that contains numerous discontinuous silt lenses. This natural foundation clay layer is a lacustrine deposit and extends as deep as el 545 ft. Glacial till that consists of a very stiff to hard, brownish-gray, silty clay (CL-CH) with sand and gravel is located beneath the lacustrine clay layer. The glacial till brownish-gray, silty clay layer is very thick and extends to bottom elevations ranging from 365 to 430 ft. Below the clay till and above the black shale bedrock of the Saginaw Formation lies glacial outwash consisting of predominantly very dense, fine sand layers (SP) with silt, which are occasionally interlayered with very stiff clayey sands, very dense sand and gravels, and very dense silts with gravel. The top of bedrock is encountered at approximately el 250 ft in the main plant area, as shown on FSAR Figure 2.5-23.

Plant fill placed beneath safety-related structures and utilities consisted mainly of the lacustrine and till clays that were excavated from the cooling pond area. Clean sands (structural backfill) from an offsite source and lean concrete, used as an alternative to the structural backfill, were also placed in the plant fill. Inadequate compaction of the clay and sand fill to required compaction criteria (95% of maximum dry density established in American Society for Testing and Materials (ASTM) test D1557 and 80% relative density established in ASTM test D2049) is considered to be the major cause of the plant fill settlement problem.

2.5.4.1.2 Site Foundation Description

Tables 2.2 and 2.3 of this SSER provide a summary of the pertinent foundation information for seismic Category I structures that are founded on the natural soils and plant fill materials. In addition to providing the bottom foundation elevations and foundation type, the footnotes to these tables also indicate the foundation remedial measures proposed for the various structures supported on the plant fill.

Table 2.2 Safety-related structures founded on natural soils

Structure	Supporting foundation soil	Foundation elevation (ft)	Foundation type
Reactor containments	Very stiff to hard clay	572 to 582.5	9-ft to 13-ft-thick reinforced concrete mat
Auxiliary building (main portion)	Very stiff to hard clay	562 to 579	5-ft- to 6-ft-thick reinforced concrete mat
SWPS (southern portion)	Very stiff to hard sandy clay	587	5-ft-thick reinforced concrete mat

Table 2.3 Safety-related structures founded on plant fill

Structure	Supporting foundation soil	Original foundation elevation (ft)	Original foundation type
Auxiliary building (portions listed below)			
CT	Plant fill	609(1)	5-ft-thick(1) reinforced concrete mat
EPAs	Plant fill	609(1)	5-ft-thick(1) reinforced concrete mat
RBA	Plant fill	630.5	4-ft-thick reinforced concrete mat
FIVPs	Plant fill	615.5(2)	4-ft-thick(2) reinforced concrete mat
SWPS (northern portion)	Plant fill	617(1)	3-ft-thick reinforced(1) concrete mat
DGB	Plant fill	628	2.5-ft-thick(3) by 10-ft-wide continuous reinforced concrete wall footing
Diesel fuel oil tanks	Plant fill	612	3-ft-thick(4) concrete pads
Borated water storage tanks	Plant fill	629	Continuous(4, 5) reinforced concrete ring wall on 1.5-ft-thick by 4-ft-wide footings

(1) To be modified with permanent underpinning wall.

(2) To have original plant fill removed and replaced with concrete and compacted granular fill.

(3) Subjected to surcharging with sand fill.

(4) Preloaded by filling tanks with water.

(5) New ring beam to be constructed around existing ring wall foundation to be constructed and Unit 1 tank to be reset.

The variations in groundwater, river, and cooling pond levels that affect foundation design are discussed in Section 2.4 of the SER.

2.5.4.1.3 Site Investigations

Some preliminary explorations were completed at the plant site as early as 1956, but the major portion of the preliminary exploratory program was completed in 1968 and 1969. FSAR Table 2.5-8 lists the borings that have been completed at the various structure locations, and FSAR Figures 2.5-16, 17, 17A, and 17B show the locations of these explorations. Approximately 200 of the more than 900 borings that have been drilled at the plant site were completed in the preliminary exploration phase. A large number of the later borings were drilled for reasons related to investigation of the plant fill problem and for design of remedial measures such as the permanent dewatering system. The major objectives of the site investigation program included determination of subsurface materials and stratification, investigation of suitable borrow sources, identification of the extent of natural and fill sand layers because of concerns about liquefaction or seepage beneath the cutoff trench beneath plant area dikes, measurement of shear and compression wave velocities of both the natural and the questionable plant fill soils, and the recovery of representative disturbed and undisturbed soil samples for field and laboratory testing to establish static and dynamic engineering properties. The depth of borings varied widely and ranged from a minimum of 4 ft up to 370 ft, where rock cores were obtained using a 3.5-in. diamond bit (NX) core barrel.

On the basis of its review of the information presented in the FSAR and technical reports, the staff concludes that the site investigations completed by the applicant are acceptable and adequate to identify the important subsurface features and foundation conditions. The staff also concludes that the investigations were completed in accordance with the guidelines recommended in RG 1.132, "Site Investigations for Foundations of Nuclear Power Plants."

2.5.4.2 Properties of Foundation Materials

The description of foundation material types and layering has been presented in Section 2.5.4.1.1. The engineering properties of these materials were determined by laboratory and field testing. In addition to the usual classification tests, laboratory testing also included compaction; shear strength (unconsolidated undrained, consolidated undrained with pore pressure measurement, and consolidated drained); permeability; consolidation; cyclic triaxial; mineralogy, cation-exchange capacity, swell characteristics, and dispersive nature of the clays; and rock compression tests. Field testing included plate load bearing, standard penetration test (SPT), permeability, in situ density, and geophysical surveys to determine depth to bedrock and to measure in situ compression and shear wave velocities of both the natural and fill soils. Descriptions of the tests and the results of the laboratory and field testing are presented in FSAR Section 2.5.4.2.

Some of the engineering properties of the supporting foundation soils at the Midland site previously identified in Section 2.5.4 are listed in Table 2.4 of this SSER.

Table 2.4 Engineering properties of natural and fill foundation soils

Foundation soil	Liquid limit (%)	Plastic limit (%)	Shear strength* Undrained C (ksf)	Shear strength* Drained Ø (degrees)	Shear strength* Drained C (ksf)	SPT (blows/ft)	Shear wave velocity** (fps)
Natural: very stiff to hard clay (CL) (reactor and auxiliary bldg)	42	20	7.0†; range = 5.2 - 9-3; median = 7.6	23†	1.2†	median = 56	850-2300†
Natural: very stiff to hard sandy clay (CL) (SWPS)	17	11	8.0†; range = 11.4 - 18.2; median = 15.0	36	0.7	median = 75	850-2300†
Plant fill: silty clay (CL) (DGB after surcharging)	19-46	11-18	3.0; 2.7†	32; 29†	0.1; 0.1†	--	500-1000†
Bedrock - black shale	--	--	Unconfined compression = 6000 to 7600 psi		--	5000†	

NOTE: *C = cohesion; Ø = angle of internal friction
**Shear wave velocity increases with depth of soil layer
†Used in design

On the basis of its review of the information provided by the applicant in the FSAR and technical reports, the staff concludes that the laboratory and field test results are acceptable with respect to adequacy and reasonableness of results and in meeting the applicable portions of the Commission's regulations, the SRP, and RG 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants."

2.5.4.3 Foundation Profiles and Design Properties

Pertinent soil profiles and sectional views that present the results of the subsurface investigations in relation to the horizontal and vertical locations of the various seismic Category I structures are listed in Table 2.5 of this SSER. The staff will require submittal of the actual as-built foundation conditions for the auxiliary building and SWPS portions in a future revision to the FSAR following completion of this underpinning construction work.

Table 2.5 Pertinent soil profiles and sectional views presenting subsurface investigation results

Structure	Profile or section
Reactor containments	FSAR Figures 2.5-20, -21, and -169
Auxiliary building	FSAR Figures 2.5-20, -21, and -169; Figure AUX-38 - from testimony of Edward M. Burke, W. Gene Corley, James P. Gould, Theodore E. Johnson and M. A. Sozen, Volume 2, "Figures," December 1, 1981 (transcript at page 5509).
SWPS	FSAR Figures 2.5-22 and -170 SSER Figure 2.8
BWSTs	FSAR Figures 2.5-176, -182, and -183; Figures 4, 5, and 6 from hearing testimony of Dr. Alfred J. Hendron, Jr., February 16, 1982 (transcript at page 7186)
DGB	FSAR Figure 2.5-177 (prior to surcharging)
Underground piping	FSAR Figures 2.5-100 and -101
Diesel fuel oil storage tanks	FSAR Figures 2.5-191

The staff concludes that the soil profiles and sectional views are adequate and acceptable in appropriately representing the results of the subsurface investigations. The staff also finds acceptable the engineering properties that have been used in design as shown in Table 2.4 of this SSER for the various foundation layers depicted on the soil profiles and sectional views.

2.5.4.4 Foundation Treatment

The following sections of this SSER provide the evaluation by the staff and its consultants of the techniques proposed by the applicant to treat the deficiencies in the plant fill and to ensure long-term foundation stability.

2.5.4.4.1 Underpinning

This section describes the cause of the need for underpinning and evaluates the design of the underpinning systems. Because underpinning work may cause movement and stressing of the underpinned structures, and because this stressing is dependent chiefly on the construction procedures used in the excavation drifts and pits to remove the present supporting soils beneath the existing structures, the staff also has evaluated the construction and construction control procedures.

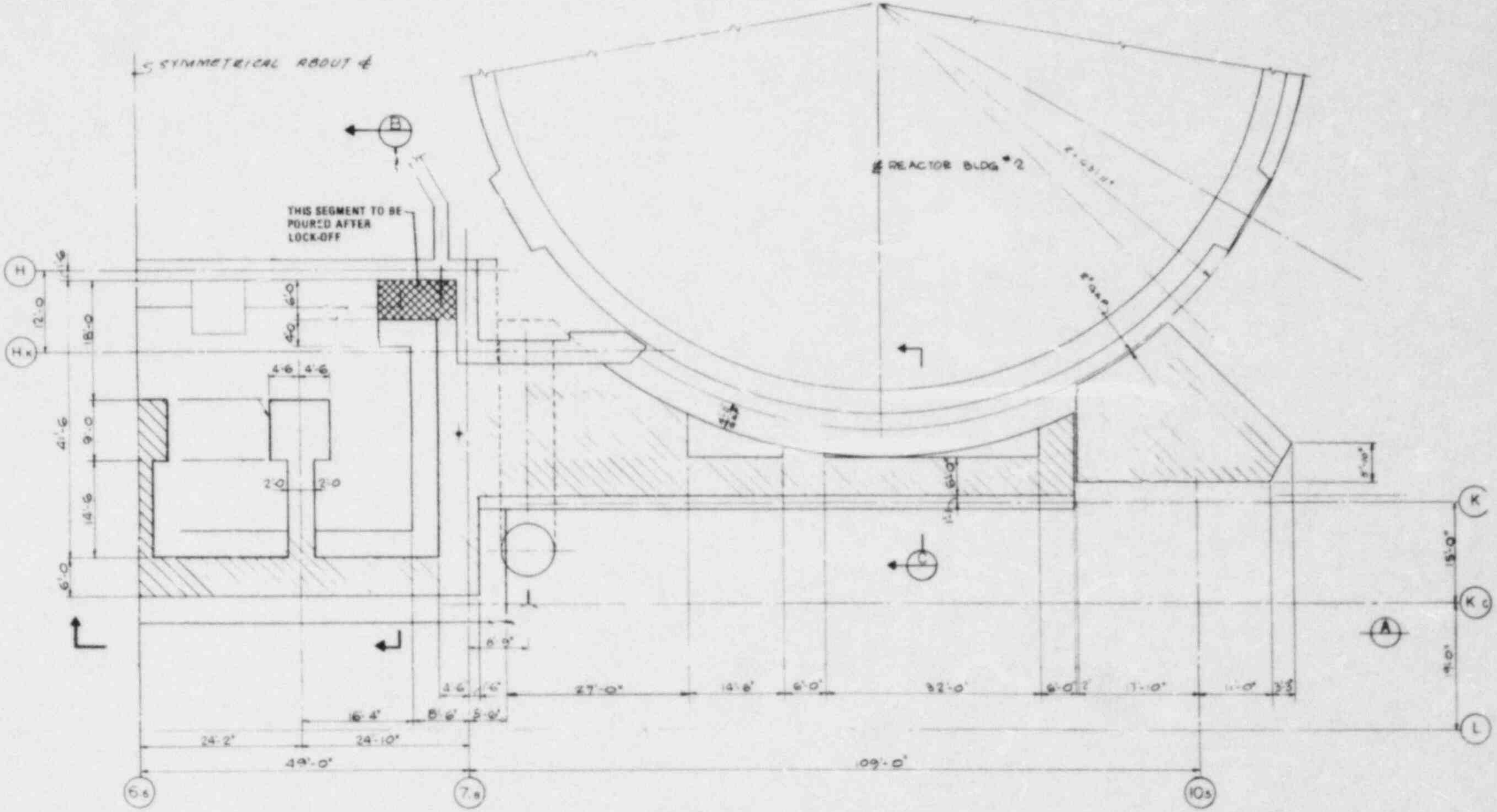
The main auxiliary building is founded on the very stiff to hard natural clay soil, with foundation elevations ranging between 562 and 579 ft. The control tower (CT) and electrical penetration areas (EPAs), which are structurally connected to the southerly end of the main auxiliary building, currently are founded at el 609 ft on inadequately compacted plant fill that varies up to 30 ft thick. Large volumes of concrete used as a replacement for structural backfill in the excavations around the main auxiliary building and reactor building foundations are also found in the plant fill. The feedwater isolation valve pits (FIVPs) are located at the extremities of the EPAs, and they are presently founded on inadequately compacted plant fill at el 615.5 ft. The FIVPs, which are structurally separated from the other buildings, house seismic Category I piping that penetrates the adjacent reactor containment and turbine building.

The low SPT blow counts in the plant fill at the auxiliary building area obtained during the late 1978 subsurface investigations caused concern about future differential settlements. Because the CT and EPAs were not designed to cantilever from the main auxiliary building, the differential settlements could cause unacceptable stresses. A 1-ft-deep void also was discovered in one of the borings beneath the mud mat under the CT during the late 1978 investigations. Evidence of cracking at several locations on the auxiliary building was additional reason for concern.

To ensure long-term foundation stability, the applicant has proposed to underpin the CT and EPAs with a new permanent underpinning wall that will extend through the plant fill to the competent hard clay natural soil on which the main auxiliary building is also founded. Plans and section views of this underpinning for the auxiliary building are shown on Figure 2.5 of this SSER. The permanent underpinning wall will be connected to the bottom of the existing mat foundations (and to the main auxiliary building beneath the CT) after the structure load has been held long enough with jacks on the underpinning to reduce future settlements to minimal values. Details of the connection for the underpinning walls are shown on Figure 2.6 of this SSER.

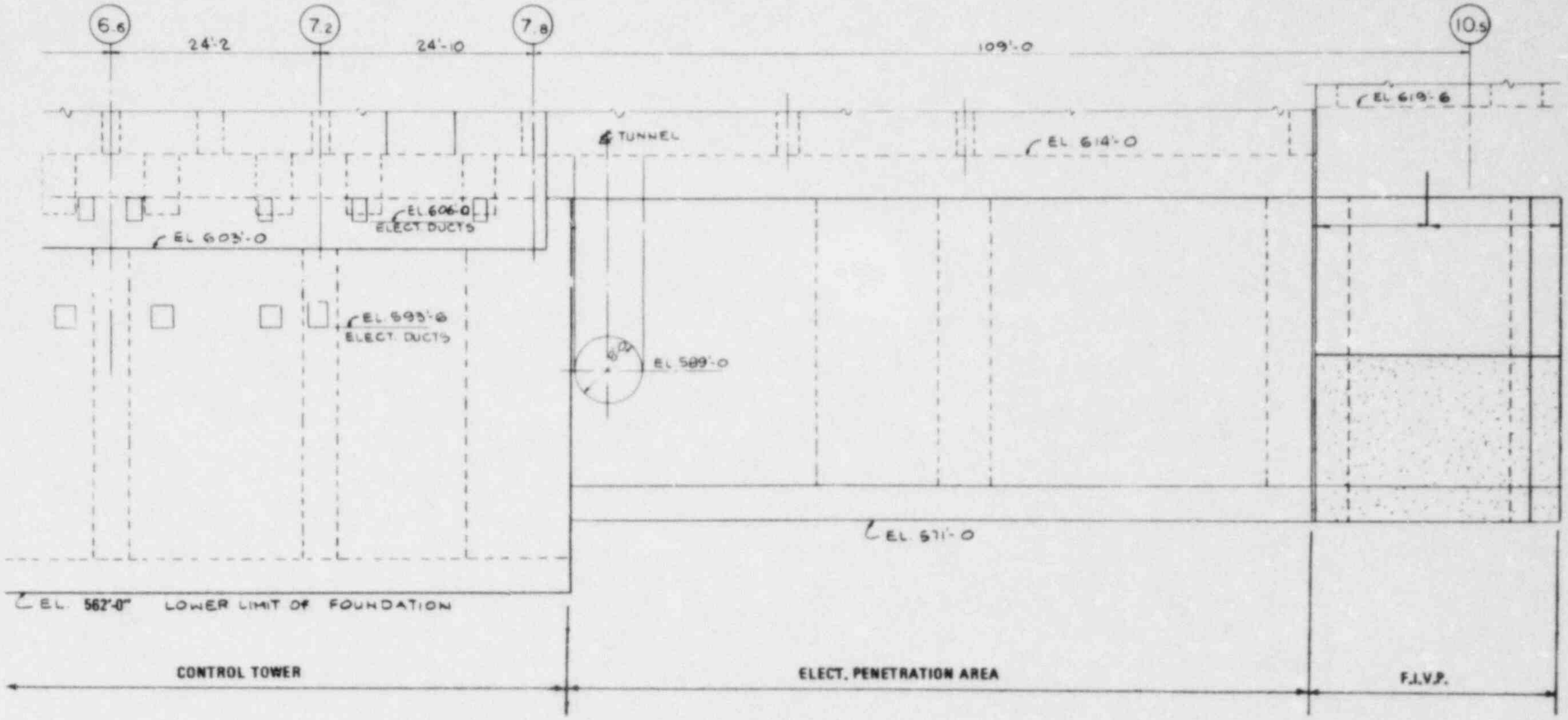
Foundation treatment for the inadequate plant fill beneath the FIVPs consists of excavating the fill and a portion of the hard clay and replacing it with approximately 30 ft of compacted granular fill and 4 ft of concrete fill. Figuare 2.7 of this SSER shows the resulting support. The granular fill is to be compacted to 95% of maximum dry density, as determined by ASTM test D-1557 or ASTM test D-2049, whichever results in the greater maximum dry density. The granular fill has been specified and is to be compacted with proper equipment and control of placement water. The applicant has committed to following a test procedure for controlling compaction that is acceptable to the staff.

The granular fill and the concrete beneath the FIVPs will be separated by a jacking slab that will be used to remove the load of the FIVP structures from the existing temporary overhead supports and place it on the granular fill. Thus, most of the settlement of the granular fill will occur while the jacks are in place and before transfer of the final load to the permanent foundation



PLAN AT EL 603'-0"

Figure 2.5 Auxiliary building underpinning plan and section
(Source: FSAR Figure 3.8-61A)



SOUTH ELEVATION OF UNDERPINNING WALL

SECTION A

Figure 2.5 Continued (Source: FSAR Figure 3.8-61B)

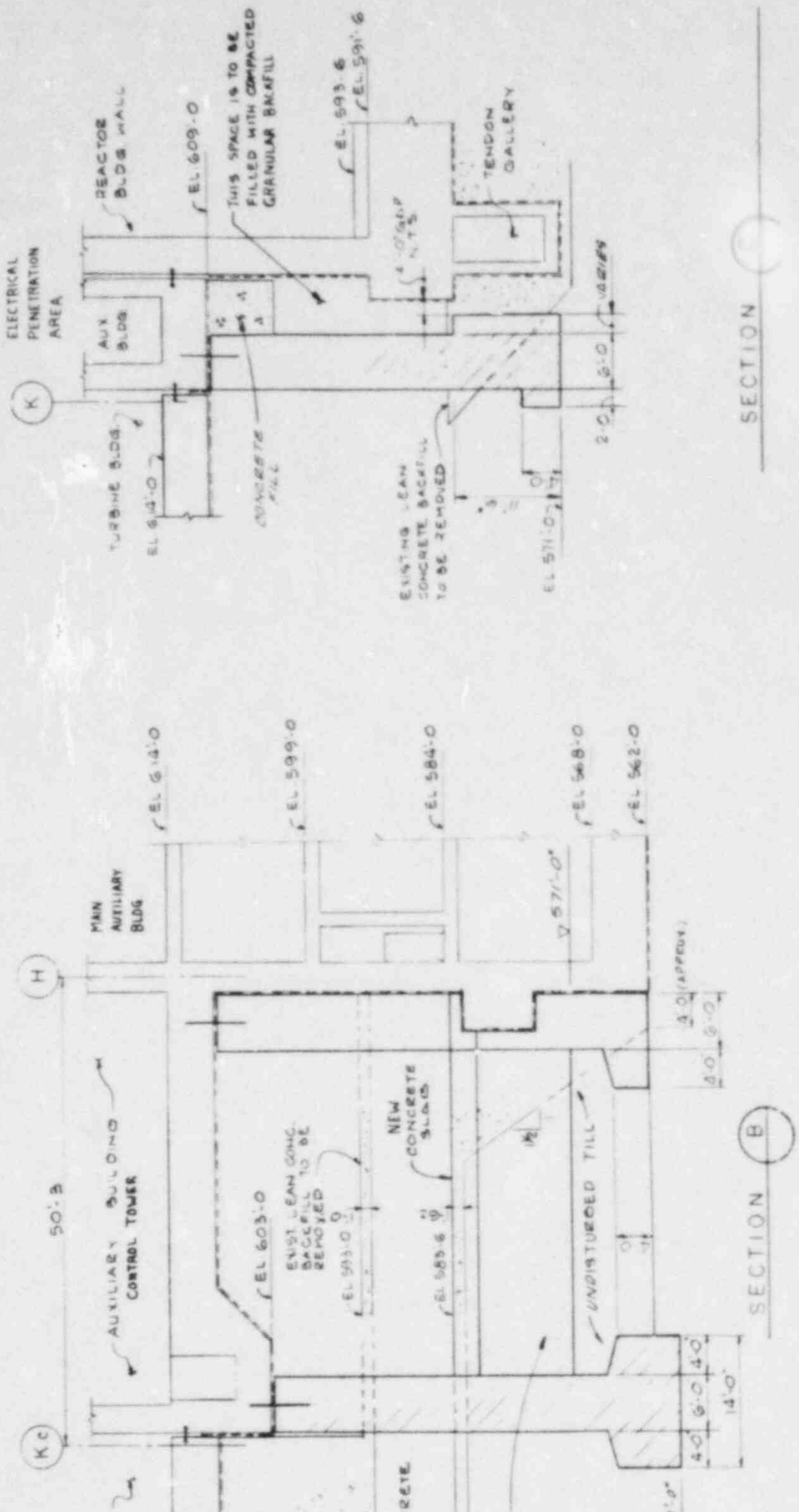
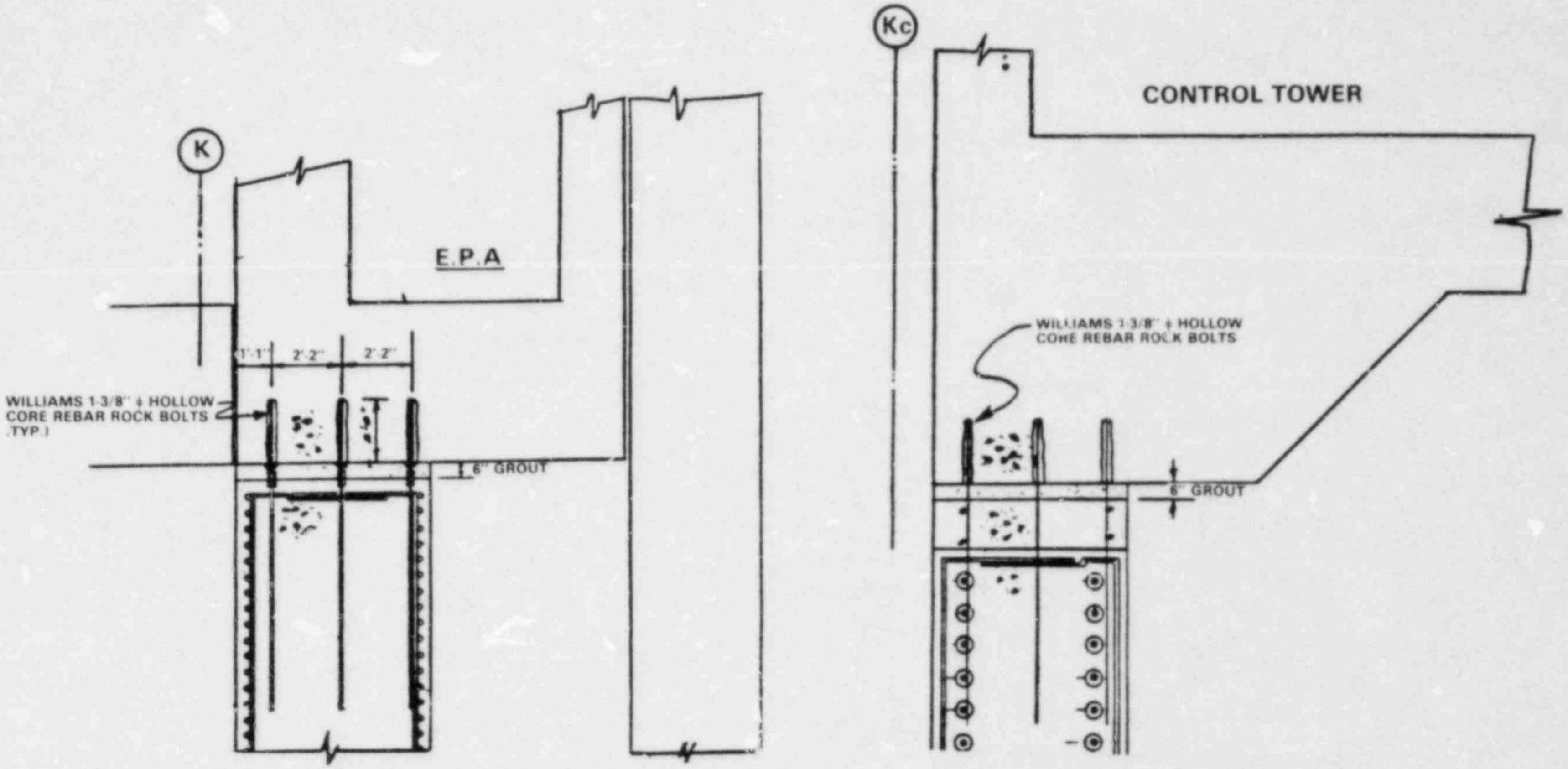


Figure 2.5 Continued (Source: FSAR Figure 3.8-61C)



Detail of connection of underpinning to electrical penetration area

Detail of connection of underpinning to control tower

Figure 2.6 Detail of auxiliary building underpinning wall connections (Source: Applicant's letter of June 7, 1982)

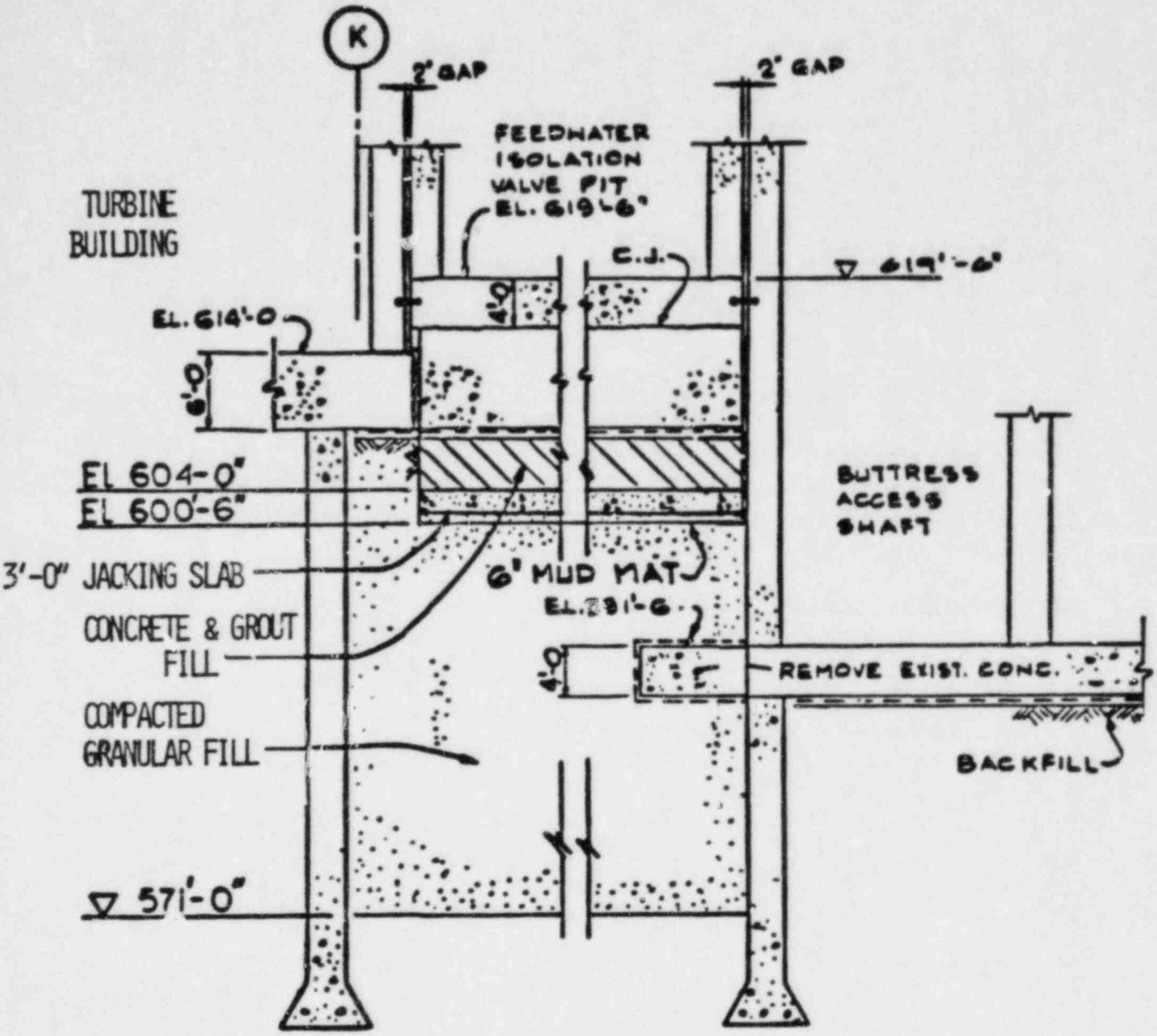


Figure 2.7 Feedwater isolation valve pit support
(Source: FSAR Figure 3.8-85)

is completed. Subsequent settlements are anticipated to be minimal. The FIVPs currently are temporarily supported by an overhead steel structure that is bolted to the existing concrete structure. The overhead structure transfers the load to the adjacent turbine building and containment buttress access shaft.

An underpinning construction sequence and load transfer procedure was developed by the applicant and reviewed by the staff. The load transfer procedure is expected to cause additional differential settlement well below 0.4 in. between the south ends of the underpinned structures and the main auxiliary building. An extensive instrumentation program has been developed to monitor settlements and strain during underpinning, as described in Section 2.5.4.6 of this SSER. In addition, a series of contingency plans has been developed (Bechtel specification C-200) that will be implemented to reduce future movements if the observed settlements and strains during early stages of underpinning are larger than expected. These contingency plans will be implemented when the movements are well within the tolerable limits for each structure, based on direct observation of the structure. The material, equipment, and personnel will be available on site to implement any necessary contingency plans.

The staff considers the underpinning design and the construction procedures, as well as the instrumentation to monitor underpinning, to be conservative. Contingency plans have been prepared and will be ready for implementation if the behavior of the buildings is found to be different from the expected behavior. In addition, the administrative and technical procedures for relating the settlement and strain data to activities in the drifts and pits have been reviewed and evaluated. The critical observations will be made hourly or more frequently during critical stages of underpinning. These procedures, in total, represent a higher degree of control over construction operations than normally applied for underpinning construction in recognition of the characteristics and safety classification of these structures.

Based on its review and evaluation of the documents submitted by the applicant for modifying the foundations of the CT, EPAs, and FIVPs, the staff concludes that the proposed permanent underpinning wall fix and the construction procedures represent a conservative solution for eliminating the plant fill problem in the auxiliary building area and, once properly executed, will provide a stable and safe foundation.

Conditions at the northerly portion of the SWPS are similar to the conditions beneath the CT and EPAs in that this portion is founded on the clay and sand plant fill and is structurally connected to the southerly part of the SWPS, which is founded on the deeper, more competent, very dense sandy clay till. The concerns about differential settlement between the shallower, northerly portion, which overlies the plant fill, and the southerly portion, which is founded on till, along with unacceptable stresses, have prompted the applicant to require a new permanent underpinning wall to ensure long-term foundation stability. In addition, cracks have been observed in the SWPS at locations where they might be expected to develop if the above differential settlements were occurring. A profile of the foundation soils beneath the SWPS is presented in Figure 2.8 of this SSER.

The proposed new permanent underpinning wall beneath the north portion of the SWPS will extend through the fill to at least el 587 ft, which is the same bearing level as the existing deeper portion. Plan and section views of the underpinning are presented in Figure 2.9 of this SSER. (As shown on Figure 2.9, the SWPS and CWIS actually face northwest; however, for ease of discussion, the text throughout this SSER assumes they face north.)

An instrumentation system as described in Section 2.5.4.6 of this SSER will be installed to monitor differential settlements and strains at critical points in the SWPS. A differential settlement of the northerly portion relative to the southerly portion of 0.07 in. will cause contingency plans (Bechtel specification C-200) to be implemented to limit further movements.

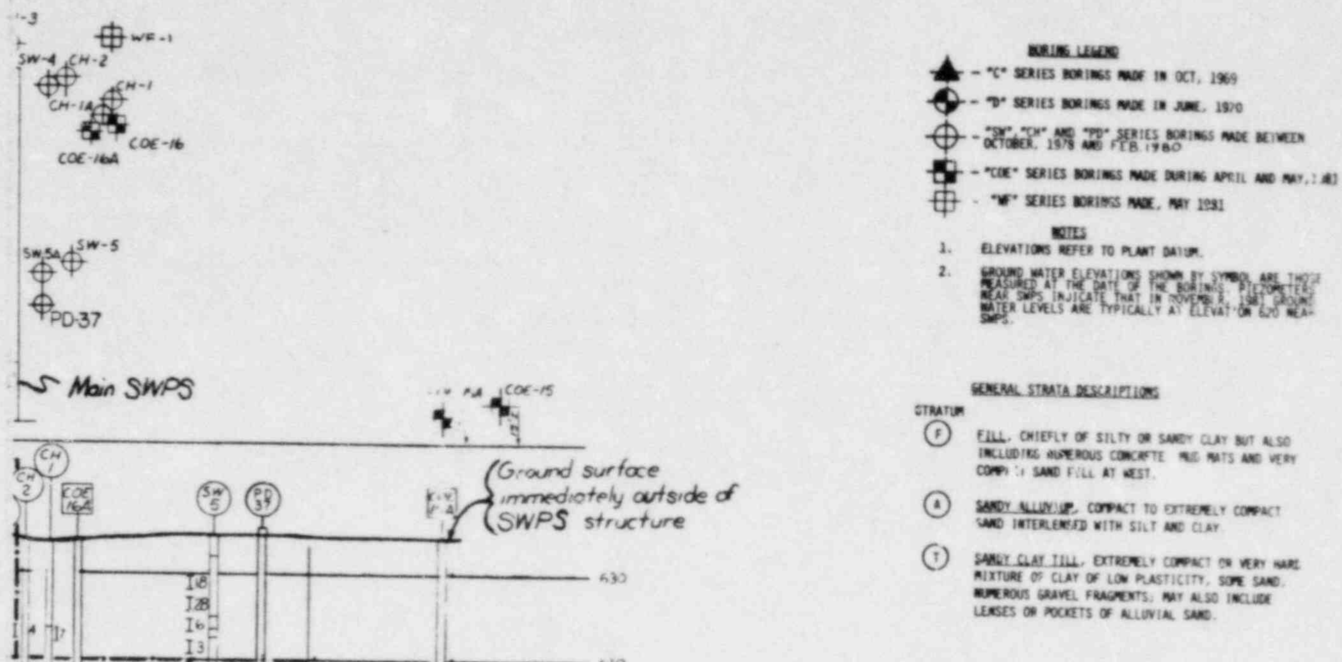
The sequence of construction and the procedures for transferring load from the jacks to the permanent underpinning wall have been reviewed. These procedures are expected to limit movements and stress increases during underpinning to values well within acceptable values. The technical and administrative procedures for implementing construction and control have been reviewed by the staff and found to be suitable.

Based on its review and evaluation of the documents provided by the applicant for underpinning the SWPS, the staff concludes that the underpinning fix is a conservative solution for eliminating the fill settlement problem and, once properly carried out in the field, will provide a stable and safe foundation.

2.5.4.4.2 Surcharging of the Diesel Generator Building Area

The diesel generator building (DGB) is a reinforced concrete structure that is supported on continuous wall footings that are founded at el 628 ft. The footings rest on approximately 25 ft of plant fill and were poured in October 1977. The structure is further described in Section 3.8.3.4 of this SSER. In July 1978, with the generator pedestals and approximately 60% of the DGB completed, field settlement measurements begun in March 1978 indicated larger-than-predicted values of settlement. By December 1978, the largest measured settlement, located in the southeast corner of the building, had reached 4.25 in., which already exceeded the building's initial 40-year settlement prediction of 2.8 in.

The applicant temporarily halted construction of the DGB and completed subsurface explorations in the plant fill in late 1978. The results of these explorations revealed that the fill did not meet specified compaction requirements at all points in the fill. The fill was shown to be highly variable and ranged in consistency from very soft to very stiff for the cohesive soils and from very loose to dense for the granular soils. After considering several alternatives for rectifying the inadequately compacted fill, the applicant, on the advice of his consultants, elected to surcharge the partially completed structure with 20 ft of sand placed above plant-grade el 634 ft. The sand fill was placed to approximately el 654 ft in each of the four interior bays of the DGB and extended horizontally at el 654 for 20 ft around the east, south, and west perimeters of the DGB. Along the north wall, where the DGB is adjacent to the turbine building, the 20-ft depth of sand extended for approximately 19 ft and was retained by a temporary wall to protect the turbine building. Placement of surcharge fill was initiated in January 1979 and reached the maximum 20-ft surcharge height in April 1979, when approximately 94% of the DGB structure was completed. The purpose of surcharging was to accelerate the settlement of the



rce: Applicant's draft hearing
uary 23-26, 1982 meetings
March 12, 1982)

Figure 2.8

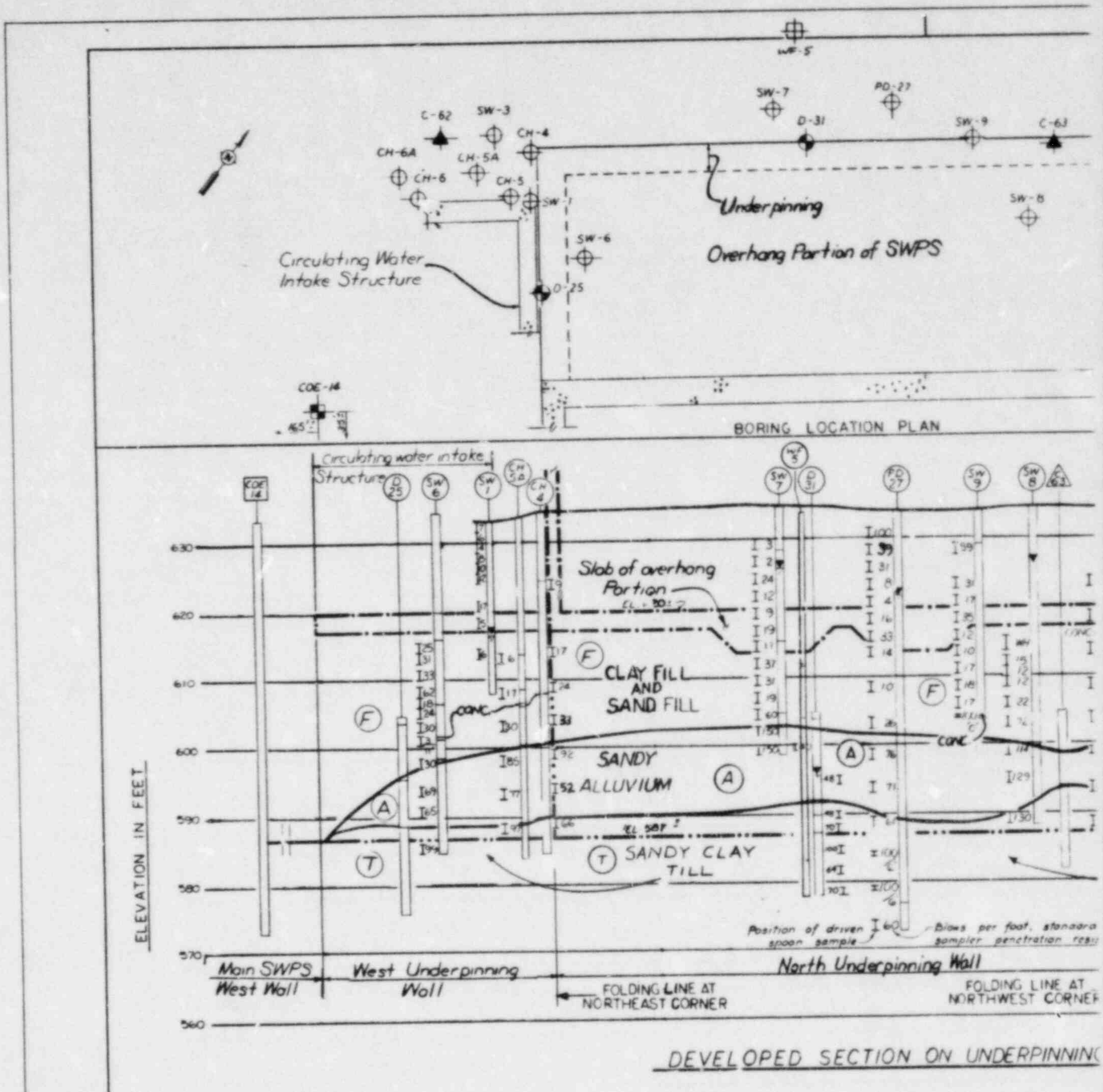


Figure 2.8 Profile of soils beneath the service water pump structure
testimony; same as Enclosure 13 of staff's summary of the
regarding remedial actions for structures on plant fill, d

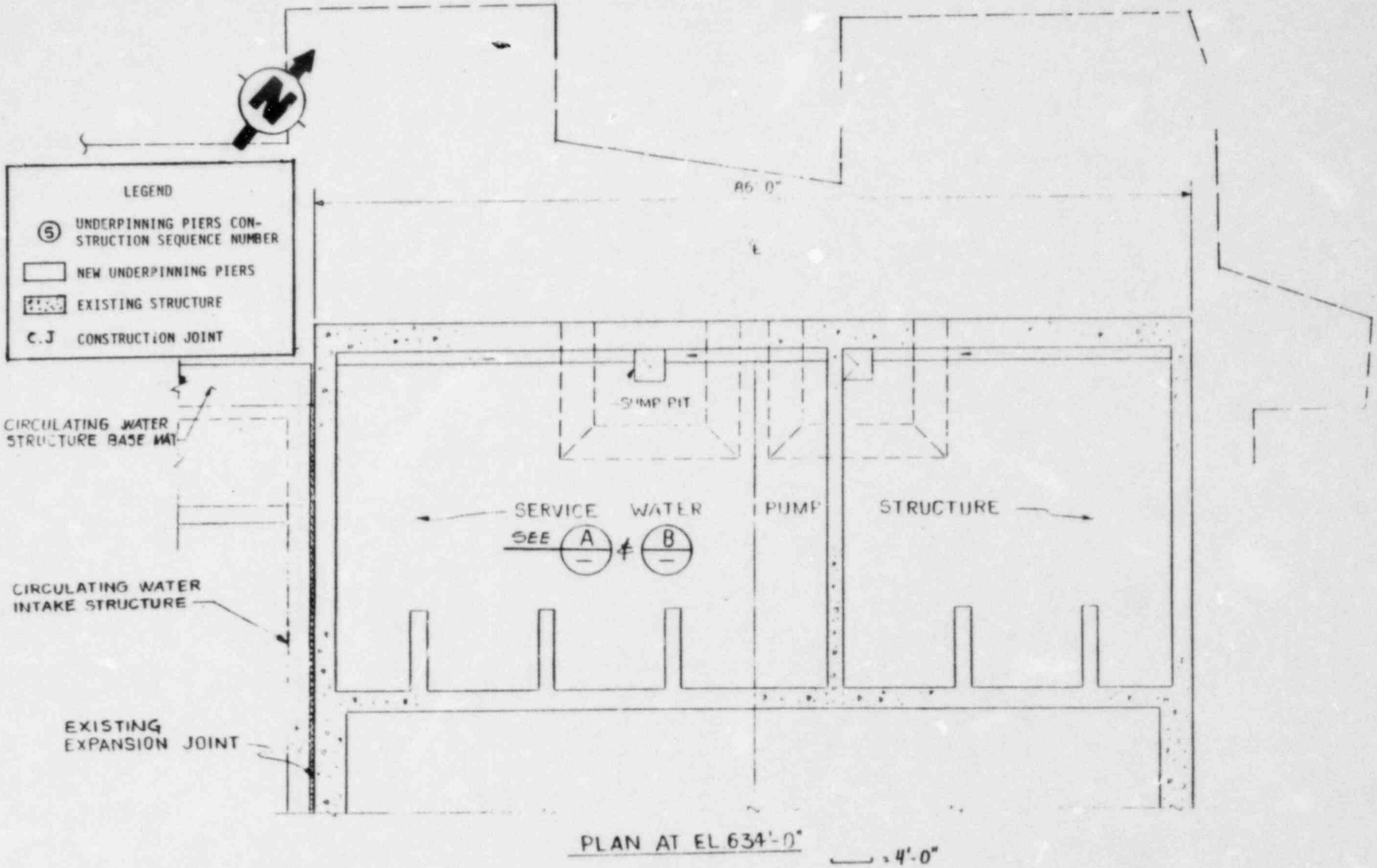


Figure 2.9 Service water pump structure underpinning plan and sections (Source: Pechtel drawing C-2035, Revision 2, provided to staff during staff audit on July 27-30, 1982; preliminary drawing not approved by applicant; issued for information only)

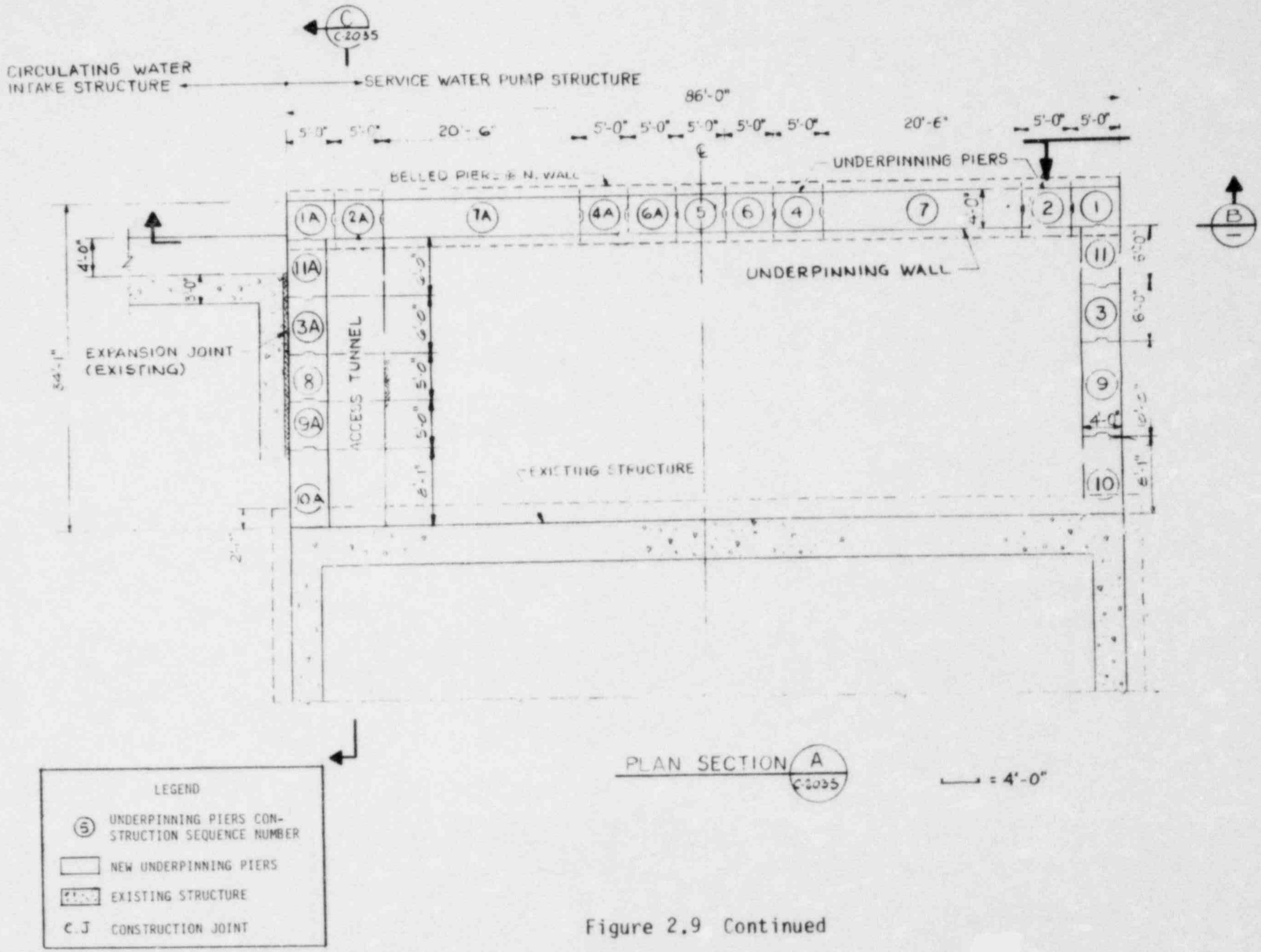


Figure 2.9 Continued

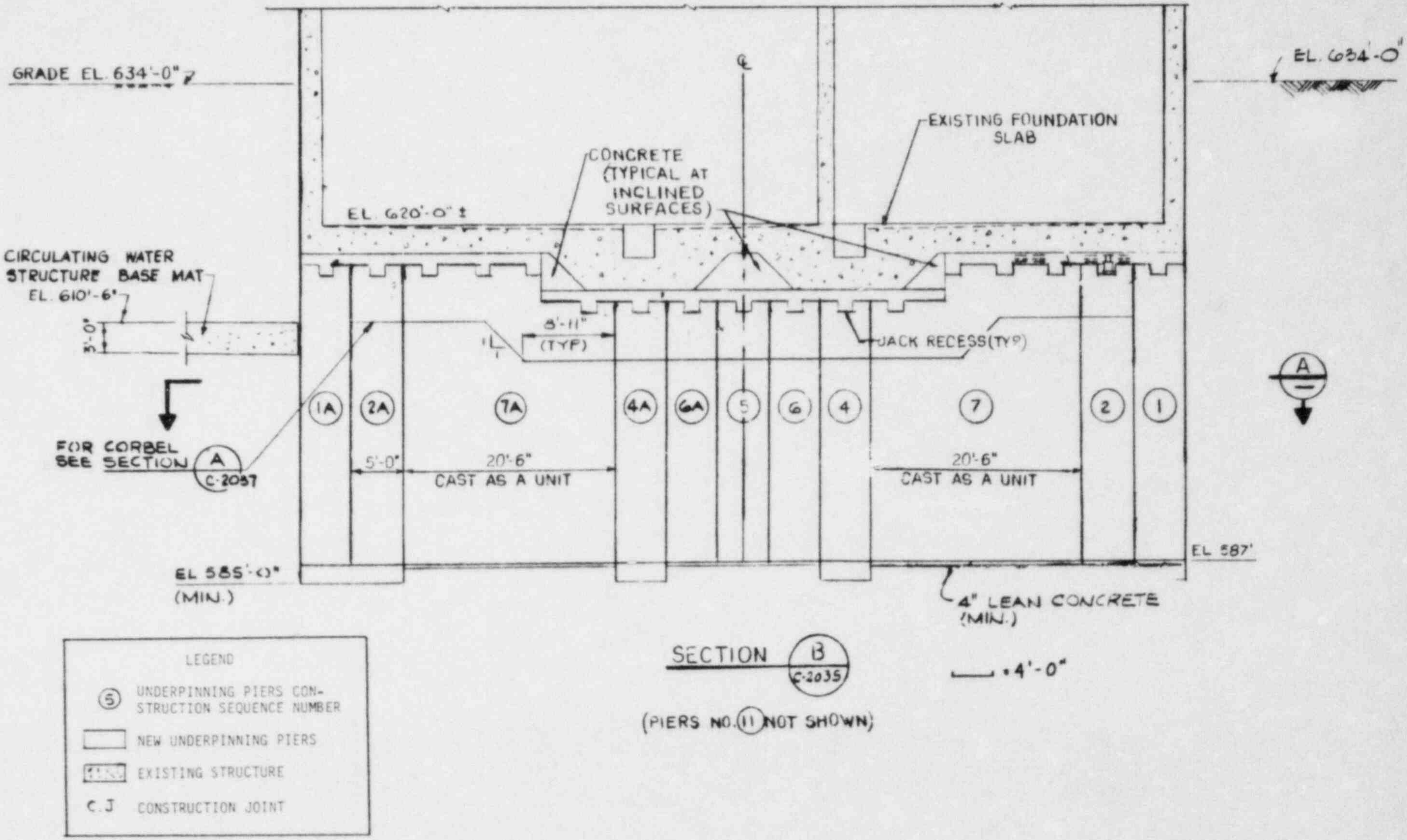


Figure 2.9 Continued

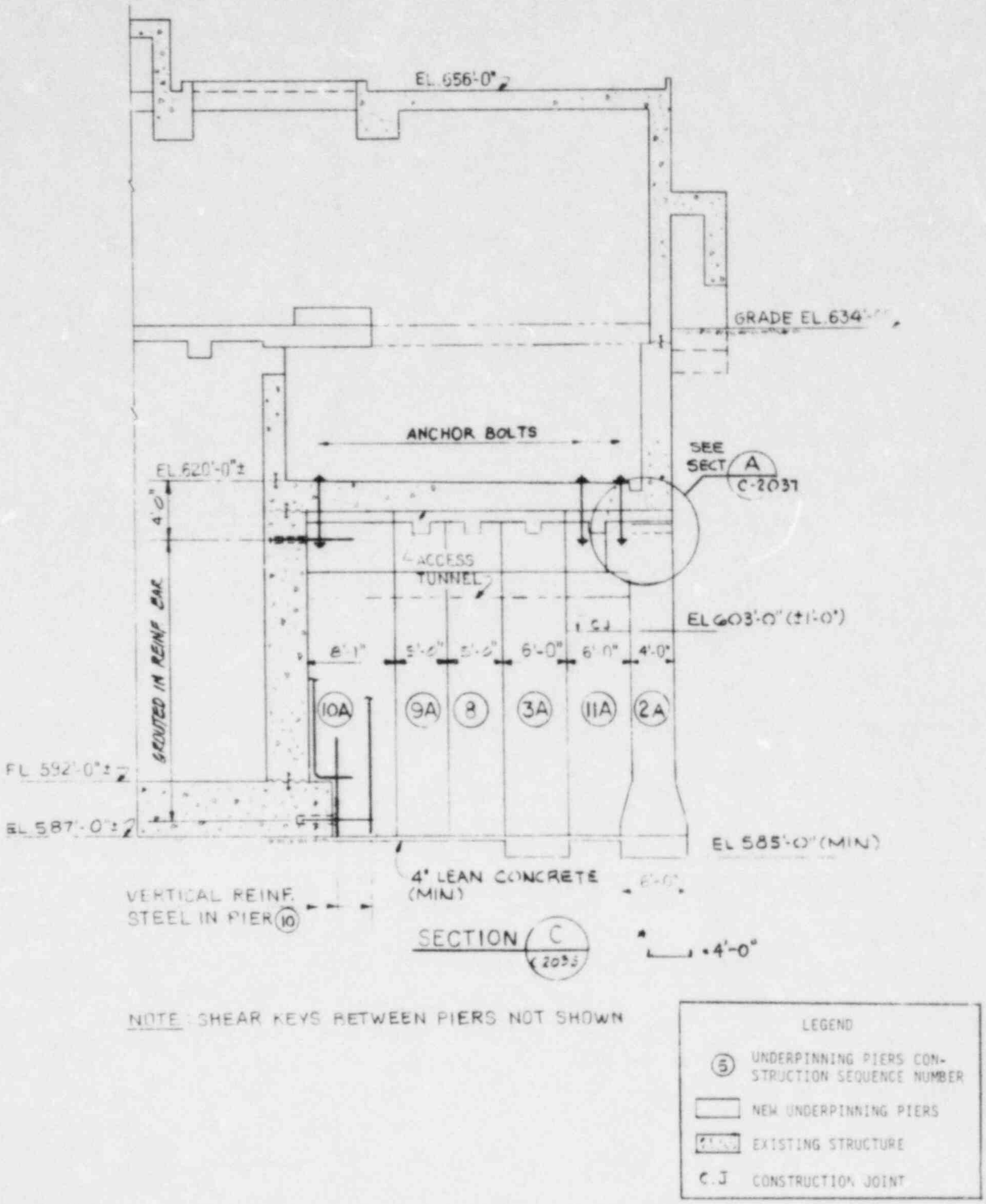


Figure 2.9 Continued

cohesive fill soils under a load that would produce vertical stresses at all depths in the fill in excess of those that would result during plant operation. Figure 2.10 shows the 20-ft sand surcharge in place in mid-1979.

The applicant recommended removal of the sand surcharge in mid-August 1979 after his favorable evaluation of the settlement and piezometer data recorded during the surcharge period. The largest amount of additional settlement recorded under the surcharge load occurred in the southeast corner of the DGB and reached 3.20 in., which resulted in a total settlement of 7.45 in. for this portion of the DGB structure. The settlements measured before, during, and after surcharging of the DGB are presented in FSAR Figures 2.5-124 through 2.5-126.

Surcharging was intended to resolve the uncertainties related to future settlements of the cohesive fill soils but was acknowledged to be limited in producing meaningful results in the granular fill soils. The concern for the safe operation of the Midland Plant because of the presence of the loose granular fill soils with potential for liquefaction has been addressed by the installation of the permanent dewatering system, which is discussed in Sections 2.5.4.4.4 and 2.5.4.5.5 of this SSER.

The staff concurs with the applicant that the surcharge program did accelerate the consolidation of the plant fill beneath the DGB and will result in smaller and more tolerable settlements during plant operation. However, the staff also recognizes that surcharging the essentially completed DGB structure did nothing to avoid the undesirable and large total and differential settlements that did result, with the accompanying concerns for structural degradation (warping and cracking of the reinforced concrete, see Section 3.8.3.4 of this SSER). The major objective of this review has been to correctly determine the amounts of total and differential settlements that have already occurred and will occur beneath the DGB. These basic settlement data are essential for use in a structural analysis that evaluates the effects of these settlement stresses, in conjunction with other required load combinations, to reach an engineering conclusion on the safe performance of the DGB.

Because of several piezometer and settlement readings recorded in the field during the time of surcharging, the staff had reasonable doubts as to whether the surcharge load was maintained long enough to cause the more compressible plant fill soils to reach secondary consolidation. To resolve this concern, the staff requested additional explorations in the surcharged plant fill to recover undisturbed soil samples of fill that could be laboratory tested for shear strength and compressibility characteristics. This work was completed in the spring of 1981 and results furnished to the staff in July 1981. The final conclusion reached by the staff following its evaluation of the laboratory results is that the future settlements (time frame of December 31, 1981 to December 31, 2025, FSAR Figure 2.5-127) identified by the applicant for use in his structural analysis of the DGB are sufficiently conservative. The future settlements identified cover the settlements calculated for the more compressible zones of cohesive fill soils that were recovered in the NRC-requested borings where attainment of 100% primary consolidation was shown not to have been achieved.

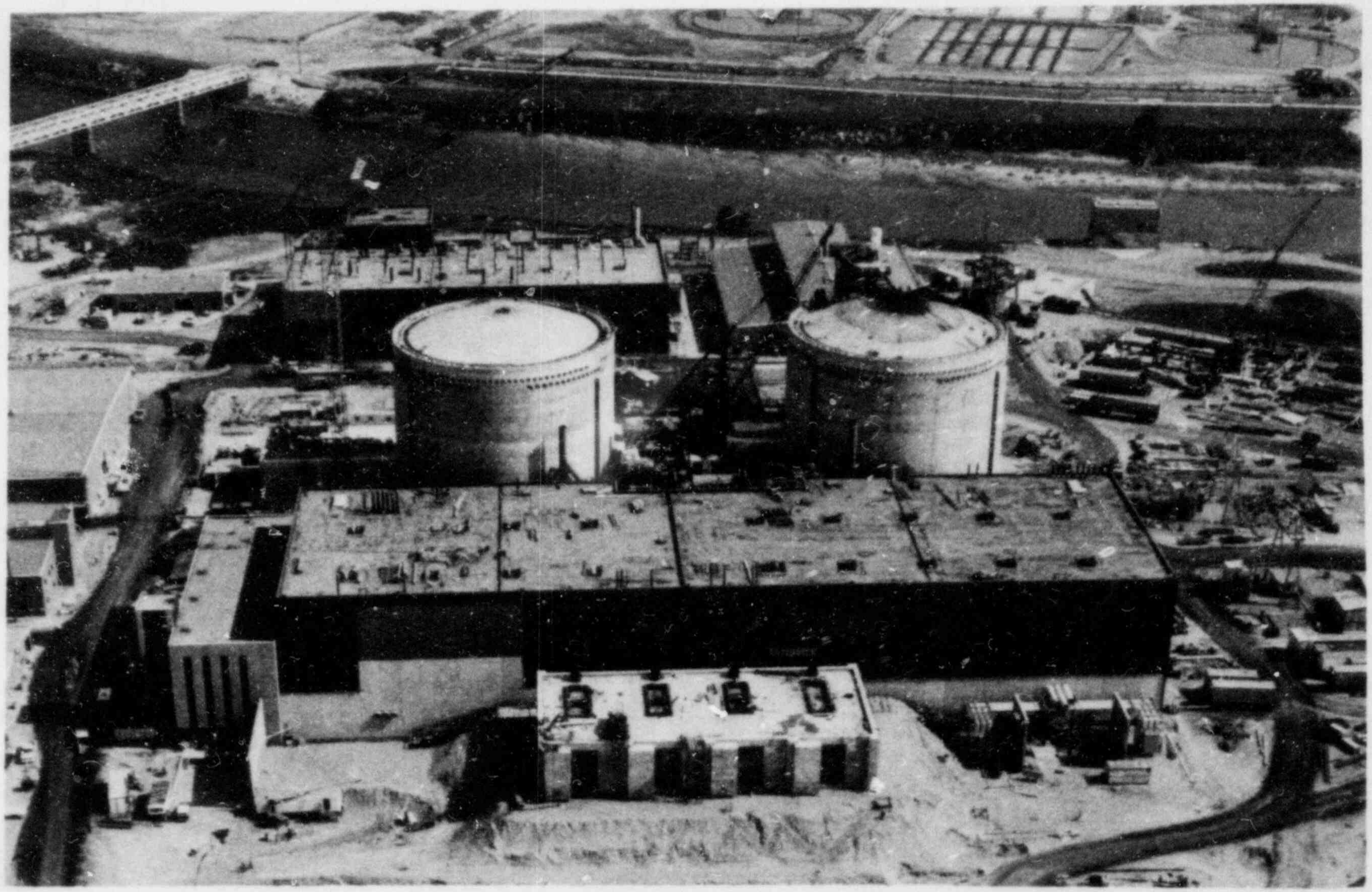


Figure 2.10 Photograph of Midland Plant showing 20-ft sand surcharge for DGB in place in mid-1979

The staff did not agree with the selection of settlement values, obtained before November 24, 1978, that were used by the applicant or with the applicant's indicated status of construction, which affected the flexibility of the integral footing and walls built before this time. These differences resulted from the staff's evaluation of the applicant's June 1, 1982 submittal, "Structural Stresses Induced by Differential Settlement of the Diesel Generator Building."

In response to these differences the applicant performed additional analyses of the effects of settlement and presented the results of this study at the July 27-30, 1982 design audit. The various time frames for which the effects of settlement have been analyzed and which were dicussed at the audit are shown in Table 2.6.

Table 2.6 Time frames for which effects of DGB settlement have been analyzed

Time frame	Case designation	Type of study	Status of construction
3/28/78 to 8/15/78 (before surcharging)	1A	Hand calculation	Top el 654 ft (east wall) Top el 656.5 ft (south wall
8/15/78 to 1/5/79 (before surcharging)	1B	Finite-element - computer	Top el 662 ft
1/5/79 to 8/3/79 (during surcharging)	2A	Finite-element - computer	Fully completed
8/3/79 to 12/31/2025 (after surcharging)	2B	Finite-element - computer	Fully completed

The staff has reviewed the calculations for Case 1A and the settlement input and results of the finite-element studies for Cases 1B, 2A, and 2B, which were provided during the July 1982 design audit. The staff's comments on the geotechnical aspects of these studies are as follows:

(1) The total and differential settlements that have been identified for Cases 1A, 1B, 2A, and 2B are correct and the staff is in agreement with the applicant on the status of construction at these various time frames. It should be noted that DGB construction began in October 1977 and settlement monitoring was initiated in March 1978.

(2) The staff is in agreement with the identified settlements tabulated for Cases 1B, 2A, and 2B. The staff has questioned, however, the manner in which the measured settlements were used as input for the structural analysis. The applicant employed a straight line "best fit" through a plot

of the measured settlement data to characterize the shape of the structure at the various stages of settlement for purposes of stress analysis. The basis for the applicant's approach is centered on judgment that the DGB structure is too rigid to bend into the apparent shape shown by the measured data and, because the straight-line fit is within his estimate of the error band of the measured settlements, it represents a realistic estimate of the building shape.

Although the staff agrees that the DGB is a rigid structure, it believes that a more appropriate characterization for a conservative analysis would be to require that the building follow a shape consistent with the measured settlements. This matter is discussed further in Section 3.8.3.4 of this SSER in the context of the staff conclusions with regard to the structural adequacy of the DGB considering both the applicant's analysis and independent analyses being performed by staff consultants.

2.5.4.4.3 Surcharging of the Borated Water Storage Tank Foundations

As discussed in SER Section 1.12.8, the foundations of the two borated water storage tanks (BWSTs) were constructed between July 1978 and January 1979. Erection of the tanks was completed by December 1979. To demonstrate the adequacy of the plant fill supporting the tanks, the applicant filled the tanks with water in October 1980 and monitored the resulting foundation settlements.

In January 1981, the applicant reported differential settlements between the ring wall foundations and the outside portions of the valve pits. The applicant's investigation indicated cracks in the ring beam of the Unit 1 tank as wide as 0.063 in., and 0.035 in. wide in the ring beam of the Unit 2 tank. The applicant concluded that the observed differential settlements were largely due to the design errors discussed in Section 3.8.3.3 of this SSER. However, on the basis of the results of the soils investigations of the fill in the tank farm area, the results of plate load tests, and the observed total and differential settlements that did occur, the staff has concluded that the differential settlement problem was primarily the result of inadequately compacted fill.

To correct the BWST foundation problem, the applicant proposed the following three actions:

(1) Surcharge the valve pits to reduce the amount of differential and future settlements. This was done over a 4-month period and successfully completed by February 1982.

(2) Construct a new reinforced concrete ring beam around the periphery of the existing cracked ring; the design basis for this beam is discussed in Section 3.8.3.3 of this SSER.

(3) Relevel to the original construction tolerance the tank (Unit 1) that had experienced the largest settlements. Staff evaluation of the proposed releveling program is in Section 3.8.3.3 of this SSER.

On the basis of the results of field settlement records and design reports provided by the applicant, the staff agrees that future differential settlements will be small because of the surcharging that has been completed for both the valve pits and ring foundations. The future settlements estimated to occur during plant operation have been enveloped for use in the structural analysis discussed in Section 3.8.3.3 of this SSER for the new ring beams. The staff finds this approach acceptable.

The applicant has committed to providing a Technical Specification for long-term settlement monitoring during plant operation and to providing FSAR documentation of the as-built conditions for the new ring beam foundations and releveling operations, once they are completed.

The staff concludes, based on the actions discussed above and in Section 3.8.3.3, that the applicant's remedial program for the BWST is acceptable.

2.5.4.4.4 Permanent Dewatering

To eliminate concerns about the liquefaction potential of the inadequately compacted loose granular fill materials, the applicant has installed a permanent dewatering system.

The staff's assessment of liquefaction potential is provided in Section 2.5.4.5.5 of this SSER. The staff's evaluation of the proposed permanent dewatering system was presented in SER Section 2.4.6.2 and is further discussed in Section 2.4.6.2 of this SSER.

During operation of the permanent dewatering system, measurement of soil fines in the collected seepage water is required. If measured fines larger than 0.005 mm exceed 10 ppm, the applicant is required to determine which well or wells are causing the loss of fines and to stop pumping the well(s). If necessary, the problem well(s) will be repaired or replaced.

2.5.4.4.5 Excavation and Backfill

This section summarizes the settlement problem beneath seismic Category I piping and describes the foundation treatment of the plant fill soils supporting these underground piping systems.

The soil profiles developed along the alignment of safety-related underground piping show predominantly stiff to hard clay fill soils with some highly variable layering of soft clays and loose sands. FSAR Figures 2.5-100 and -101 show typical profiles with subsurface conditions based on borings completed near the buried piping.

To permit an assessment of the condition of the underground piping because of the plant fill problem, internal profiling of some of the buried pipes was done to establish pipe deflection (settlement) profiles. The results of the profiling indicated that the present pipe invert elevations have maximum deviations from 6 to 21 in. below the originally intended design invert elevations. The majority of these deviations are in the range of 9 to 11 in. The allowable placement tolerances for installing the pipe in the field during construction were specified at ± 2 in. from the established design invert elevations. Allowing for the lower tolerance of - 2 in. during installation (which was reported to have been verified in the field) would indicate that pipe settlements of 4 to 19 in. have occurred.

Using the actually observed settlement records of a series of markers (Borros anchors) in the vicinity of the buried piping, the applicant has estimated future settlement for the piping system to be a maximum of 3 in. during the 40-year period of plant operation. The staff agrees that the estimated 3-in. maximum settlement is a conservative upper limit provided no additional significant load is placed over the piping. The applicant has committed to providing a Technical Specification by the fall of 1982 that will include the control measures to be required in restricting placement of heavy loads over buried piping and conduits.

The applicant has committed to providing a plan for addressing a staff concern that arises from a modification to the originally proposed freezewall crossing design. The freezewall is a temporary barrier provided for construction purposes to prevent groundwater from entering the underpinning excavations for the control tower, EPAs, and the FIVP. This freezewall crossing modification has the potential for creating differential settlements along the piping or conduit. This concern will be addressed in a later supplement to the SER after the applicant submits information that describes the crossing modification, details on surcharging the piping and conduit foundations during ground freezing, and the monitoring records on heave and/or settlement. Details on backfilling the excavations at the freezewall crossings are also to be provided by the applicant.

Some of the piping lines have already been relieved of stresses resulting from differential settlement by excavating down to the installed pipes, cutting the lines, and then refitting the pipes. The extent of this completed work and future planned rebedding work are shown in Figure 2.11 and is further discussed in Section 3.9.3.1 of this SSER. Figure 2.11 also shows where excavation, pipe replacement, and backfill for the 36-in.- and 26-in.-diameter service water pipes are to be completed just north of the SWPS and circulating water intake structure.

Excavation, rebedding, and backfilling for the 26-in. service water lines just north of the SWPS and CWIS will be carried out because the loose sand fill in this area, which is indicated by low SPT blow counts, has potential for liquefaction under a safe shutdown earthquake loading condition. If failure of the non-Category I permanent dewatering system were assumed, there might not be sufficient time to either repair the dewatering system and/or shut down the plant because of the close proximity of this problem area to the cooling pond. Recharge of the groundwater in this area has been demonstrated to occur rapidly (within approximately 3 days).

The applicant has committed to excavating the loose sand fill down to el 610 ft within a braced excavation that will also temporarily support the existing service water piping, which has an invert elevation at approximately el 626 ft. Backfill of this braced trench excavation will consist of K-KRETE, a commercial brand name for a fly ash concrete mix with low strength (minimum compressive strength of 250 psi). The K-KRETE is to be placed to a level of 1 ft above the top of the pipe using the applicable portions of Bechtel concrete specifications C-230 and C-231.

Concerns about differential settlement have been addressed by requiring the service water piping to be encircled with 6-in.-thick polyethylene planks that

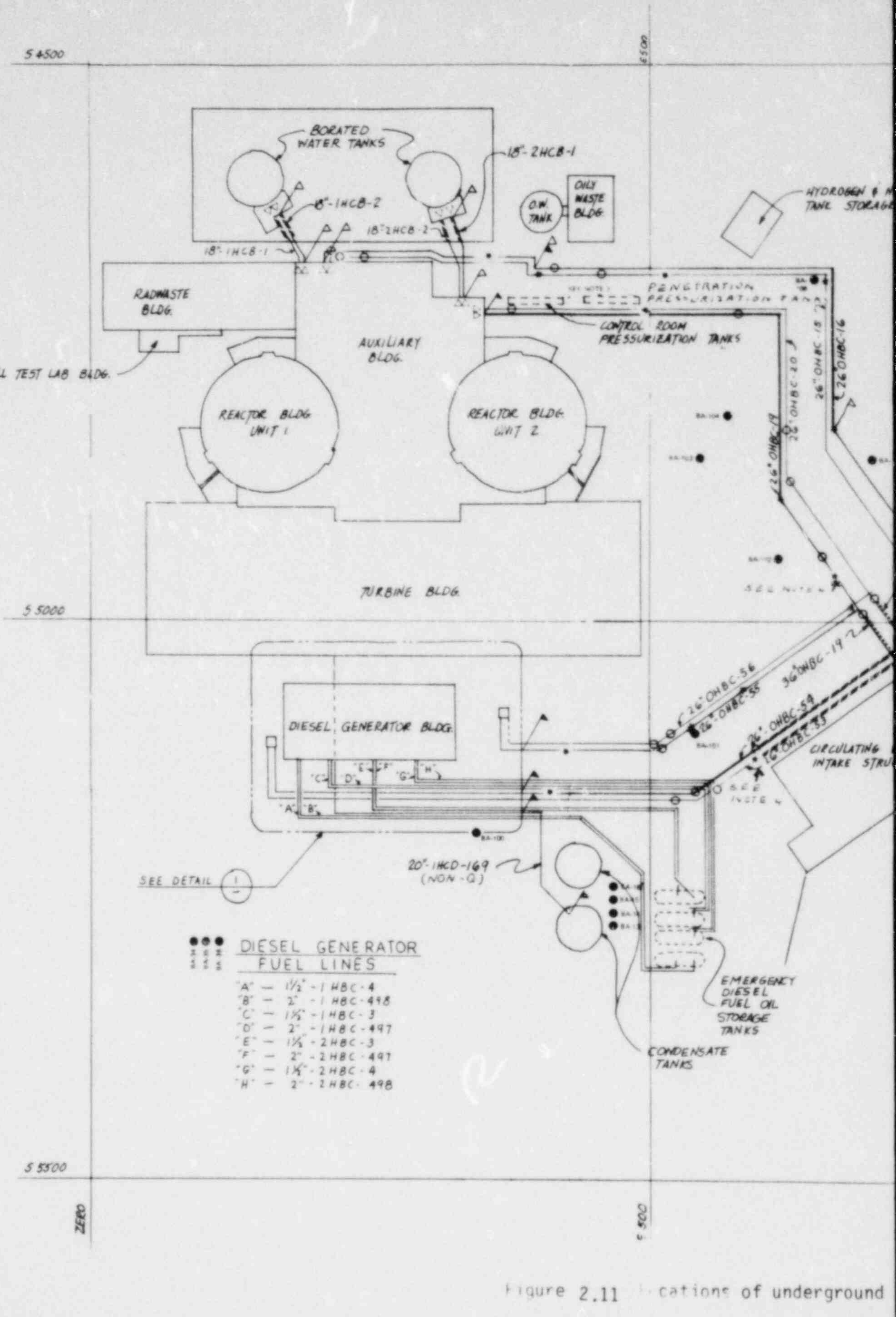


Figure 2.11 Locations of underground

are commercially named Ethafoam 220. The length of piping to be wrapped with this compressible product within the K-KRETE is 40 ft and spans from the portion of piping on the full depth of K-KRETE (to el 610 ft) to where it is supported on the existing clay fill soils. This transition length has been established in an analysis of pipe stresses where a 3-in. differential settlement over this length has been assumed and shown to be tolerable. These analyses are discussed in Section 3.9.3.1 of this SSER.

To verify that actual differential settlements do not exceed the assumed design values, the staff has required the placement of additional settlement markers at each end of the transition lengths at four locations, as shown on Figure 2.11 of this SSER. Discussions on future settlement monitoring of underground piping is presented in Section 2.5.4.6 of this supplement.

The above discussion on excavation and backfill details for the 36-in.- and 26-in.-diameter service water pipelines is based on information presented by the applicant at the July 27-30, 1982 design audit. This information will be formally documented in an FSAR amendment in the near future. The staff plans to review the formal FSAR submittal but does not, at this time, feel an additional supplement to the SER will be necessary on this issue.

On the basis of the information provided by the applicant, the staff concludes that the proposed excavation and backfill remedial fix is a conservative and acceptable solution to the plant fill problem in this area, and, once properly carried out in the field, will provide a stable and safe foundation for the underground piping.

2.5.4.5 Foundation Stability

2.5.4.5.1 Bearing Capacity

The following discussions on the adequacy of the foundations to resist bearing-type failure are based, in part, on information received at the July 27-30, 1982 design audit. The staff anticipates that this information from the audit will be formally documented in an FSAR amendment in the near future. The staff plans to verify the accuracy of the information documented in the formal FSAR submittal but does not feel an additional supplement to the SER on this topic will be necessary.

The applicant has estimated that the maximum static bearing pressures for seismic Category I structures that will occur will be on the very stiff to hard clay natural soils beneath the underpinned CT and EPAs. The gross bearing pressures for these structures are 15 ksf and 11 ksf, respectively, for both dead and live loads. The maximum gross static bearing pressure for structures founded on the plant fill is 4.4 ksf at the DGB.

The maximum gross bearing pressures under the addition of dynamic loading also occur at these structures and are 20.6 ksf for the CT, 19.8 ksf for the EPA, and 5.7 ksf for the DGB.

The applicant has calculated factors of safety against bearing-capacity-type failure with the factor of safety defined as the ratio of the net ultimate

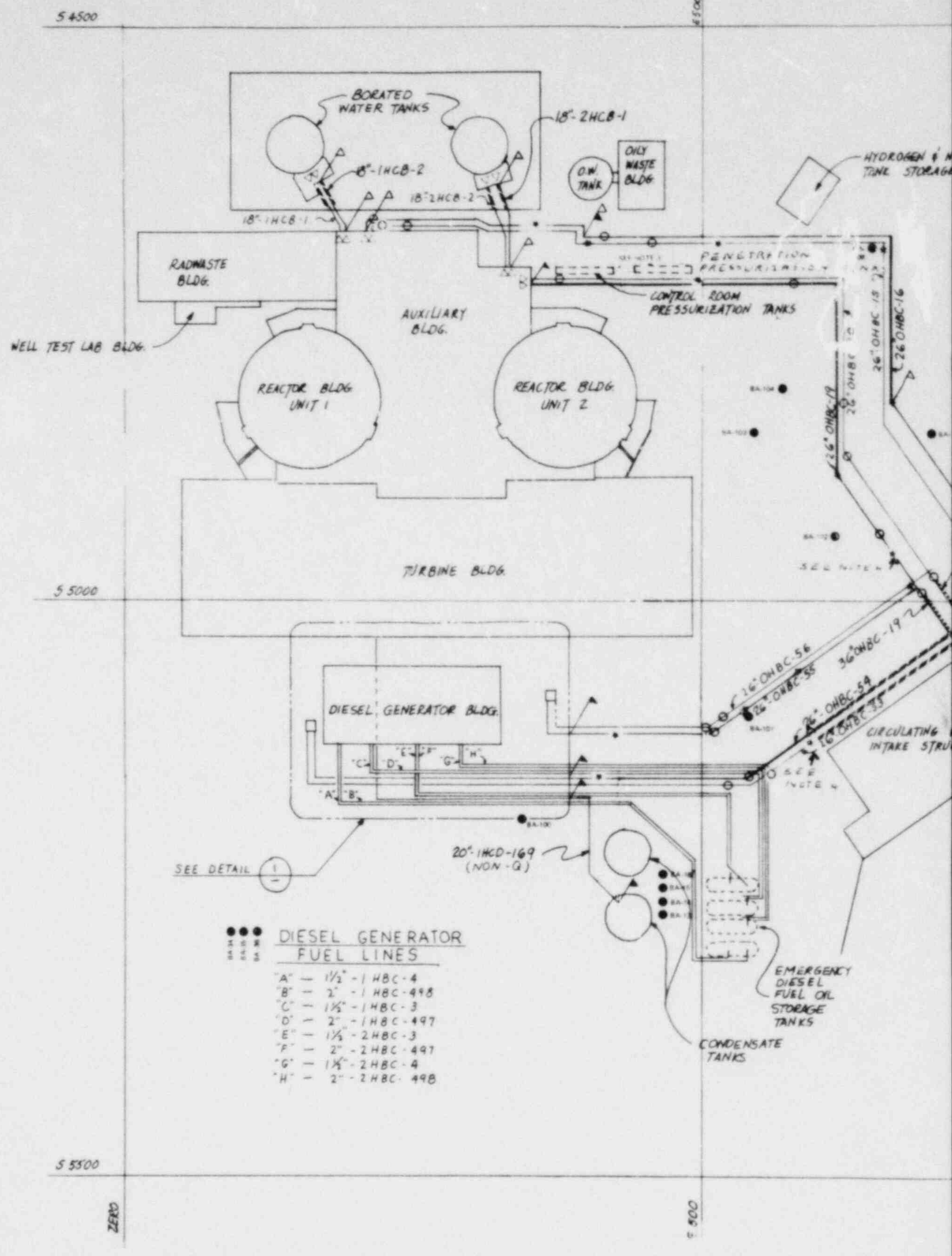
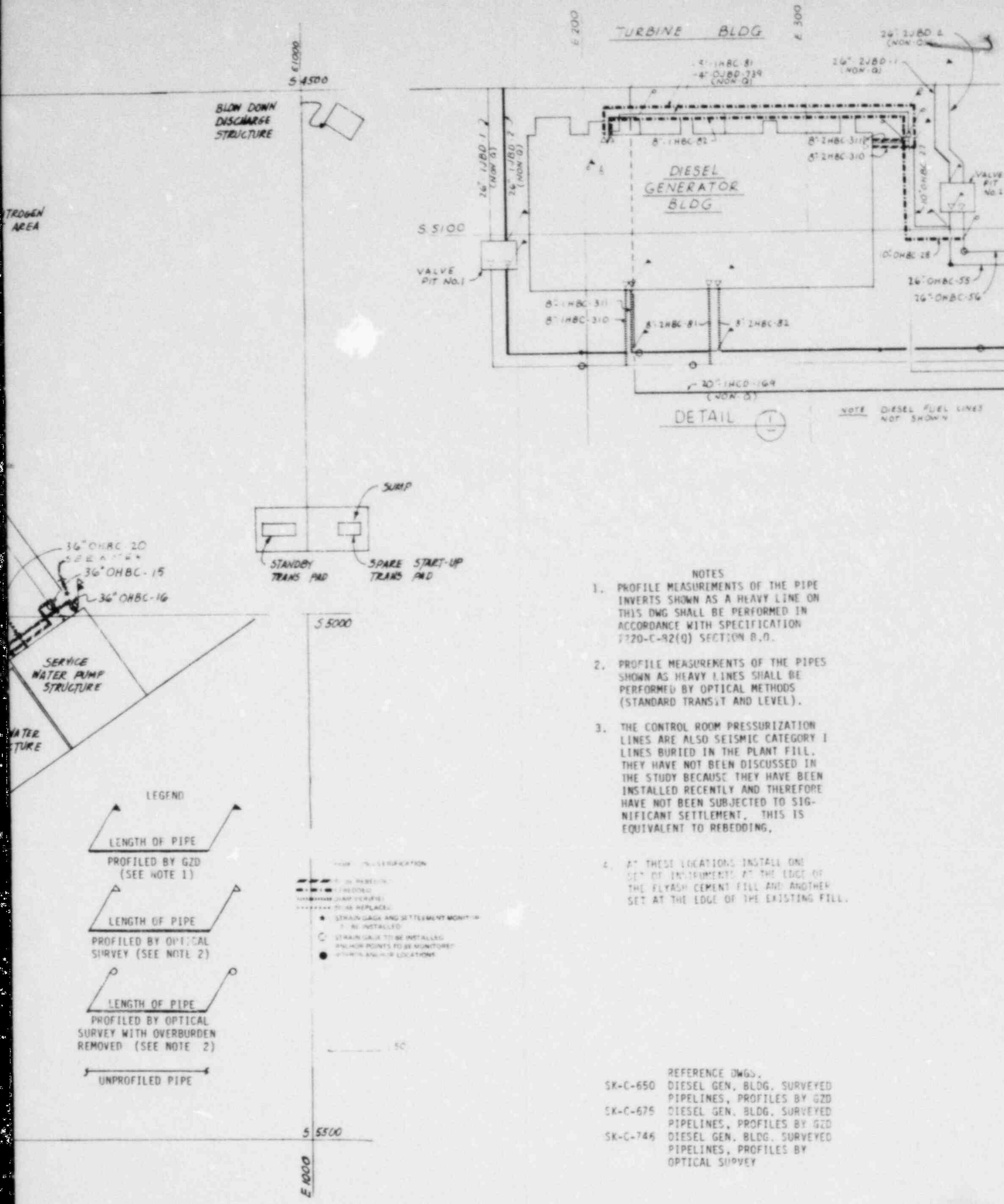


Figure 2.11 [illegible]cations of underground



Q-listed pipes (Source: Applicant's letter of September 3, 1982) Figure 2.11

are commercially named Ethafoam 220. The length of piping to be wrapped with this compressible product within the K-KRETE is 40 ft and spans from the portion of piping on the full depth of K-KRETE (to el 610 ft) to where it is supported on the existing clay fill soils. This transition length has been established in an analysis of pipe stresses where a 3-in. differential settlement over this length has been assumed and shown to be tolerable. These analyses are discussed in Section 3.9.3.1 of this SSER.

To verify that actual differential settlements do not exceed the assumed design values, the staff has required the placement of additional settlement markers at each end of the transition lengths at four locations, as shown on Figure 2.11 of this SSER. Discussions on future settlement monitoring of underground piping is presented in Section 2.5.4.6 of this supplement.

The above discussion on excavation and backfill details for the 36-in.- and 26-in.-diameter service water pipelines is based on information presented by the applicant at the July 27-30, 1982 design audit. This information will be formally documented in an FSAR amendment in the near future. The staff plans to review the formal FSAR submittal but does not, at this time, feel an additional supplement to the SER will be necessary on this issue.

On the basis of the information provided by the applicant, the staff concludes that the proposed excavation and backfill remedial fix is a conservative and acceptable solution to the plant fill problem in this area, and, once properly carried out in the field, will provide a stable and safe foundation for the underground piping.

2.5.4.5 Foundation Stability

2.5.4.5.1 Bearing Capacity

The following discussions on the adequacy of the foundations to resist bearing-type failure are based, in part, on information received at the July 27-30, 1982 design audit. The staff anticipates that this information from the audit will be formally documented in an FSAR amendment in the near future. The staff plans to verify the accuracy of the information documented in the formal FSAR submittal but does not feel an additional supplement to the SER on this topic will be necessary.

The applicant has estimated that the maximum static bearing pressures for seismic Category I structures that will occur will be on the very stiff to hard clay natural soils beneath the underpinned CT and EPAs. The gross bearing pressures for these structures are 15 ksf and 11 ksf, respectively, for both dead and live loads. The maximum gross static bearing pressure for structures founded on the plant fill is 4.4 ksf at the DGB.

The maximum gross bearing pressures under the addition of dynamic loading also occur at these structures and are 20.6 ksf for the CT, 19.8 ksf for the EPA, and 5.7 ksf for the DGB.

The applicant has calculated factors of safety against bearing-capacity-type failure with the factor of safety defined as the ratio of the net ultimate bearing capacity to the net bearing stress. The net bearing stress is equal to the applied gross load intensity minus the depth of embedment times the unit weight of the soil above the bottom of the foundation footing. The lowest calculated factors of safety are 3.4 and 2.4 at the underpinned control tower for static and dynamic loading conditions, respectively.

On the basis of its review of the information provided by the applicant in the FSAR and technical reports, including the design audit information, the staff concludes that the resulting margins of safety against bearing-capacity-type failure are sufficiently conservative and are acceptable.

2.5.4.5.2 Vertical Movement

Control Tower

The downward movement of the south end of the CT relative to the south end of the spent fuel pool in the auxiliary building has been 0.24 in. during the period July 1978 through August 1981. Because the CT was completed more than a year before settlement observations were begun in July 1978, and because the largest settlements of the poorly compacted fill are likely to have occured early in the loading, in the staff's judgment it is reasonable to expect that differential settlements of 0.5 to 1.0 in. or more may have occurred from the beginning of loading to the present.

Electrical Penetration Areas

The downward movement of the east end of the east EPA relative to the adjacent control tower has been 0.2 in. during the period July 1978 through August 1981. There has been negligible differential settlement between the west end of the west EPA and the adjacent control tower.

The total recorded settlement of the control tower and the EPAs for the period July 1978 through January 1982 has been 0.5 to 0.7 in., as shown in FSAR Figure 2E.1-1. The settlement between the start of construction and July 1978 was not measured.

Auxiliary Building

The applicant has estimated the differential settlements that will occur between the new underpinning wall and the auxiliary building for a 40-year plant life to be

(1) maximum differential settlement of the CT relative to auxiliary building — 0.25 in.

(2) maximum differential settlement of auxiliary building relative to the CT — 0.25 in.

The staff considers the settlement behavior of (1) to be the more reasonable assumption because it allows for greater settlement of the CT and finds the 0.25-in. estimate for (1) to be reasonable. Both estimates have been used in the analysis of the structure to demonstrate that the FSAR loading conditions plus these differential settlements will not cause unacceptable stresses.

Steel plates are to be added to the slab at el 659 ft in the auxiliary building, after underpinning is complete, to strengthen that location where high stresses have been calculated.

Service Water Pump Structure

The maximum measured differential settlement of the overhang of the SWPS relative to the portion founded on till has been about 0.2 in. The total settlement of the SWPS has been about 3/8 in. and is shown in FSAR Figure 2E.1-27.

The fact that the differential settlement noted above for the SWPS is small indicates that either (1) the poorly compacted fill under the overhang has not settled significantly or (2) the overhang is being supported as a cantilever and did not follow the fill settlement, which would mean a gap may be found beneath the overhang during underpinning.

Settlements predicted by the applicant after completion of the underpinning wall of the SWPS overhang relative to the portion currently on the till are 0.1 to 0.2 in.

The staff considers these estimates of differential settlements for the underpinned SWPS reasonable and acceptable.

Diesel Generator Building

The settlement history of the DGB and the evaluation done by the staff regarding the impact of settlement are discussed in Section 2.5.4.4.2 of this SSER. The settlement history of the DGB is shown in FSAR Figures 2E.1-6 through 2E.1-12.

Borated Water Storage Tanks

The settlement history, the applicant's proposed remedial measures, and the evaluation done by the staff regarding future foundation stability of the BWSTs are discussed in Section 2.5.4.4.3 of this SSER. The settlement history of the BWSTs is shown in FSAR Figures 2E.1-17, -18, -20, and -21.

Reactor Containment Buildings

The reactor containment buildings are founded on the overconsolidated, very stiff to hard, natural clay soil. Total settlements based on the adopted low recompression indices ranging from 0.002 to 0.006 are conservatively estimated to be approximately 2.4 in. and 2.3 in. for the containments for Units 1 and 2, respectively. These estimated settlements include a settlement of 0.6 in. resulting from lowering of the groundwater to el 590 ft by the permanent dewatering system. As shown in FSAR Figure 2E.1-2, the average settlement actually recorded in the field up to January 1982 was approximately 0.75 in. for both containment buildings, with the maximum settlement of 1.1 in. occurring beneath the Unit 1 containment.

The staff considers the estimated settlements for the reactor containment buildings to be conservative and acceptable.

Underground Piping

The settlement of seismic Category I underground piping is discussed in Section 2.5.4.4.5 of this SSER.

2.5.4.5.3 Strain and Horizontal Movement

No measurements have been made of the horizontal movement of structures to date, but settlements that may take place during underpinning of the CT and EPAs may cause the top of these structures to move southward toward the turbine building. Monitoring instruments are being installed to measure potential horizontal movements between all adjoining structures during underpinning.

In addition, strains that may develop in the SWPS and auxiliary building will be measured at critical locations. Strain monitoring is discussed in Section 2.5.4.6 of this SSER. The pertinent references describing the program for monitoring differential horizontal movements between adjacent structures are identified in Section 2.5.4.6.1.1 of this SSER. Acceptance criteria on differential horizontal movement have not been required; however, these measurements will be evaluated in conjunction with the required differential settlement and strain monitoring records where criteria have been established.

The staff considers the strain and horizontal movement monitoring program (locations, frequency of readings, etc.) that has been proposed during underpinning operations by the applicant to be acceptable.

2.5.4.5.4 Lateral Loads

The walls of seismic Category I structures below plant-grade el 634 ft were designed to resist at-rest lateral earth pressures using the equivalent fluid pressure concept. The adopted design equivalent fluid unit weights are presented in FSAR Table 2.5-15. The adopted fluid pressures are equivalent to an at-rest lateral earth pressure coefficient of 0.5 for sand soils and approximately 0.7 for clay soils. Walls were conservatively designed, allowing for full hydrostatic groundwater pressures from a water level at el 627 ft in combination with safe shutdown earthquake loading.

For dynamic loading conditions, the Seed-Whitman (1970) simplified procedure for approximating the Mononobe-Okabe (1926, 1929, respectively) approach was used in design. A peak horizontal ground surface acceleration of 0.12 g was used for estimating inertial forces.

The staff concludes that the methods used to estimate lateral earth pressures on seismic Category I subsurface walls are conservative and acceptable and are in accordance with current state-of-the-art engineering practice.

2.5.4.5.5 Liquefaction Potential

In February 1978, during its review of the Midland FSAR, the staff forwarded Request 362.2 to the applicant asking for documentation on the method that was used to remove loose natural sands (sands with less than 75% relative density) from the foundations of safety-related structures, as the applicant had committed to do in the Preliminary Safety Analysis Report (PSAR). In his response to Request 362.2, the applicant was unable to furnish documentation on the field operations completed to remove the loose natural sands. Instead, the applicant provided the results of boring explorations performed in August and September 1978 and in 1979 (these borings were completed after site area fill had been placed to plant grade) that did not indicate the presence of loose natural sands beneath safety-related structures. On the basis of the results of all completed exploration programs, including the later 1978 and 1979 standard penetration test (SPT) data, the applicant concluded that the natural sands existing in the plant area have relative densities greater than 75%.

The two methods for analyzing safety factors with respect to liquefaction of the natural granular soils that the applicant has presented in FSAR Section 2.5.4.8 use the results of SPT blow counts. On the basis of the high SPT values recorded in the natural soils in the extensive subsurface investigation programs that have been completed, the applicant has concluded that there are no liquefiable natural granular soils beneath safety-related structures at the Midland site. The staff has reviewed these data and concurs in this finding.

In the subsurface exploration programs completed in late 1978 and early 1979, following discovery of the DGB settlement problem, potentially liquefiable granular soils were discovered in the structural backfill placed beneath certain seismic Category I structures and underground utilities. The affected facilities included the DGB, EPAs, RBA, the cantilevered portion of the SWPS, and a portion of the service water piping.

In July 1979, the applicant reported the findings of liquefaction studies using the results of the 1978 and 1979 explorations. In these studies the applicant had adopted a peak ground surface acceleration of 0.12 g, and a groundwater level at el 627 ft (the operating level of the cooling pond) and conservatively adopted a magnitude 7.5 earthquake for relating cyclic stress ratio causing liquefaction to SPT values. Of the three areas investigated for liquefaction, the applicant concluded that liquefaction could be a problem at the DGB, was unlikely at the RBA, and was not a problem at the auxiliary building CT area. To alleviate his concerns about liquefaction potential, the applicant ultimately chose to provide a permanent dewatering system, which is discussed in Section 2.5.4.4.4 of this SSER.

In May 1980, the staff's consultant, the U.S. Army Corps of Engineers, concluded an independent liquefaction analysis using the Seed-Idriss (1971) simplified method. In the Corps of Engineers study, a groundwater level of 610 ft was selected based on (1) the applicant's stated intention to maintain, by pumping, groundwater below this elevation; (2) a magnitude 6 earthquake; and (3) a peak ground surface acceleration of 0.19 g. The results of the study indicated that fill soils are safe against liquefaction for earthquakes that would produce a peak ground surface acceleration up to 0.19 g if the groundwater were maintained below el 610 ft. A minimum factor of safety equal to 1.5 was met using the simplified method of analysis.

The areas of the site where it is necessary to maintain the groundwater level below el 610 ft are the DGB area and the RBA. The problems with loose granular backfill soils previously identified in other areas (EPAs, the cantilevered portion of the SWPS, and service water piping) are acceptably resolved by the proposed underpinning or by excavation and backfill remedial measures.

The staff concurs with the applicant's finding that the permanent dewatering system will eliminate the potential for liquefaction in the granular backfill soils identified above. An acceptable margin of safety against liquefaction potential is available for an earthquake with a peak ground surface acceleration up to 0.19 g, which is more severe than the earthquakes used to establish the site-specific response spectra at the top of fill, provided the groundwater is maintained below el 610 ft. Sections 2.4.6.2 of the SER and this SSER discuss the permanent dewatering system and the staff's basis for reasonable assurance that the groundwater will be maintained below el 610 ft during plant operation.

2.5.4.5.6 Dynamic Loading

Section 2.5.2 of the SER provides the staff's assessment of the safe shutdown earthquake (SSE) and operating basis earthquake (OBE) to be used for the design of the Midland Plant. The site-specific response spectra approach was used and was independently checked by the staff's consultant, utilizing the SHAKE computer code, which provides a one-dimensional wave propagation analysis to study possible local amplification effects of the earthquake ground motion. The independent study also evaluated the effects of variations in plant fill properties (stiffnesses) and the effects of variations in input accelerograms on the ground motion. The effects of both these variations were shown to have significant impact on amplification of the earthquake ground motion.

The independent study also identified a problem with the applicant's input into his wave propagation analysis. The problem was that acceptable input values of shear wave velocities were being incorrectly reduced by the computer code to unacceptably low values for the plant fill. This problem was corrected and resulted in better agreement with results of the site-specific response spectra approach. A major conclusion of the staff consultant's study was that the amplification for the top of plant fill over that at the top of natural till soils, using the site-specific response spectra approach, is more conservative than the spectrum developed by application of the SHAKE results.

A check was made on the seismically induced settlement values that were estimated by the applicant to range from 0.25 in. to 0.50 in. (These settlement values are provided in Table 4-1A in "Responses to NRC Requests Regarding Plant Fill"*). The staff concludes that these settlements are acceptable for design and consider them to be an upper bound for the design earthquake loading conditions.

The applicant has estimated average shear modulus (G) values of 7700 ksf for the glacial till and 1,510 ksf for the plant fill in the shear strain range of 10^{-2} to 10^{-3}%. The staff considers these values to be reasonable and acceptable for use in dynamic analysis and concludes that the applicant's decision to allow ± 50% variation in the soil spring constants is conservative.

2.5.4.6 Instrumentation and Monitoring

2.5.4.6.1 Underpinning

The following monitoring measurements and criteria are provided for underpinning of the auxiliary building area and SWPS.

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*This document was incorporated into the FSAR by reference by Amendment 72.

2.5.4.6.1.1 Measurements

The construction controls to be required during underpinning and the types of instrument to be used are listed below; they are further described in the referenced documents and Bechtel drawings.

Auxiliary Building

(1) Total and differential settlements of the CT, EPAs, and FIVPs and total settlement of the auxiliary building: Dial gages and LVDTs (linear variable displacement transducers) on deep-seated bench marks and on supports between adjacent structures. References: Drawings C1490, dated June 21, 1982; C1491, dated July 16, 1982, and C1493, dated July 16, 1982.

(2) Differential horizontal movements: Dial gages and LVDTs on supports between adjacent structures. References: Drawings C1490, dated June 21, 1982; C1491, dated July 16, 1982; and C1493, dated July 16, 1982.

(3) Strains at critical locations: Four extensometers over a 20-ft length, each 5-ft long, on concrete structures and strain gages on steel members. References: Drawings C1495, dated May 21, 1982, and C1493, dated May 21, 1982.

(4) Settlement of some temporary and all permanent underpinning piers relative to superstructure, at top and bottom of piers: Dial gages and telltales. Reference: Figure 2.12 of this SSER.

(5) Concrete stress in selected temporary and all permanent underpinning piers: Carlson stress meters near top and bottom. Reference: Figure 2.12 of this SSER.

(6) Crack mapping: Length and width of cracks. Reference: Applicant's letter of January 25, 1982.

(7) Groundwater levels: Observation wells and piezometers. References: Drawing SK-G-566, Revision 1, dated May 14, 1982, and Bechtel specification 7220-C-198, dated January 18, 1982, as amended at the June 25, 1982 meeting, during a conference call on July 1, 1982, and during the design audit of July 27-30, 1982.

(8) Fines in discharge from dewatering wells: Filter medium with criteria at 0.05 mm and 0.005 mm size. Reference: Applicant's letter of April 22, 1982, p. 19. Although this reference deals with the SWPS, the same monitoring will be performed at the auxiliary building.

Service Water Pump Structure

(1) Total and differential settlement between the north end of the overhang and the portion now founded on till: Dial gages and LVDTs on deep-seated bench marks. References: Applicant's letter of April 19, 1982, p. III-9; meeting of June 24-25, 1982; design audit, July 27-30, 1982; and Figure 2.13 of this SSER, which is to be modified to require permanent bench marks

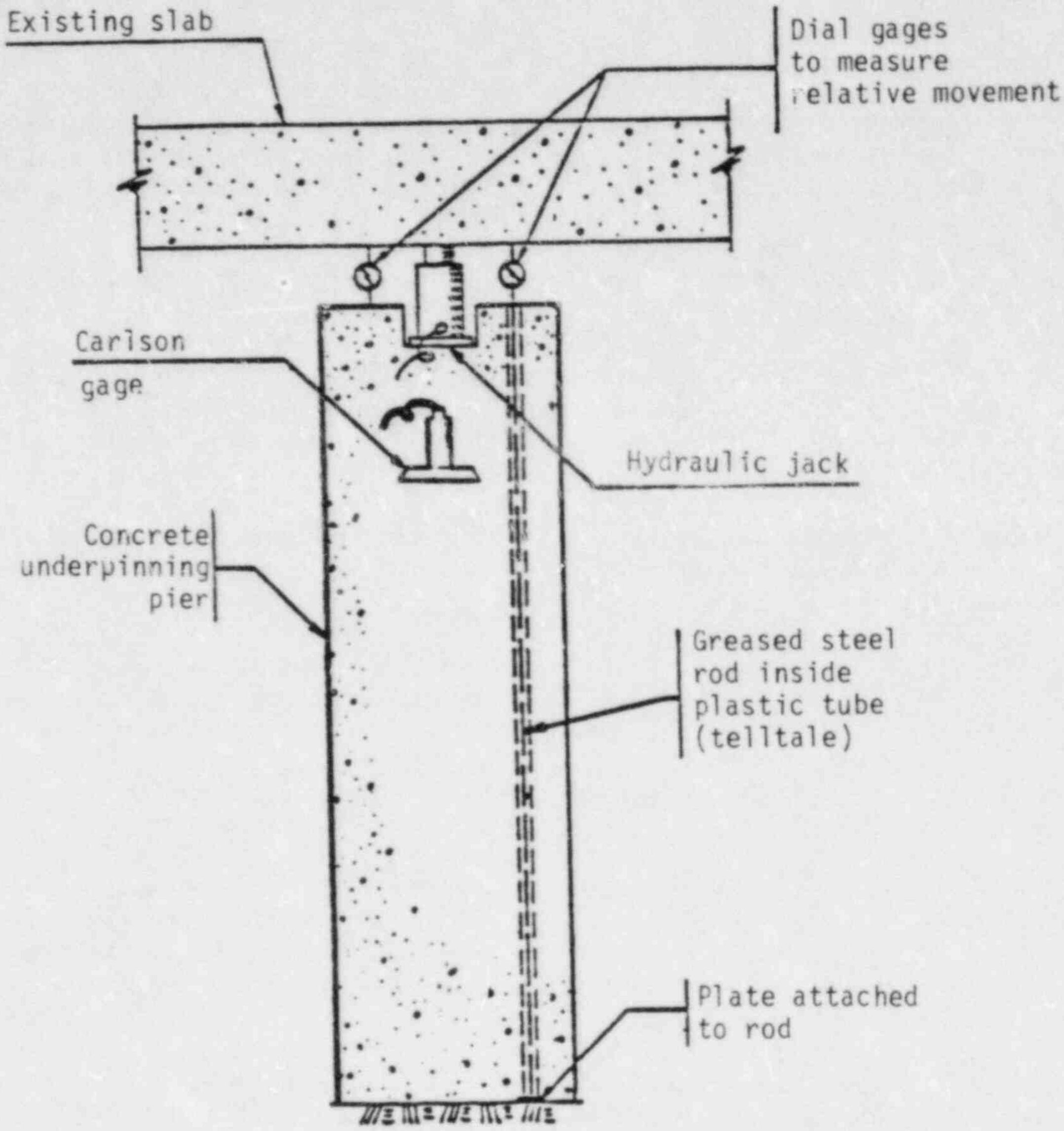


Figure 2.12 Conceptual view of instrumented underpinning pier (Source: Testimony of Edward M. Burke, W. Gene Corley, James P. Gould, Theodore E. Johnson, and Mete Sozen, Vol. 2, p. 5509, December 1, 1981)

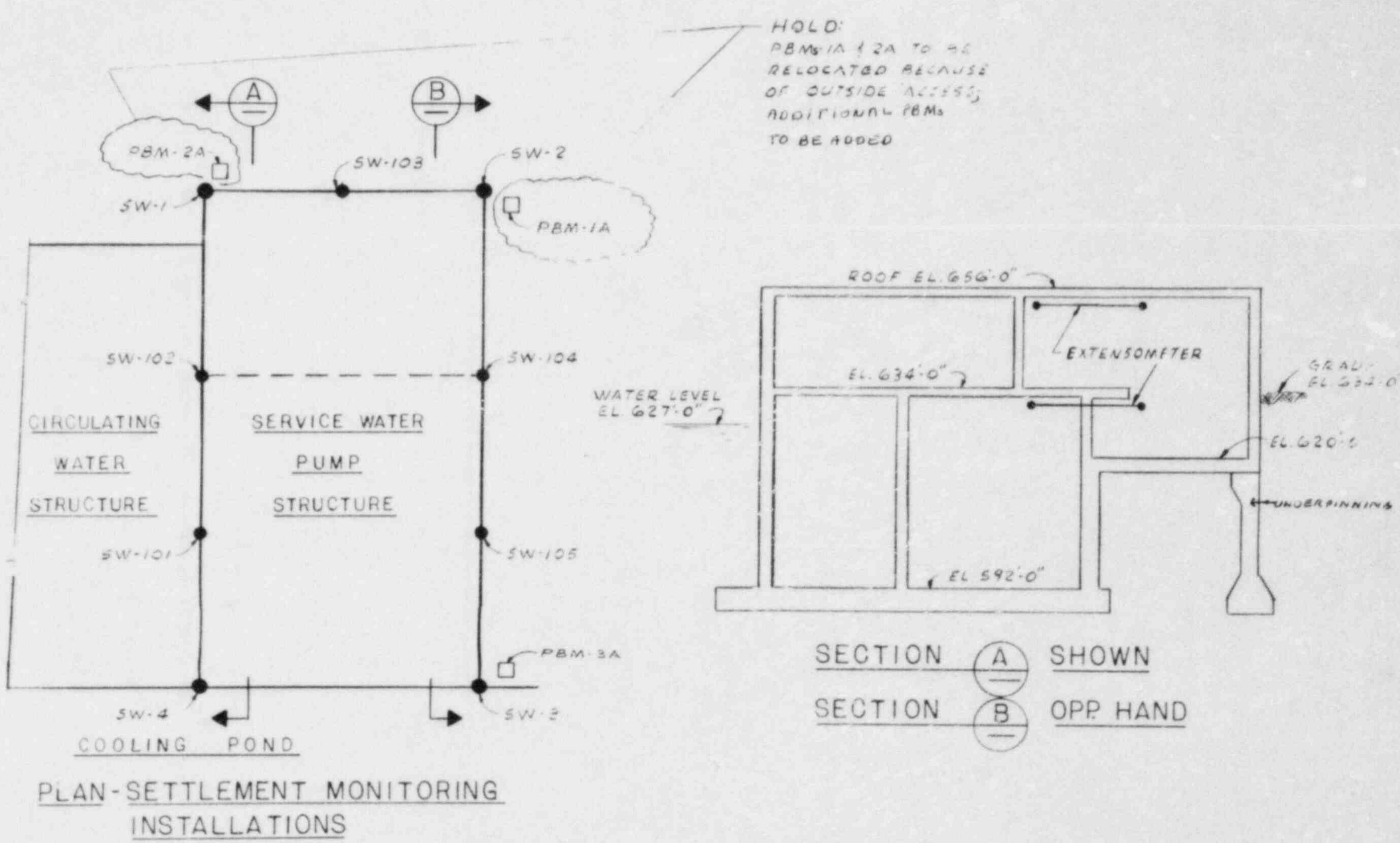


Figure 2.13 Service water pump structure monitoring instrumentation
(Source: Bechtel drawing 7720-SWPS-14)

adjacent to the building settlement marks designated as SW-1, 2, 3, 4, 102, and 104.

(2) Strain of the concrete at critical locations near the intersection between the overhang and the deep portion: Four extensometers over a 20-ft length, each 5-ft long. References: Applicant's letter of April 19, 1982, p. III-9, and Figure 2.13 of this SSER.

(3) Settlement of the underpinning piers relative to the underside of the foundation mat, at both top and bottom of the piers: Dial gages and telltales. Reference: Applicant's letter of April 19, 1982, p. III-10.

(4) Concrete-stress levels within the underpinning piers near the top and bottom for the three piers at each northerly corner: Carlson stress meters. Reference: Applicant's letter of April 19, 1982, p. III-10.

(5) Crack mapping: Length and width of cracks. Reference: Applicant's letter of April 19, 1982, p. III-10.

(6) Groundwater levels: Observation wells and piezometers in the fill and in the sandy clay till. References: Applicant's letter of April 22, 1982; meeting of June 25-26, 1982; and conference calls on July 1-2, 1982.

(7) Fines in the dewatering wells discharge: Filter medium with criteria at 0.05 mm and 0.005 mm sizes. References: Applicant's letter of April 22, 1982, p. 19, and conference calls on July 1-2, 1982.

2.5.4.6.1.2 Criteria

Acceptance Criteria for Auxiliary Building

Differential settlement and strain:

The differential settlements between the southerly ends of the CT and the main auxiliary building and between the southerly ends of the EPAs and the main auxiliary building will be used to control underpinning construction. Alert limits have been set at which the applicant will begin a reevaluation of the behavior of the structure. Also, action limits have been established at which the applicant is required to implement contingency plans (Bechtel specification C-200) to minimize subsequent movements. Limits that were agreed to by both the staff and the applicant at the July 27-30, 1982 design audit are shown in Table 2.7 of this SSER.

If the differential settlements shown in Table 2.7 reach 0.5 in., the applicant will start discussions with NRC for consideration of and concurrence with future actions before implementing those actions.

Settlement of underpinning piers:

After the full jacking load has been applied to the permanent underpinning for the EPAs and the CT, settlement will be monitored until it has been shown that

Table 2.7 Alert and action limits for auxiliary building underpinning

Phase*	Alert limit (in.)	Action limit (in.)
Phase 2 construction	0.10	0.15
Phase 3 construction		
Step 3.1 (through jacking of grillage at E/W8)	0.15	0.25
Subsequent steps (after jacking of grillage at E/W8)	0.15**	0.25**
Phase 4 construction	0.20***	0.40***

*Phases of construction are shown on Bechtel drawings C-1418 (Phase 2) and C-0101 (Phases 3 and 4) (see Appendix I).

**These values may be raised to 0.20 and 0.30 in. gage, respectively, if each extensometer (maximum 5-ft gage length) on the structure shows a strain change less than 0.0010 in./in. (0.1%) during underpinning and the observations of the cracks in the structure all indicate that the long-term behavior of the structure will not be significantly influenced.

***Phase 4 represents the period of load transfer from the jacks to the permanent underpinning. At this stage, movements of the structure are well under control and should be negligible. The previous observations of the cracks and strains in the structure will be used to judge whether these limits are satisfactory.

secondary compression of the bearing stratum is occurring. References: Applicant's hearing testimony at the December 1, 1981 OM+OL hearing, and July 27-30 design audit.

Width of cracks:

Any new cracks exceeding 0.01 in. in width and existing cracks exceeding 0.03 in. in width will be evaluated to determine whether underpinning procedures should be altered or continued.

Groundwater levels:

Test holes (between 1 in. and 4 in. in diameter) will be advanced to a depth of 5 ft beneath the proposed bearing level from a level 5 ft above the bearing level in 11 selected piers to determine whether groundwater under pressure exists and is of sufficient volume to require special pier dewatering. The selected piers are E12, W12, E10, W10, E7, W7, E4, W4, CT-1, CT-6, and CT-12, as shown on Figure I.2 of Appendix I of this SSER.

If water pressures or volumes of seepage are shown to be low in the 11 selected pier locations, and the soils below the pier foundation level are clay as shown in the test holes, then the underpinning excavation to the bearing level is acceptable without additional special dewatering measures. If, however, water pressures or the volume of seepage is shown to be high in the test holes or sand material is shown to be present, special dewatering (e.g., wellpoints or other suitable measures) will be used to lower and maintain the water table at that pier location to a minimum of 2 ft below the bottom of the pier excavation. Test holes beneath the final pier bearing surface are to be grouted.

Fines in well discharge:

If fines larger than 0.05 mm (0.002 in.) in the well discharge exceed 10 ppm, the applicant will determine which well or wells are causing the difficulty and stop pumping from the indicated well(s). If necessary for dewatering, the well(s) would be replaced. Also, the applicant will bring to the attention of the staff any measurement indicating fines more than 10 ppm and coarser than 0.005 mm (0.0002 in.) in the discharge water and will evaluate its significance with respect to the volume of foundation soil that may be eroding.

<u>Acceptance Criteria for SWPS</u>

With respect to underpinning the SWPS, the following movement and strain limits have been established:

Differential settlement:

Alert limit: 0.05 in.
Action limit: 0.07 in.

Strain in concrete as measured by the 5-ft extensometer:

Alert limit: 0.0001
Action limit: 0.0002

Settlement of underpinning piers:

After jacking loads have been applied to final design values, settlement will be monitored until it has been shown that secondary compression of the bearing stratum is occurring. Reference: Applicant's draft hearing testimony, which is the same as Enclosure 13 to the staff's summary of the February 25-26, 1982 meeting, issued March 12, 1982; and July 27-30 design audit.

Width of cracks:

Any new cracks exceeding 0.01 in. in width and existing cracks exceeding 0.030 in. in width will be evaluated to determine whether underpinning procedures should be altered or continued.

Groundwater levels:

Water levels will be monitored to ensure that the groundwater level has been lowered to at least the top of the sandy clay till. An evaluation of potential pervious layers in the bearing stratum below the underpinning piers for the SWPS will be made by continuous sampling in the six borings for the observation wells. At locations where such pervious strata exist within 8 ft below the pier bottom, the groundwater level will be lowered a minimum of 2 ft below the bottom of the pier excavation.

Fines in well discharge:

These criteria for the SWPS are the same as those for the auxiliary building, as discussed previously in this SSER section.

Acceptance Criteria for FIVP

When the differential settlement between the FIVP and any adjacent structure reaches 3/8 in., the FIVP will be lifted back up to its original position.

Pier Foundation Load Tests

Pier W11 will be load tested at the auxiliary building and Pier 1 at the east side of the SWPS. An additional pier will be tested at the SWPS if the bearing level is within the dense sandy alluvium rather than the hard sandy clay till. The piers will be load tested so that a pressure equal to 130% of the maximum predicted bearing pressure throughout the operating life of the plant will be applied to the bearing stratum. The sequence initially requires loading up to 50% of the maximimum load, unloading to 25%, and then raising to 130%. When the load test is complete, the load will be lowered to the design jacking load. The load test procedures have been reviewed by the staff and are acceptable. References: Applicant's letters of April 22, 1982 and June 14, 1982; staff letter of May 25, 1982; meeting of June 25-26, 1982; telephone calls of July 1-2, 1982; and design audit of July 27-30, 1982.

The monitoring programs and the technical and administrative procedures for evaluating and using the data from settlement, strain, crack width, groundwater levels, and fines in well discharge during underpinning for both the auxiliary building and SWPS have been reviewed and are acceptable to the staff.

2.5.4.6.2 Underground Piping

Both settlement and strain monitoring programs are to be carried out during plant operation as a check on the effects of future soil settlement on the safe functioning of seismic Category I underground piping. In this section only the settlement monitoring program is covered. Strain monitoring is discussed in Section 3.9.3.1 of this SSER.

In his March 16, 1982 letter to the NRC (Enclosure 1, "Future Monitoring Program of Buried Service Water Piping for Midland Plant Units 1 and 2"), the applicant provided the criteria used to select settlement marker locations, monitoring frequency, acceptance criteria, and details of typical installation.

The criteria used to select the locations of settlement markers included locating them in areas of loosely compacted fill and where a large potential for differential settlement existed. Using the soil profiles and boring records along the piping alignments, the applicant selected 10 settlement marker locations.

The staff reviewed the proposed locations and, using the same selection criteria as the applicant, concluded that five additional markers were required. At the July 27-30, 1982 design audit, the applicant agreed to install two additional markers on line 26"-OHBC-15, one on 26"-OHBC-20, one on 26"-OHBC-54, and one on 26"-OHBC-55, at stationing recommended by the staff. These markers are in addition to those required for the transition zone, which is discussed in Section 2.5.4.4.5 of this SSER.

The applicant has committed to increase the frequency of his settlement monitoring from the originally proposed rate of once every 90 days (March 16, 1982 letter) to monthly readings for the first 6 months after markers have been installed. This increased frequency is intended to develop background and trends until readings have stabilized (no more than 0.10 in. settlement from previous monthly reading). If after 6 months the settlements have not stabilized, monthly readings are to continue until stabilization has been reached.

On the basis of its review of the information presented in the FSAR and technical reports and on the basis of the above commitments by the applicant, the staff concludes that the settlement monitoring program for underground piping is acceptable and, in conjunction with the strain monitoring program and pipe flow measurements, will provide a suitable verification of the safe functioning of seismic Category I piping.

2.5.4.6.3 Long-Term Settlement Monitoring

The applicant will provide in the Fall of 1982 a Technical Specification covering long-term settlement monitoring during plant operation. The Technical Specification will include identification of total and differential settlement action and alert levels, with remedial measures to be implemented if these levels are reached. The settlement monitoring Technical Specification will address all seismic Category I structures and piping systems.

2.5.4.7 Remaining Review Items

The remaining operating license safety review items are listed in Table 2.8 of this SSER. These items are related to the development of operating Technical Specifications and the future submittal of confirmatory information normally required by FSAR documentation to record as-built construction conditions. Information such as a graphical summary of actual differential settlement records during underpinning and the results of the completed pier load tests are examples of the anticipated as-built records to be provided in future FSAR amendments.

In addition to the items listed in Table 2.8, the staff in SER Section 2.5.6.8 identified the need for the applicant to provide a monitoring plan to visually inspect the dike system during plant operation. The monitoring plan, when approved, will be required to meet the applicable requirements given in RG 1.127,

Table 2.8 Remaining review items

Involved structures	Review items
All seismic Category I structures and piping	Technical Specification covering long-term settlement monitoring
CT, EPAs, SWPS, BWSTs, underground piping	FSAR documentation on as-built condition
Underground piping and conduits	Technical Specification covering restriction on placement of heavy loads over buried piping and conduits
	FSAR documentation on design modification at freezewall crossings

"Inspection of Water-Control Structures Associated With Nuclear Power Plants," for the portions of the cooling pond dike slopes whose failure could adversely impact seismic Category I structures and components.

2.5.4.8 Conclusions

In summary, on the basis of its review of the information provided and identified in the preceding sections, the staff concludes that the site and plant foundations are acceptable and will be adequate to safely support the seismic Category I structures, underground piping, and conduits at the Midland Plant, Units 1 and 2.

This conclusion is subject to the satisfactory evaluation by the staff of the remaining review issues identified in Section 2.5.4.7 and to the results of the structural analysis of the DGB being performed by a staff consultant using a settlement profile that represents the actual measured settlement values.

3 DESIGN OF STRUCTURES, COMPONENTS, EQUIPMENT, AND SYSTEMS

3.7 Seismic Design

3.7.1 Seismic Input

3.7.1.1 Spectra

As discussed in Section 2.5.2 of the SER, the staff determined during the OL review that the original seismic design spectrum used in the design of structures, systems, and components at Midland was not as conservative as current staff practice would require. (This original spectrum was presented in the applicant's FSAR and will be referred to in this SSER as the "FSAR spectrum.") The major structures, systems, and components designed to the FSAR spectrum have been constructed or fabricated and are in place at the plant.

To provide an appropriate representation of maximum seismic motion at the Midland site by current standards, the applicant developed site-specific spectrum. Staff review and approval of this spectrum are also addressed in Section 2.5.2 of the SER. The site-specific spectrum provides a standard of comparison for seismic adequacy of existing structures, systems, and components as well as a basis for evaluating the proposed design of new construction.

By letter dated July 14, 1982, the applicant requested that the staff accept higher seismic damping values for cable trays, conduits, piping, tubing, and their respective supports. This SSER section will address cable trays and conduits. The evaluation of the piping and tubing will be addressed in Section 3.9 of a future SER supplement.

The applicant cited available experimental test data in support of the request for higher seismic damping values. The main report cited to support test results for cable trays and conduits is "Cable Tray and Conduit Raceway Seismic Test Program - Release 4 (Final)," Test Report 1053-21.1-4, December 15, 1978, Anco Engineers, Inc. The applicant requested, for the cable trays and their supports, the use of 4% damping for the operating basis earthquake (OBE) and 7% damping for the SSE in lieu of the FSAR value of 2% damping for both the OBE and SSE. For the conduits and their supports, the applicant requested the use of 2% to 5% damping in lieu of the FSAR value of 2%. Based on the above-stated test results, the staff has approved similar requests (see, for example, SSER 1 of NUREG-0830; SSER 1 of NUREG-0831). Therefore, on the basis of previous similar reviews, the staff finds the proposed higher seismic damping values for cable trays, conduits, and their supports acceptable for the Midland Plant.

3.7.2 Seismic Analysis

3.7.2.1 Remedial Measures

Although many of the foundation remedial measures discussed in SSER Section 2.5.4.4 and below (BWST foundation rings, underpinning for the auxiliary building, and underpinning for the SWPS) represent significant new construction, the new and old structures will be integral at the completion of the program. To avoid speculation on the acceptability of the new construction, in recognition of the ongoing development of the site-specific spectrum and the trend toward more precise design methods, the staff took the position that new construction should be designed explicitly to the site-specific spectrum. The staff also required that a review program be developed for existing structures, systems, and components to ensure that safe shutdown and continued heat removal could be accomplished under the seismic loading of the site-specific spectrum. The applicant has agreed to this approach and, as discussed below, has developed an overall seismic program acceptable to the staff.

At the time design of the remedial measures was initiated, the staff and the applicant had agreed that the development of a site-specific spectrum was an appropriate step and sufficient work had been done to approximate an acceptable spectrum. However, the final spectrum development and staff approval were known to require considerable additional time. In order to proceed with design of the remedial measures while the final site-specific spectrum was developed and approved, the applicant used the FSAR spectrum to calculate structural forces; the applicant then increased those calculated structural forces by a factor of 1.5 to compensate for anticipated increases in seismic input. As noted in SER Section 3.7.2, the applicant committed to demonstrate, before actual construction, that the final design was adequate with regard to the site-specific spectrum. This demonstration was to be made by comparing structural responses calculated by employing the 1.5 factor with structural responses calculated using the final approved site-specific spectrum. The applicant provided for staff inspection during the July 27-30, 1982 design audit comparative displays that indicate that typical floor response spectra for 1.5 times the FSAR spectrum envelop those using the site-specific response spectrum. The applicant plans to provide additional supportive information as part of the seismic safety margins evaluation.

3.7.2.2 Seismic Margin Study

The applicant has initiated a study of the margins above original design requirements that are available to accommodate increased seismic loading from the site-specific spectrum. Similar margin studies have been conducted for several nuclear power plants (e.g., North Anna, Diablo Canyon, Byron, Wolf Creek, and Sequoyah) over the past decade. The experience gained from these studies has shown that nuclear power plants typically have seismic capacities well in excess of the basic seismic design requirement. These margins occur as a result of conservative design requirements (e.g., assumed minimum strength for various classes of materials) and because service requirements, other than seismic loading, usually control final configuration of the various structural elements. The controlling service requirements include pressure and temperature, in the case of containment structures and primary systems; impact loads from tornado missiles and turbine missiles, in the case of auxiliary buildings and diesel generator buildings; and deformation-limited design under thermal and pressure loadings, in the case of valves and pumps.

Although considerable seismic margin has been typically demonstrated throughout the plants, the systems and components necessary for shut down and residual heat removal generally have less stringent normal service requirements and, therefore, are sometimes design-controlled by seismic loading. The staff required, therefore, that the seismic margins in these systems and components be specifically examined.

Initial criteria for the applicant's seismic margin review program were submitted by a letter dated September 25, 1981. Further development of criteria and preliminary results were presented at the ACRS Subcommittee meeting for Midland held on May 20, 1982. Additional discussions concerning analytical methods and implementation of the seismic margins study took place at meetings between the staff and the applicant in Bethesda on June 30, 1981 and during the July 27-30, 1982 structural and mechanical audits. Further discussion of the audit results for soil damping values to be employed in this program appears below under the heading "Static and Dynamic Analysis."

In essence the seismic margins study at Midland consists of reanalyzing structures and samples of systems and components in sufficient detail to determine the response to an earthquake characterized by the site-specific spectrum. The dead load of the structure will be combined with operating live loads and seismic loads. Live loads will include any load occurring as a direct result of earthquake loading. Live loads will also include thermal effects in those cases where thermal effects induce loads that are considered primary (i.e., for vessel nozzles and component supports); pressure loads will be included for containment systems. In those instances where the responses have not increased sufficiently to cause exceedance of allowable criteria (e.g., stress, moment, and shear) no further review is necessary. In those instances where the basic criteria cannot be met, more sophisticated analyses will be performed to determine actual margin to failure.

The seismic margin study proposed by the applicant will include analyses of the reactor buildings, the auxiliary building, the SWPS, DGB, and the BWSTs. These constitute the major seismic Category I structures at the Midland site. The mathematical models employed for the auxiliary building and the SWPS include the final design of the remedial measures developed as a solution to the soil settlement problem. A new model, including staff-approved design changes, has been developed for the BWSTs. Current analytical methods and current criteria (e.g., damping) are also employed.

The applicant has stated that results of the seismic margin study will be submitted to the NRC for the various structures throughout the first quarter of 1983.

The ACRS, in a letter to Chairman Palladino dated June 8, 1982, stated

> The Applicant is currently reevaluating by selective audit the seismic capability of the plant, as originally designed, to withstand the revised SSE. Measures taken to assure safe shutdown in the event of

> an earthquake include the use of dewatering to reduce the potential for soil liquefaction. We recommend that all systems and components important to decay heat removal be carefully evaluated for their ability to accomplish necessary functions in the unlikely event of lower-probability, more severe earthquakes in order to provide the necessary degree of assurance. This matter should be resolved in a manner satisfactory to the NRC Staff. We wish to be kept informed about the resolution of this matter. We believe that any recommendations for changes in the plant resulting from this evaluation should be implemented by the end of the second refueling outage.

Recognizing this recommendation, the staff will require that the seismic margin program as presently constituted be expanded to address all systems and components important to decay heat removal. The schedule for this extended program will allow recommendations for changes in the plant--beyond those, if any, found necessary prior to operation--to be implemented at least by the end of the second refueling.

Based on its review of the program criteria and the structural and mechanical audits cited above, the staff concludes that the applicant's seismic margin review program is acceptable and will provide assurance of the seismic capability necessary for compliance with GDC 2 and Appendix A to 10 CFR 100.

3.7.2.3 Soil-Structure Interaction

By letter of May 7, 1982, the applicant has provided a report, entitled "Relative Soil Impedances Beneath Electrical Penetration Wings of the Midland Auxiliary Building," describing the techniques used to calculate soil-impedance functions (i.e. equivalent soil springs). The staff reviewed this report and found the techniques were equivalent to those identified in Bechtel Topical Report BC-TOP-4/A, Revision 3, which had been approved by the staff. However, the staff asked the applicant to demonstrate that the three peaks in floor response spectra resulting from a variation of ± 30% of the soil stiffness would be enveloped by the design spectra. The applicant's response in Section 3.7.2.9 of the FSAR indicates that the floor response spectra are broadened using the equation recommended in Paragraph C of RG 1.122. The frequency shift as a result of the ± 10% variation in mass, ± 20% variation in structural stiffness, and ± 50% variation in soil modulus of elasticity is considered. Also the three peaks are enveloped. Therefore, this item is closed.

3.7.2.4 Structure-to-Structure Interaction

The applicant has evaluated possible structure-to-structure interaction between the SWPS and the circulating water intake structure (CWIS), and interaction between the turbine building (TB) and the auxiliary building. The applicant's analyses and construction drawings show that no interaction between structural members can be expected between these two sets of buildings for the postulated load conditions. The separation of structural members between the SWPS and the CWIS is about 12 in. for the structural components at the top of the SWPS, el 656 ft, and 1 in. at el 634 ft.; the maximum separation of structural members between the TB and the auxiliary building is 8 in. at the CT and 6 in. at the EPAs. Based on staff inspection of the applicant's dynamic analyses during the

July 27-30, 1982 design audit (see discussion under "Dynamic and Static Analyses" in Sections 3.8.3.1 and 3.8.3.2 of this SSER), the staff concurs with the applicant's analyses and results that show that interaction of the structural members between the SWPS and CWIS and between the TB and the auxiliary building will not occur. The results of the applicant's controlling analyses indicated a 0.518-in. relative movement at el 634 ft for the interaction between the SWPS and the CWIS, and a 3.1-in. movement between the TB and CT at el 704 ft. The calculated relative movement between the TB and the EPAs was 2.6 in. at el 695 ft. These results were developed for SSE loading.

The SWPS is separated from the cooling pond retaining wall by about 1 in. The applicant's dynamic analysis demonstrates that the maximum combined east-west seismic movement of the SWPS and the retaining wall is about 0.25 in., which is less than the 1-in. gap. Hence there is no contact during a seismic event.

3.8 Design of Seismic Category I Structures

3.8.3 Other Seismic Category I Structures

As discussed above, the applicant has proposed remedial actions for the auxiliary building and FIVPs, the SWPS, and the BWSTs, and has completed most of the remedial action for the DGB. For all of these structures, the applicant has performed acceptable analyses considering near-term and long-term conditions, and has proposed acceptable monitoring plans and repair plans to ensure the integrity of the structures during and after the necessary construction. The following discussion provides a description of these analyses and monitoring programs, including staff findings.

3.8.3.1 Auxiliary Building and Feedwater Isolation Valve Pits

The underpinning operation for the auxiliary building is the most complex operation to be executed in the remedial repair work. It requires several stages of construction and, if not properly executed, these could effect the containment buildings, the turbine building, and underground utilities. The applicant did not achieve adequate soil support beneath portions of the auxiliary building. These portions are the CT, both EPAs, and the two adjacent feedwater isolation valve pits (FIVPs). The inadequate soil support under the CT and EPAs is to be corrected by underpinning, and the FIVP is to be corrected by replacing the existing fill with compacted granular fill and concrete. As discussed in Section 2.5.4.4.1 of this SSER, the underpinning scheme proposed by the applicant will result in support of the auxiliary building on competent glacial till. Based on the staff review of soil parameters and dynamic structural calculations, the staff finds that the auxiliary building, through the permanent underpinning walls and the FIVPs with replaced fill, will have a foundation which will provide acceptable structural support.

The applicant is providing both temporary and permanent underpinning systems. The temporary support will be used during the construction of the permanent foundation; it was designed by use of engineering analyses before implementation to avoid damage to existing structures.

Principal features of the temporary support system include

- Steel frames that support the FIVPs from above: These frames are supported by the turbine building on one end and by the containment buttress access shaft on the other end.

- Thirty-six concrete piers at the north end of the turbine building: These piers support the column load of the turbine building along its north side, and they retain soil under the turbine building basemat. These piers are permanently left in place. The piers are braced with struts and tie rods to transmit lateral loads to the containment wall.

- Three frame supports under each EPA: Each frame support consists of a concrete pier, needle beams, and steel columns supported on the reactor building foundation slab or on another concrete pier. These frames also support part of the turbine building load.

- Ten concrete piers under the south side wall of the control tower: These piers are a part of the underpinning wall for the control tower. Struts are provided to transmit lateral loads from the soil under the turbine building to the auxiliary building.

- Additional concrete piers under each of the three existing steel columns inside the control tower: These piers are part of the permanent underpinning.

- Two concrete piers below each buttress access shaft: These support the reaction load from the temporary steel frames that support the FIVPs and retain soil under the buttress access shaft. These piers are permanently left in place.

- Tunnels under the turbine building and access drift tunnels: These tunnels and drifts are constructed by the usual construction methods utilizing lagging and steel frames.

- Temporary post-tensioning: The temporary dewatering system removes the buoyancy force normally provided by groundwater under the EPAs. To compensate for this effect during construction, a temporary system of post-tensioning ties is installed to apply a compressive force to the upper part of the east-west walls of the EPAs. The post-tensioning ties are removed when the temporary supports are installed and jacking loads are applied under the EPAs.

Principal features of the permanent underpinning include

- Underpinning for the Units 1 and 2 EPAs that is a 6-ft thick, reinforced concrete wall 38 ft high, belled out to 10-ft thick at the bottom: The belling limits bearing pressures to the allowable values. The underpinning walls under the control tower are 6-ft thick, 41 to 47 ft high, and are belled out to 14-ft thick. The walls are constructed to act as a continuous member under the perimeter of the structures. Individual piers are provided to underpin interior columns of the building. The entire wall and pier system is founded on undisturbed natural material.

- Design jacking forces that are applied to the existing structure to provide adequate load transfer from the structure to the permanent underpinning: These jacking forces transmit the structural loads through the permanent underpinning wall to the bearing stratum.

- Dowels that 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. These dowels are connected after the permanent load transfer is accomplished.

A permanent wall jacking sequence, a load transfer jacking sequence table, and the other auxiliary building underpinning construction sequences plan and logic are shown in Mergentime/Hanson Engineers drawings S-74 and S-74a and Bechtel drawing SK C-0101, respectively. These drawings were provided for information purposes during the staff design audit of July 27-30, 1982 and portions of them also are shown in Appendix I to this SSER. These preliminary drawings, which had not been approved by the applicant, provide details of the temporary and permanent underpinning walls, construction sequences, and details on values of loads and sequence for the application of the jacking forces.

The staff identified the following concerns as licensing conditions during the review of the auxiliary building and FIVPs: (1) effects on the structure of the post-tensioning system currently in place, (2) the proposed jacking operation, with regard to the forces to be placed on the existing structures, (3) load transfer schemes, (4) analyses of the structures and evaluation to acceptance criteria, and (5) evaluation of cracks.

The staff has since reviewed details of the building post-tensioning system to evaluate the applicant's position that the post-tensioning scheme compensates for the reduction in buoyancy force under the EPAs of the auxiliary building. The staff has also reviewed the proposed jacking operation for implementation of the underpinning scheme on the FIVPs and auxiliary building and connecting details for the CT and EPAs to the main part of the auxiliary building ensure that the underpinning scheme could transfer the loads to the till without overstressing the existing structure. The staff has reviewed the analyses and design calculations, including related drawings, to determine acceptability of the proposed design and construction procedures in relation to current staff acceptance criteria. The staff required an evaluation of the existing cracks in these reinforced concrete structures to ensure that the structure had not been overstressed to a level that would reduce its function under normal operational and accident loadings. Monitoring programs were reviewed to verify that appropriate performance data for structures during and at the conclusion of the underpinning operation are provided for. Further details on the review are given below.

Temporary Post-Tensioning

The temporary dewatering system (Section 2.4.6 of this SSER) removes the buoyancy force normally provided by groundwater under the EPAs. To compensate for the loss of the buoyancy force during construction, a temporary post-tensioning system is installed to apply a compressive force to the upper part of the east-west walls of the EPA. The post-tensioning system consists of two tendons that are located at approximate el 704 ft and run through the CT. They are anchored to the outer walls of the EPA structures both on the inner and outer faces. Each post-tensioning tendon is manufactured using 10 0.5-in.-diameter strands and has an ultimate capacity of 410 kips. These tendons will be removed when the temporary supports are installed and jacking loads are applied under the EPAs.

The applicant's analysis for determining the post-tensioning loads and his evaluation of the supports were based upon the American Institute of Steel Construction (AISC) Code and the American Concrete Institute (ACI) Code 349, which are acceptable to the staff. The staff reviewed the post-tensioning system design and analysis, which showed that the compressive loads imposed on the higher levels of the east and west walls of the EPAs by this system are sufficient to compensate for the loss of buoyancy force and impose no detrimental effect on the structure. Therefore, based on the results of the staff's review and on the applicant's use of codes acceptable to the staff, the staff finds the applicant's post-tensioning scheme to aid construction acceptable.

Jacking Forces

The jacking forces are applied to the existing structure to provide adequate load transfer from the structure to the underpinning. Static analyses for the construction sequence of the underpinning scheme, including jacking loads, are discussed later in this section. The values of the jacking forces are determined from the analyses described below (Dynamic and Static Analyses). These jacking forces are transmitted from the structure through the permanent underpinning wall to the bearing soil stratum.

Installation of permanent load-carrying piers under the CT and details of construction sequence and values of loads and sequence for application of the jacking forces are identified in Mergentime/Hanson Engineers drawings S-74 and S-74a and Bechtel drawing SK-C-0101 (see Appendix I). Specifically, the loads from the EPA, CT, and portions of the TB are considered during the underpinning work by providing an effective transfer mechanism, as follows:

- EPA

  Initial loads will be transferred to a system of grillage beams supported on deep-seated reinforced concrete piers and steel columns that are supported by the containment mat. After fill material is removed, a permanent wall will be constructed on undisturbed natural soil (till). The initial loads will then be transmitted to the underpinning structure.

- CT

  Loads will be transmitted to a series of deep-seated reinforced concrete piers that will subsequently be incorporated into a permanent wall that follows the perimeter of the existing structure.

- TB

  Loads will be directly transferred to the permanent deep-seated reinforced concrete piers.

The transfer of loads described above will in all cases be accomplished by the use of hydraulically actuated steel jacks that are incrementally increased to the specific loads determined by the structural analyses. The analyses consider dead loads, including block walls and equipment loads, and 25% of the estimated live loads to account for construction live loads. When the predetermined loads have been developed by the jacks, the loads will be maintained and locked off provided that the following criteria are met:

(1) The pier will be loaded to 125% of its specified jacking load and continued at that load until the relative movement between the top of the pier and the underpinned structure is less than 0.01 in. in a continuous 1-hour period. When this condition is satisfied,

(2) The pier load will be reduced to 110% of its specified jacking load and continued at that load until the relative movement between the top of the pier and the underpinned structure is less than 0.01 in. for a continuous 24-hour period. When this condition is satisfied, the pier will be locked off.

(3) Jacking loads for the permanent underpinning will be maintained at the specified value for at least 30 days.

(4) A semilogarithmic plot of settlement versus time will show that secondary consolidation has been reached.

(5) The settlement increment in the last 30 days of sustained load will not exceed 0.05 in.

(6) The settlement in the last 10 days of sustained load will not exceed 0.01 in.

(7) Wedges to be used for the permanent wall will be driven tight and permanently welded in place. In case a predicted jacking load is not obtained (when a 0.03-in. upward movement of the existing structure occurs) jacking loads should be reduced to 80% of the load at which the movement occurred and this load will be used in the analyses to determine subsequent jacking loads.

Details of the pier load tests for one representative pier, along with the location and time for performing this test, are discussed in Section 2.5.4.6.1.2 of this SSER.

Based on its review of design drawings and calculations (see discussion under "Dynamic and Static Analyses" below), the staff concludes that the jacking forces have been determined so that the structure will not be stressed above the allowables as specified in American Concrete Institute (ACI) Code 349 under dead load and live load conditions. The staff, therefore, finds the results relevant to jacking forces acceptable.

Dowel and Rock Anchor Load Transfer Scheme

The connection of the underpinning walls and the existing structure under the CT and EPAs is accomplished through the use of embedded dowels. However, rock bolts are used for the vertical connection of the underpinning wall and the existing structure under the CT. The design loads for dowels and rock bolts are in Table 2.1 of the applicant's June 7, 1982 submittal. Both rock bolts and dowels are designed to transfer shear and tension forces between the structure and the underpinning walls. In July 1982 the staff audited the applicant's design calculations yielding the shear and tension forces at the junction between the underpinning wall and the existing building. The staff found that the rock bolts and dowels used will have sufficient strength and are, therefore, acceptable because the results meet design criteria acceptable to the staff (ACI 318/349).

Dynamic and Static Analyses

The seismic model for the auxiliary building is represented by a three-dimensional lumped-mass stick model. The main part of the auxiliary building and the CT, excluding the electrical penetration wing areas, were lumped into two stick models. The EPA are represented by six sticks connected to a series of plate elements. The connection between the main auxiliary building and the CT sticks is represented by a series of beam elements at floor elevations. The wing stick nearest the CT is connected to the control stick by rigid beam elements. In all cases, the individual sticks have been located at the calculated center of shear resistance. The masses associated with the main auxiliary building and CT have been lumped at major floor elevations. For the wing areas the masses associated with plate elements have been lumped considering the effect of each plate thickness; the remaining masses are lumped at the floor elevations along the six sticks. The proposed underpinning design has been accounted for in the section properties below el 614 ft. Torsional effects were considered in the dynamic analysis. Soil-structure interaction was represented by equivalent soil spring constants and damping coefficients based on the elastic half-space theory. Embedment effects also were considered.

The model described above was used to evaluate overall building response to seismic loading as well as to generate instructure response spectra. The response developed from this model provided input to other static analyses to develop forces in the individual structural elements.

Four different finite element (static) models are being used to analyze the auxiliary building: (1) the construction sequence for the proposed underpinning, (2) long-term loading without connection between the underpinning and the building, (3) long-term loading with full connection between the underpinning and building, and (4) short-term loading with full connection between the underpinning and building.

These models consist primarily of plate elements. Beam elements are used, however, to represent columns, minor concrete elements, and major steel components of the structure. The unique characteristics of each model are briefly described below.

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 and the spring constants for the boundary springs reflect the soil properties before underpinning. The load cases applied to the model are: dead load, live load, jacking loads, external hydropressures, soil pressures, and wind loads.

The model using long-term loading without connection between underpinning and building investigates 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 this model are: dead load, live load, external hydropressures, soil pressures, settlement, jacking loads, and wind loads.

The model using long-term loading with full connection between the underpinning and building investigates 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 include differential settlement loads.

The model using short-term loading with full connection between the underpinning and building investigates 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 this model are: east-west earthquake, north-south earthquake, vertical earthquake, tornado, and pipe rupture loads.

The results of the analyses based on the above models and loads were combined using the load combinations specified in the FSAR. The combined forces or stresses were then used to evaluate the structural adequacy of the existing structure and the underpinning structure as a whole. The information on these analyses was provided in part by Enclosure 2 of the applicant's letter of September 30, 1981, and Enclosure 2 of the applicant's June 7, 1982 letter. Details of the above-referenced results were audited in February and July 1982. The staff reviewed the results of the dynamic analyses and ascertained that proper outputs from these analyses were used as inputs to the static analyses. In addition, the results of the static analyses were compared to code allowables (ACI-349) and the staff determined that adequate safety factors have been provided. The results audited by the staff will be summarized and presented by the applicant in a future revision to the FSAR.

The staff has reviewed and audited the applicant's static and dynamic analyses and finds that the assumptions used to define the models, loads, and load combinations are adequate. The models adequately represent the structures, and the results of the analyses are acceptable because they meet the requirements identified in SRP Sections 3.7.2, 3.8.3, and 3.8.5.

Monitoring Program

The monitoring program considers differential settlement and strain for the structure as well as a crack monitoring and repair program. Critical locations for monitoring on the structures were identified from the results of the analyses. Three locations of high stresses were identified as critical structural locations: a slab at el 659 ft, the structural area adjacent to the post-tensioning cable anchors at el 704 ft, and a shear wall at approximate el 600 ft. The final details of the monitoring plan are identified in Bechtel drawings C-1490 and C-1491 (see Section 2.5.4.6.1.1 of this SSER). Acceptance criteria are identified in Section 2.5.4.6.1.2 of this SSER.

Crack Evaluation and Repair

The crack evaluation and repair program is discussed as a general item for all structures in Section 3.8.3.5 of this SSER.

3.8.3.2 Service Water Pump Structure

As discussed in Section 2.5.4.4.1 of this SSER, the soil support intended for the north portion of the SWPS, which is located on fill material, was not achieved. The applicant has proposed underpinning similar to that under the auxiliary building, and the approaches presented by the applicant are generally equivalent to those identified in Section 3.8.3.1. The underpinning operation for the SWPS, however, is much smaller in scale and complexity than that for the auxiliary building and does not require a temporary underpinning scheme. Also, because of the limited area of concrete, embedded dowel anchors rather than rock anchors will be used at the vertical interface between the existing structure and the underpinning wall.

The proposed SWPS remedial underpinning is approximately a 4-ft-thick reinforced concrete wall that is approximately 30 ft high with a flared base. The underpinning wall is constructed to act as a continuous member under the perimeter of that portion of the structure originally founded on fill (i.e., the "overhang" portion located to the north). To preserve the structural integrity of the building, the individual piers are constructed in a predetermined sequence. The piers are tied together with threaded reinforcing bar couplers provided by Fox-Howlett Industries, Inc. (see Section 3.8.3.6 of this SSER) and shear keys to form a continuous underpinning wall. The final underpinning scheme and construction sequence for the SWPS are identified in Figure 2.9 of this SSER and Bechtel specification 7220-C-194.

The areas of concern identified during staff review of the structural engineering aspects of the SWPS addressed in this SSER included the following: (1) the temporary post-tensioning system and effects on the structure, (2) jacking loads, (3) load transfer schemes, (4) analyses of the structures and evaluation to acceptable criteria, and (5) evaluation of existing cracks.

Temporary Post-Tensioning

A temporary post-tensioning system is designed to apply a compressive force to the upper part of the building along the east and west exterior walls. This post-tensioning system is provided to compensate for the loss of bouyancy, which results in additional forces on the overhang, when the construction site dewatering is implemented. The post-tensioning system consists of two tendons on each side of the building in the north-south direction, and contributes a compressive force of 100 kips. These tendons are located on the outside of the building and are anchored to the exterior walls with heavy steel brackets.

The staff reviewed the post-tensioning system and concluded that the compressive loads imposed at the higher levels of the structure by this system are sufficient to compensate for the loss of bouyancy force and impose no detrimental effect on the structure. Use of the post-tensioning scheme to aid in construction is, therefore, acceptable to the staff.

Jacking Forces

Predetermined jacking forces will be applied between the underpinning wall and to the full perimeter of the SWPS overhang foundation during the construction of the underpinning wall to provide adequate load transfer from the structure to the underpinning wall.

The staff reviewed the methodology for transferring loads from the jacks to permanent wall and the locking-off procedure to be used for the underpinning operation. Details of the construction sequence and the values of the loads and sequence for application of jacking were identified in documents submitted to the staff (specifically Bechtel drawing C-2035-Q, Rev. 2 and Bechtel specification 7220-C-194(Q)). These values were determined from the static analyses. Load transfer to the piers will be completed and the jacks will be locked off when the following criteria are met:

(1) The pier will be loaded to 125% of the specified jacking load and continued at that load until the relative movement between the top of the pier and the underpinned structure is less than 0.01 in. for a continuous 1-hour period. When this condition is satisfied,

(2) The pier load will be reduced to 110% of the specified jacking load and continued at that load until the relative movement between the top of the pier and the underpinned structure is less than 0.01 in. in a continuous 24-hour period. When this condition is satisfied, the pier will be locked off.

(3) Final load transfer to the completed underpinning system will be controlled by the following acceptance criteria:

   (a) The jacking load for the permanent underpinning will be maintained at the specified value for at least 30 days.

   (b) A semilogarithmic plot of settlement versus time must approach a straight line.

   (c) The settlement increment in the last 30 days of sustained load will not exceed 0.01 in.

   (d) Wedges to be used for the permanent wall will be driven tight and permanently welded in place. In case a jacking load, as predicted by the analyses, is not obtained (when a 0.03-in. upward movement of the existing structure occurs) jacking loads should be reduced to 80% of the load at which the movement occurred and this load will be used in the analyses to determine subsequent jacking loads.

Details of the pier load test for one representative pier, along with location and time for performing this test, are discussed in Section 2.5.4.6.1.2 of this SSER.

Static analyses for the construction sequence of the underpinning scheme, including jacking loads, are discussed later in this section. These jacking forces are transmitted from the structure through the permanent underpinning wall to the bearing soil stratum. Based on its review, the staff concludes that the jacking forces are limited so that the structure will not be unduly stressed under dead load and live load conditions. Therefore, the staff finds the results relevant to jacking forces acceptable.

Dowels and Anchor Bolts Load Transfer Scheme

The permanent connection between the underpinning wall and the existing structure is made by dowels consisting of no. 9 steel reinforcing bars (rebars) at the vertical interfaces and 2-3/4-in.-diameter anchor bolt assemblies at the horizontal interfaces. The connectors are designed to transfer shear and tension forces to the underpinned wall. The connectors are not subjected to stresses during the jacking procedures because the connectors will not have been installed and the anchor bolts will not have been tightened. After the underpinning wall is connected to the existing structure, the connectors begin to resist the applied structural loads.

The connectors for the vertical interface between the underpinning and the side of the SWPS are designed to resist the combined effects of a shear force of 1668 kips, an axial force of 1144 kips, and a moment of 736 ft/kips, with a safety factor of 1.23. For the horizontal interface, anchor bolts of varying lengths are provided for the connections with the base slab and the north wall. For the base slab connection, the anchor bolts are designed to resist the combined effects of a shear force of 1646 kips, an axial force of 885 kips, and a moment of 261 ft/kips, with a safety factor of approximately 2.4. For the north-wall interface, the anchor bolts are designed for the combined effects of a shear force of 2198 kips, an axial load of 3515 kips, and a moment of 1834 ft/kips, with a factor of safety of approximately 4.5.

These values were obtained during the audit conducted in July 1982. The staff finds the above-stated values acceptable because the stresses resulting from these loads meet the allowable limits specified in ACI-349 with conservative safety factors.

Dynamic and Static Analyses

The SWPS is represented by a three-dimensional lumped-mass stick model with individual sticks located at the center of shear resistance. The mass of the structure is lumped at the major floor elevations. The center of mass is established for each floor level and the eccentricity between the center of mass and the center of rigidity is included in the model. Rigid beam elements are used to connect the center of stiffness and the center of mass. The proposed underpinning of the building has been accounted for in the section properties below el 620 ft. Soil/structure interaction is represented by equivalent spring constants and damping coefficients based on elastic half-space theory. Torsion effects from structural eccentricities are considered. Embedment considerations also have been included.

The static structural analysis uses a finite-element analytical model to represent structure behavior. The interface between the existing structure and the underpinning wall is modeled to transfer loads. The soil media are represented by springs of appropriate stiffness at the base of the structure.

The analysis uses two different analytical models and appropriate springs. The two analytical models that the applicant has developed include a "disconnected model" and a "connected model." The disconnected model, in which the underpinning wall is not connected to the structure, is used to investigate various construction stages. This model also is utilized in combination with the connected model to determine preload effects on the existing structure that result from jacking. The same level of analysis described in Section 3.8.3.1 of this SSER for the auxiliary building is provided for the SWPS.

The connected model, in which the underpinning wall is connected to the structure, is used to investigate the effects of long-term loading such as differential settlement and short-term loading such as seismic forces. The differential settlement is considered in the model by calculating appropriate spring constants based on settlements of the underpinning and the existing structure.

A summary of the results of structural analyses for the major construction stages was provided in the applicant's April 22, 1982 submittal. The specific calculations were audited in July 1982. The staff ascertained that the controlling stresses due to the jacking loads were combined with other applicable loads identified in the design load combinations. This validation assured the staff that the applicant has considered all relevant loads in the design of the underpinning structure. The results for these analyses, at the critical section, are identified in Table 3.8-22 of Revision 44 to the FSAR. The staff finds these results acceptable because they meet the allowable limits specified in ACI-349.

The staff also reviewed the sliding and lateral soil pressure calculations under dynamic loading during the audit. Seismic loads equal to 1.5 times SSE loads were used. These exceeded the SSRS seismic loads. Factors of safety against sliding were determined to be 1.45 in the north-south direction and 1.50 in the east-west direction. These values exceed the staff requirements of 1.1. Therefore, this issue is considered closed.

The staff has reviewed and audited the applicant's static and dynamic analyses and finds that the assumptions used to define the models, loads, and load combinations are adequate. The models adequately represent the structures, and the results of the analyses are acceptable because they meet the requirements identified in SRP Sections 3.7.2, 3.8.3, and 3.8.5.

Monitoring Program

The monitoring program includes differential settlement, strain, and crack monitoring. The monitoring measurements and criteria are discussed in Section 2.5.4.6 of this SSER.

Crack Evaluation and Repair

The crack evaluation and repair program is discussed in Section 3.8.3.5 of this SSER.

Conclusions

The general underpinning scheme proposed by the applicant will effectively transfer support of the SWPS to the competent undisturbed natural material, as described in Section 2.5.4 of this SSER. The staff concludes that successful implementation of this program will result in an acceptable foundation for the SWPS. The structural aspects proposed for the underpinning scheme and design (use of a temporary post-tensioning system, jacking loads, load transfer schemes, dynamic and static analyses, and monitoring program) are acceptable because they meet acceptable code requirements as specified in Sections 3.7.2, 3.8.4, and 3.8.5 of the Standard Review Plan (NUREG-0800).

3.8.3.3 Borated Water Storage Tanks and Foundations

Each unit of the Midland Plant has a 500,000-gal, stainless steel, borated water storage tank (BWST). These tanks are 32 ft high and 52 ft in diameter.

Each BWST has a flexible, flat bottom. The tank shell, roof, and part of the water in the tank are supported by a reinforced concrete ring wall. Compacted granular fill lies inside the ring wall, and a 6-in. layer of oiled sand is between the tank bottom and the granular fill. Approximately 25 ft of compacted fill lies under the foundation structure. The tank bottom is flexible and most of the vertical pressure from the weight of the water and the tank bottom is transferred directly to the soil inside the ring wall. This vertical pressure also causes a lateral pressure, which is resisted by the ring wall. Forty 1-1/2-in.-diameter anchor bolts, which are evenly spaced and embedded around the periphery of the foundation ring wall, provide anchorage to the tank for resisting overturning loads caused by externally applied lateral forces.

The existing ring wall measures 4 ft 6 in. high and 1 ft 6 in. wide with footings that are 4 ft wide and 1 ft 6 in. thick. Each valve pit is 10 ft 8 in. high and extends 4 ft 8 in. below the bottom elevation of the ring wall foundation. The valve pit walls are 1 ft 6 in. thick, and the top and bottom slabs are 1 ft 6 in. and 2 ft thick, respectively. A depth transition is located at each ring wall and valve pit intersection to provide a continuous ring wall through each valve pit. The valve pit provides access to the piping connection to the tank and houses the valves for the fill and drain lines.

The original design of the reinforced concrete BWST foundations included the load of two other small tanks located on the top slab of each valve pit. The applicant relocated these tanks but did not reevaluate the foundation design for this change, nor potential settlement because of the site soil condition. Differential soil settlement developed as a result of a water load test; the valve pit rotated relative to the ring walls and induced bending moments. These additional bending moments were not considered in the original design.

The BWST foundation design deficiency was confirmed by a structural analysis that indicated that the allowable moment capacity for the dead load and the differential settlement condition was exceeded in several locations in the foundation structure. A visual inspection at the predicted overstressed locations during the water load test showed cracks in the Units 1 and 2 foundations, which indicated large strains and potentially overstressed reinforcing steel.

The applicant adopted a three-phase corrective action consisting of

(1) a surcharge program to reduce or eliminate the effects of differential settlement. Staff evaluation of the surcharge program is discussed in Section 2.5.4.4.3 of this SSER.

(2) construction of a new ring wall around the periphery of the existing ring and designed to carry all of the postulated loads. Shear connectors would transfer the shear forces from the existing ring wall to the new ring beam.

(3) establishment of a releveling program for the Unit 1 tank.

A monitoring program will also be conducted during and after construction of the new ring.

New Ring Beam Design

The new reinforced concrete ring beam to be added to each tank foundation as a foundation design correction is shown in Figure 3.1 of this SSER. The ring beam will be 2 ft wide and 4 ft 6 in. deep and will be located on top of the exterior footing projection. A depth and width transition will be located adjacent to the valve pit that is similar to the transition zone of the ring wall. A portion of the ring beam 1 ft 6 in. wide and 3 ft 6 in. deep will extend through the valve pit. A portion of the reinforcing bars will extend through the valve pit embedded in a 5-in.-thick slab added on top of the original valve pit floor slab. The 5-in.-thick slab will connect to the floor slab by 1-in.-diameter anchor bolts grouted into holes drilled in the floor slab. The portion of the ring beam reinforcing bars that does not extend through the valve pit will be grouted into holes drilled in the valve pit slabs. Steel bolts with nuts will be used to connect the ring beam to the ring wall and the valve pit.

Dynamic and Static Analyses

The staff has examined the applicant's dynamic and static analyses of the BWST design proposal and finds that the applicant has used a satisfactory method of evaluation. In these analyses, the existing ring wall was modeled with curved shell elements; the ring wall footing and valve pit were modeled with plate elements; the new ring beam was represented by thickening the curved shell elements representing the ring wall and affected plate elements; and the soil subgrade was modeled by brick elements. Two sets of elastic moduli were used in the analysis, one set for long-term loads and the other set for short-term loads. Long-term loads include differential settlement loads as well as dead and live loads.

The applicant has modeled the tanks by considering the horizontal impulsive mode, the sloshing mode, and the vertical mode of the fluid-structure interaction. Each of these modes is modeled with its own individual model, and the seismic forces imposed upon the tank shell ring foundation from each of these models was added by the square-root-of-the-sum-of-the-squares (SRSS) method.

In the impulsive mode the tank shell stiffness is modeled by vertical beam elements between mass points distributed up the tank shell. The beam elements represent the shear and flexural stiffness of the tank. The roof weight is lumped at the roof level, and the shell wall weights are lumped at discrete points of the tank shell. The effective weights of impulsive fluids are added to the tank shell weights at each of these node points at and below the top of the fluid. This horizontal impulsive tank model was attached to the ground at its base by soil-structure interaction impedance functions (defined in terms of translation and rocking stiffnesses) and dashpots. The resulting overturning moment and base shear at the base of this model represent the forces imposed on the ring foundation by the horizontal impulsive mode.

For the sloshing and vertical modes, empirical formulas were used to evaluate the natural frequency, the fluid effective sloshing weight, height of application, base shear, and overturning moment on the ring foundation. Use of these empirical formulas is acceptable to the staff. The model used in the design of the foundation rings was a simpler model that developed higher loads. The foundation rings have been designed to these higher loads, resulting in a conservative design.

The applicant evaluated the current condition of the BWST. Survey measurements of the ring walls indicate that the tops of the foundation rings have been distorted from their original positions, resulting in a nonuniform support condition for the steel tanks. Based on the staff's review of the results of the finite element analyses, this condition did not impose any unacceptable stresses to the tank components.

A three-dimensional finite element model was constructed to represent the BWST cylindrical wall. The wall is constructed of flat plate elements possessing both bending and membrane stiffness. Vertical boundary elements are utilized at the bottom surface to represent the nonlinear behavior of the asphalt-impregnated fiberboard between the tank and ring wall. Additional horizontal boundary elements at the tank wall lower edge represent the stiffness and restraint offered by the flat tank bottom. The upper edge of the tank is stiffened by a ring girder with properties chosen to represent the stiffness of the tank roof and the restraint that it offers to the tank at the roof/cylinder intersection.

Loading conditions include dead weight of the tank roof, weight of the cylindrical shell, and weight of effective annulus of water plus measured anchor bolt loads. Hydrostatic radial pressure loads acting on the tank wall also were included.

The finite element model provided by the applicant for the evaluation of the old foundation ring wall consisted of a symmetrical T-beam cross-section loaded through the center of mass. The addition of the new ring beam changed the original T-beam configuration into a composite beam formed by the old and new rings. The resulting composite beam does not have a symmetrical cross-section and is not loaded through the center of mass. Therefore, it could not be adequately analyzed using the old model without additional considerations. The applicant has reevaluated the foundation ring wall using the composite configuration of the old and new ring and has determined that the new ring wall is able to resist the resulting stresses without exceeding the allowable stress values identified in the acceptance criteria (ACI-349). Also, the applicant has provided justification for the amount of water column assumed to act on the ring beam, based on the stiffness of the plate wall junction on an elastic foundation. The staff finds that the proposed models and analytical techniques are acceptable because they follow approaches previously approved by the staff.

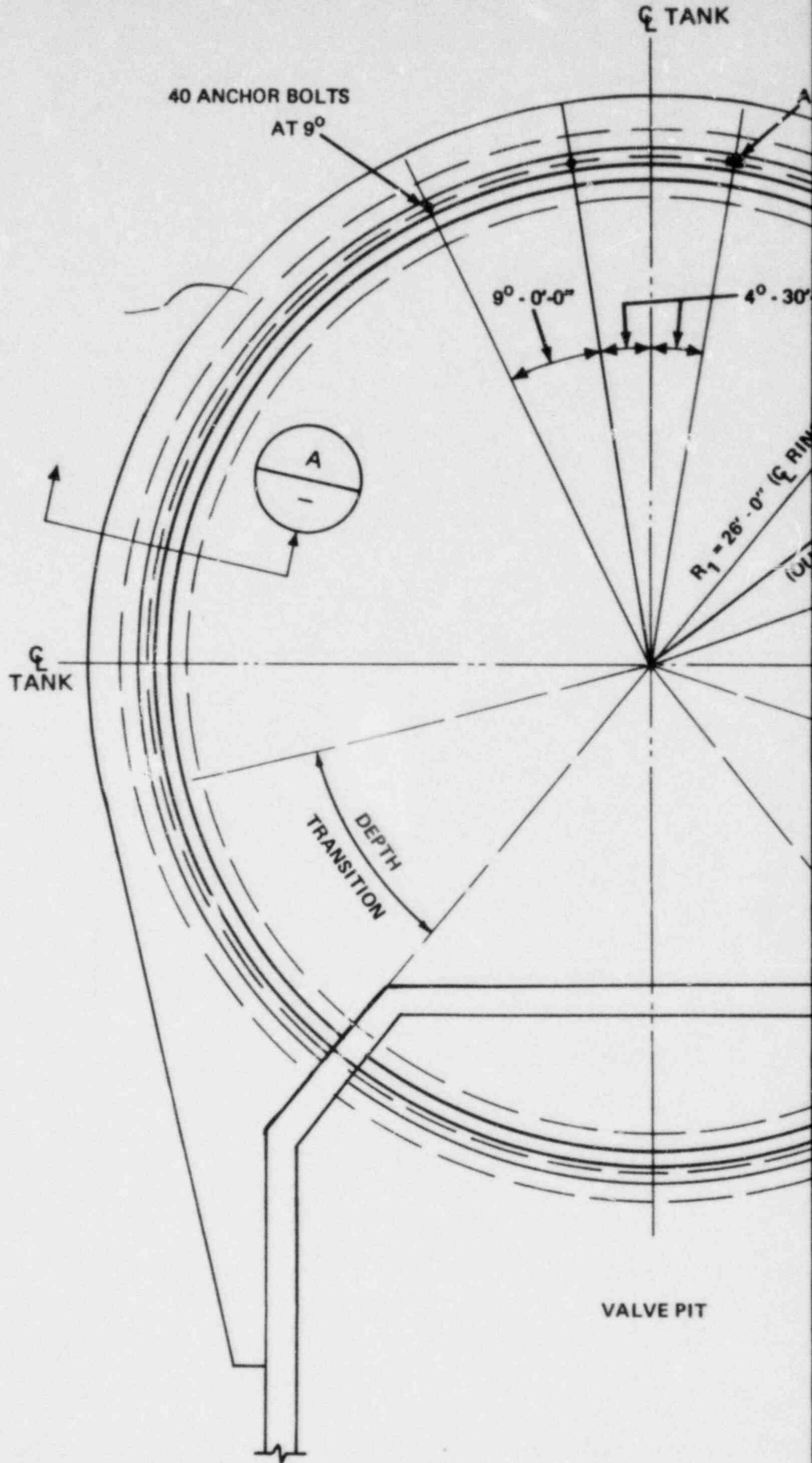
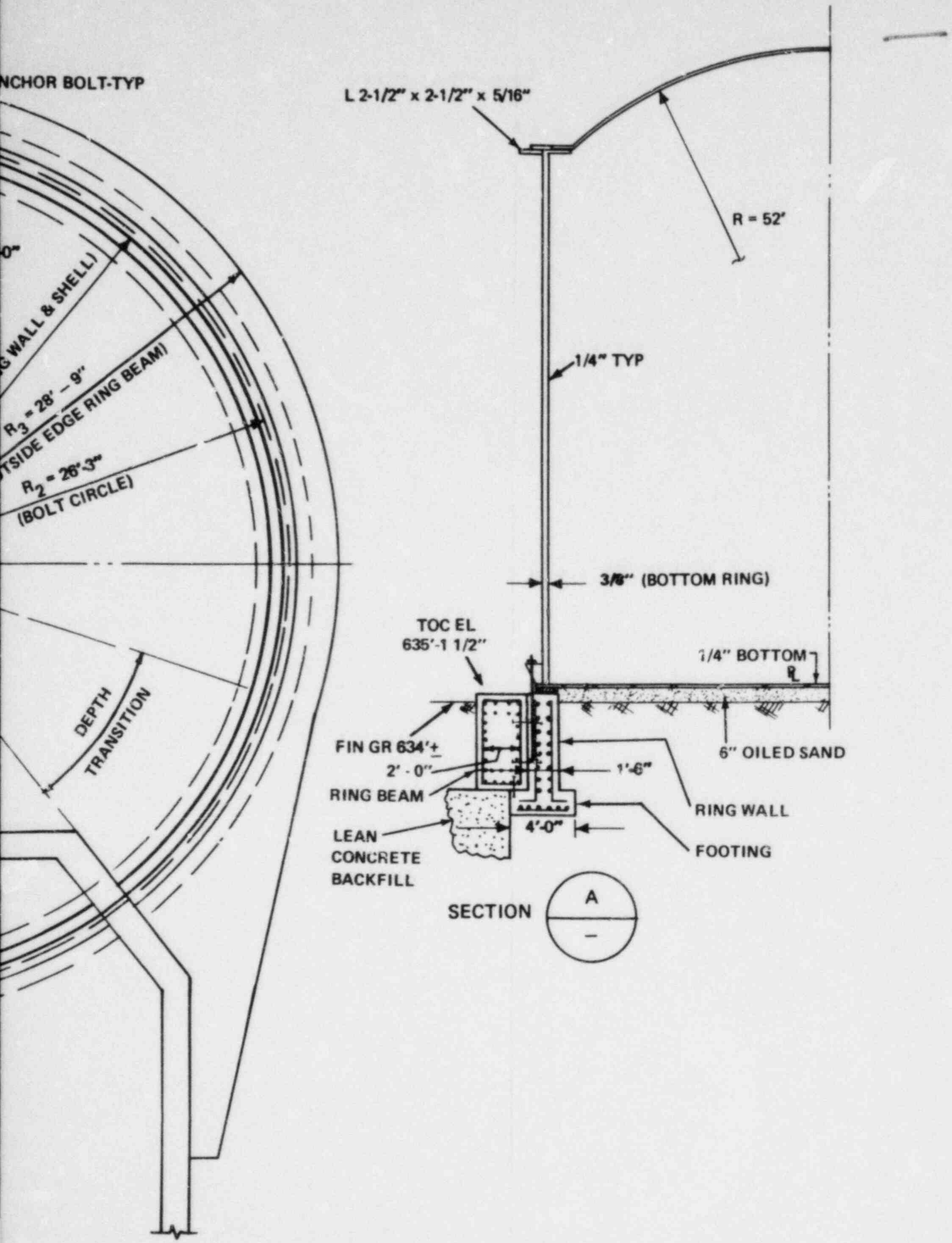


Figure 3.1 Reinforced corcrete ring beam for



BWST foundations (Source: FSAR Fig. 3.6-60, Rev. 44)

Figure 3.1

Load Combinations and Design Criteria

Before the SER was issued, the applicant had not clearly identified the load combinations and respective evaluation of the foundation rings. The applicant has since stated that the new ring beam interface shear connectors and the new ring foundation are designed to the requirements of ACI 349-76, as supplemented by RG 1.142, "Safety-Related Concrete Structures of Nuclear Power Plants Other Than Reactor Vessels and Containments." These criteria agree with the staff requirements identified in SRP Section 3.8.4 and are acceptable.

Releveling

The staff agrees that releveling of the Unit 1 tank should be performed as identified in the applicant's letter of April 22, 1982. The proposed technique is acceptable to the staff. Releveling of the empty tank will be accomplished by jacking the tank after the anchor bolts are disconnected, leveling the existing ring wall by grout, releveling the oil-sand layer below the tank bottom plate, and reattaching the tank to the foundation by anchor bolts. Strain gages will be installed on the tank walls and bottom. The releveling process will be stopped and the condition evaluated if measured stress (from strain data) exceeds 1.25 times anticipated stress level as follows:

Location	Anticipated stress, psi
Bottom plate	4328
Tank wall, first stage lift (about 18 in.)	7000
Tank wall, second stage lift (about 3 ft)	6200

The applicant has not proposed, and the staff does not require, that the Unit 2 tank be releveled. The applicant has not elected to relevel the Unit 2 BWST based on the results of the measurements taken on the ring wall and on the measured loads in the bolt supports following the water load test. With the submittal of January 11, 1982, the applicant enclosed a report prepared by his consultant, Structural Mechanics Associates (SMA) entitled "Evaluation of Midland Nuclear Power Plant Borated Water Storage Tanks for Non-Uniform Support Loading Resulting from Ring Settlement." This report provided information justifying the decision to not relevel the Unit 2 BWST.

Although the two tanks were loaded with the same loads during the water test, the Unit 2 BWST was subjected to much lower displacements than Unit 1 BWST. Also, the relative displacements between various points along the circumference of the ring wall were much less for the Unit 2 BWST than those for the Unit 1 BWST. The values of relative displacements for the Unit 2 BWST met the American Petroleum Institute (API) Standard 659, "Welded Steel Tanks for Oil Storage," which provides leveling criteria for ring foundations supporting storage tanks.

Finally, measurements of loads impacted on the tank through the anchoring bolts indicated that, for the worst location, the measured loads for Unit 2 BWST were lower than those for the Unit 1 BWST by approximately a factor of 10. Based on the above justifications, the staff agrees with the applicant that no significant benefit would result from releveling the Unit 2 BWST.

Crack Repair Program

Based on prior staff approval in a letter dated March 26, 1982, the applicant has applied epoxy under pressure to cracks larger than 0.01 in. in the old foundation ring in preparation for construction of the new ring beam.

Crack Monitoring Program

The new ring beams will be monitored for cracks in critical areas (i.e., near the depth transition zones of the ring beam) for at least 6 months after the tanks are filled initially with water. If any crack in these areas reaches 0.03 in., the tank will be emptied and the condition evaluated. At the end of the 6-month period, an evaluation of the effect of any existing cracks will be made and submitted to the NRC.

Conclusion

Adding a new ring to the periphery of each existing ring and releveling of one tank, as proposed by the applicant, will result in an acceptable foundation for the BWSTs.

3.8.3.4 Diesel Generator Building

The DGB is a reinforced concrete structure with three crosswalls that divide the structure into four cells. Each cell contains a 6 ft-6 in.-thick concrete pedestal to support a diesel generator unit. The building is supported on continuous footings that are founded at el 628 ft and rest on backfill that extends down to approximately el 603 ft. This rectangular boxlike structure covers an area of approximately 70 ft by 155 ft. The exterior walls are 30 in. thick, and the interior walls are 18 in. thick. The foundations of the exterior and interior walls of the DGB consist of continuous reinforced concrete footings, 10 ft wide and 2 ft 6 in. thick, with their base at 628 ft. The walls rise from an elevation of 628 ft (bottom of footing) to 680 ft (top of roof slab).

Soil-related structural problems were first identified during early construction stages in July 1978 when excessive settlement of the DGB was detected. After a review of settlement observations and the results of an exploration program, surcharging and a subsequent permanent dewatering program were implemented as the remedial action plan, as discussed in Sections 2.5.4.4.2 and 2.5.4.6.2 of this SSER.

An early investigation also showed that the four electrical duct banks that were supported on the deeper, more competent, natural clay were providing resistance to DGB settlement in localized areas, thus resulting in formation of cracks. To eliminate this problem, a positive clearance between the building foundation and the duct bank was provided before placement of the surcharge.

The staff review and evaluation of the remedial action proposed and completed for the DGB focused on the cause and elimination of the excessive differential settlement condition, the applicant's structural acceptance criteria, the determination of proper soil and structural models to be used for additional analyses and evaluation of present and future conditions of the structure, the evaluation of the cracks developed during differential settlement and duct impingement loading, and the establishment of an adequate differential settlement and crack monitoring and repair program. The surcharge of the DGB accelerated settlement and produced soils with improved engineering properties. These properties have been used in both the static and seismic reanalyses of the DGB. Differential settlements, both measured and 40-year predictions, have been included in the applicable Midland load combinations.

The applicant has performed both static and seismic reanalyses using soil properties acceptable to the staff (Section 2.5 4.2 of this SSER). For the seismic analysis, the structural stiffness of the existing lumped mass model was not revised; only the treatment of the soil structure interface was changed. Because no modifications have been identified for this structure, the staff finds this approach acceptable. In addition to horizontal and torsional acceleration, the weight of soil and concrete base slabs, together with the diesel generator pedestals within the building, was included in the reanalysis. The responses developed from this analysis were used as input to the static reruns as required. For the static reanalysis, a new set of springs with varying properties (one vertical and one horizontal at each foundation boundary node point) representing the nonhomogenous nature of the existing soil conditions was developed and used in the finite element model. Sets of springs were developed for the long-term settlement (40 years) and short-term (tornadoes, earthquake) loadings. Once the finite element model was completed, the Midland load combinations (ACI 318) for the DGB were applied to the structure and the static analyses rerun. Stresses and displacements at each of the nodal points for each of the load combinations were obtained. The applicant also reanalyzed the DGB in accordance with ACI 349 as supplemented by RG 1.142. The staff finds these methods acceptable.

During a July 27-30, 1982 design audit, the staff and its consultant reviewed four analyses of the DGB, two for each of the configurations and loadings existing before surcharge, one during surcharge, and one after surcharge. The staff and its consultant also reviewed the applicant's analysis of a surcharge, an extreme hypothetical case in which part of the foundation support has zero spring stiffness and the remaining part supports an equivalent spring stiffness. This latter condition is considered an upper bound on the differential settlement calculations for the foundation structure. The staff review of the settlements used for this analysis is discussed in Section 2.5.4.4.2 of this SSER. In addition, the applicant has committed to crack monitoring and mapping programs, a displacement measurement program, and a crack evaluation program (SER Section 3.8.3.5).

The staff has reviewed and evaluated the analyses and design calculations, including related drawings, to determine their acceptability in relation to the current staff acceptance criteria. This review has been accomplished through various submittals and during structural design audits at Bechtel offices.

The analytical models used for the original seismic analysis and for the seismic reanalyses described in this report are one-dimensional, stick-type, lumped-mass models using beam elements to represent the structural stiffness and impedence functions of the foundation medium.

The effect of soil-structural interaction is accounted for by coupling the structural model with the foundation media. The foundation media are represented by impedence functions that represent the equivalent spring stiffness and radiation damping coefficients as specified in Bechtel Topical Report BC-TOP-4/A, "Seismic Analysis of Structures and Equipment for Nuclear Power Plants."

The structural stiffness of the lumped-mass model was not revised in the new dynamic analysis. The difference in the new model was confined to the treatment of the soil-structure interface. The revised analysis developed the impedance functions based on the building's foundation dimensions and modification in the soil properties. In addition, for the horizontal accelerations, the weight of the soil and the concrete pedestals and diesel generator pedestals within the building were included in the revised model.

The original (presettlement) DGB seismic analysis was based on the underlying till material, which has a shear wave velocity value of 1359 fps. The first seismic reanalysis accounted for the soil properties of the fill by averaging the measured shear wave velocity of the fill and underlying till over a depth of 75 ft, which is the smallest dimension of the building. This resulted in the value of 796 fps, which was used in the seismic reanalysis. However, the effect of decreasing shear wave velocity to a lower bound estimate of 500 fps was also analyzed. Both the measured shear wave velocity value of 796 fps and the lower bound shear wave velocity value of 500 fps are acceptable to the staff.

The floor spectra at all elevations of the diesel generator building were generated using a shear wave velocity value of 796 fps. The resulting floor response spectra were combined in an enveloping fashion with the spectra developed in the original analysis, which used a shear wave velocity value of 1359 fps. The floor response spectra were further broadened to account for a lower bound shear wave velocity of 500 fps. The staff concludes that use of this broad range of input ensures that conservative floor response spectra were generated. The results of the seismic reanalysis indicated that the seismic forces at all elevations of the DGB were somewhat higher than the forces determined in the original analysis. The highest seismic acceleration was derived from an analysis using a shear wave velocity value of 796 fps. This increased seismic load was conservatively simulated by applying the maximum acceleration occurring in the dynamic model to the finite-element model in north-south, east-west, and vertical directions. The combined effect of the three directional responses was assessed using the SRSS method recommended in RG 1.92, "Combining Model Response and Spatial Components in Seismic Response Analysis."

The DGB structural (static) reanalyses use a finite-element model consisting of an assemblage of plates, beams, and boundary elements. The structure is defined by a set of 853 nodal points and 1294 elements. Of these elements, 901 are plate elements, and 252 are boundary elements (translational springs, in both the vertical and horizontal directions) representing varying soil conditions. Vertical springs were used for dead load, live load, and settlement load in the analysis. To represent the soil-structure interaction, various horizontal and vertical springs representing the nonhomogeneous nature of the existing soil conditions beneath the DGB were used. Loads were then applied to the model as either surface loads on the elements or nodal loads at specific nodal points. The loads applied are consistent with FSAR Section 3.8.6.3.

The settlement effects were modeled into the structure with vertical springs as boundary elements representing varying soil conditions. At 84 locations along the building footing, springs with varying properties were applied to represent the nonhomogeneous nature of soil conditions existing beneath the DGB.

Values for vertical springs were developed for two general cases: those springs calculated for long-term loading (dead load, live load, surcharge load, and differential settlements) and those springs calculated for short-term loading (wind, tornado, and seismic).

For long-term loading, the settlement analysis addresses four cases, each covering a distinct time period. A unique set of measured or estimated settlement values then corresponds to each of the following:

(1) March 28, 1978 to August 15, 1978: Although construction of the DGB began in Spring 1978, survey data on the DGB were available only as of March 28, 1978. August 15, 1978 represents the closest survey date prior to the halt of DGB construction.

(2) August 15, 1978 to January 5, 1979: DGB construction resumed and the duct banks were separated from the structure during this period. January 5, 1979 is the last survey date before the start of surcharge activities.

(3) January 5, 1979 to August 3, 1979: Surcharge activities occurred within the structure during this period. August 3, 1979 is the last survey date available before the start of surcharge removal from the DGB.

(4) Forty-year settlement estimates: This estimate is comprised of the following:

    (a) Actual measured settlements from September 1979 to December 1981. These settlements are small when compared with the predicted settlements and occurred mainly because of dewatering.

    (b) Predicted secondary consolidation from December 1981 to December 2025. These values are based on the conservative assumption that the surcharge remains in place over the 40-year life of the plant, thus exceeding the settlement which will actually occur.

To determine forces resulting from settlement, the applicant performed an analysis separately for each of the above four cases. Analysis results were first combined with each other to form one settlement term, then combined with other load cases (e.g., tornado, seismic, etc.) to form the required load combinations of a Midland position, given in the applicant's letter of June 1, 1982, and of ACI 349 as supplemented by RG 1.142.

For settlement Case 1, a longhand analysis was performed to account for stresses in the partially completed structure. With the actual settlement values from survey data imposed on the partially completed structure (represented as a grade beam up to el 656.5 ft), this calculation indicated that the measured displacements would result in a maximum rebar stress of 11 ksi. For the other three settlement cases, individual finite-element models were used. For settlement Case 2, the finite-element model represents the structure as built to el 662 ft 0 in. For settlement Cases 3 and 4, the finite-element model represents a fully constructed structure. For Cases 2, 3, and 4, springs were typically calculated at each nodal point along the foundation by dividing the structural load represented at the selected point by the measured or predicted settlement at that point. The finite-element analysis of each case then involved several iterations in which the soil springs were varied until the deflected shape of the DGB, as calculated by the model, approximated the "best fit" settlements, as discussed in Section 2.5.4.4.2 of this SSER. As far as that one aspect of the analyses is concerned, the staff believes that a better characterization for analytical purposes would have been to consider that the deflected shape of the DGB was consistent with the measured settlements. However, this difference has not had a significant impact on the staff's conclusion as discussed below.

The calculated moments and forces resulting from settlement as characterized by the applicant have been combined with the required load combinations and the results presented to the staff. A review of these results showed that the allowable stress limits were not exceeded, and, in fact, the great majority of structural elements were under very low stress. A more conservative analysis in which deflection to the measured settlements were assumed would result in some changes in calculated reinforcing bar stress with some local regions very possibly exceeding original design values. However, the staff's review of the extensive analyses performed to assess this matter indicates that such local regions, were they to exist, would have no significant impact on the functional quality of the DGB under all operating and environmental conditions. In addition, staff consultants are performing an independent analysis of the DGB using measured settlements as a confirmatory measure.

The staff also requested that a finite-element analysis of the DGB be performed for the 40-year dead load case, modified with zero and near-zero soil spring constants (i.e., little or no support from soil) in areas to represent potential bridging. The primary purpose of this analysis would be to investigate the structure's ability to span any soft soil condition. Specifically, the value of zero was used at the junction of the south wall and east center wall, while soil spring values would then be linearly varied so that springs returned to their original 40-year values within a distance of approximately 15 ft from the zero spring. The results from this analysis (maximum rebar stresses) were compared with the previous analysis. This comparison showed that, except for some increases in the south wall, the footings, the box missile shield, and the south shield wall, the maximum rebar stress values remained essentially unchanged. Typically, stress level increases were limited to approximately 5 ksi except in the south shield wall, where the modeling technique caused the rebar stress value to increase approximately 18 ksi, and in the footings where the nature of the analysis causes the rebar stress value to increase approximately 20 ksi.

As a result of the analysis performed, the staff concludes that the DGB can successfully span an assumed soft soil spot introduced into the analysis without significantly increasing rebar stress levels.

3.8.3.5 Cracks

The staff, during the course of its review, identified a concern about the cracks mapped for the siesmic Category I structures fully or partially founded on fill material. The applicant has shown, by example where necessary, that existing cracks do not significantly affect the strength in tension, compression, and shear of properly reinforced concrete elements. Evidence from the field and from the laboratory has been presented to indicate that reinforced concrete structures will develop their design strength even if they have "precracks," provided the structure has been proportioned and detailed to resist design load combinations. In addition, the applicant proposed a monitoring plan to detect differential settlement of the structure and the propagation and enlargement of new and existing cracks, along with an independent evaluation of conditions which exceed predetermined limits acceptable to the staff and a crack repair program acceptable to the staff.

Evaluation of Cracks

Originally, the applicant identified all cracks in seismic Category I structures as shrinkage cracks. Accordingly, the applicant planned to identify only these cracks, without performing any evaluation of the effects of the cracks on the structure. The staff agreed that some of the cracks might be shrinkage-type cracks, but, based on the differential settlement records, on impingement loads, on recorded sizes and patterns of cracks, and on predicted values of settlement for the life of the plant (40 years), the staff required an evaluation of the effect of cracks on the respective structures.

The applicant has evaluated the existing mapped cracks and plans to evaluate future sizes and patterns of cracks that may develop during the underpinning operation and during the life of the plant. Initially, the applicant attempted to resolve this staff concern with an evaluation of a single through-crack within the DGB. A crack was analyzed using the Automatic Dynamic Incremental Non-linear Analysis (ADINA) computer program. A subsection of the east wall of the building was isolated and modeled using truss and three-dimensional solid elements. The prescribed boundary displacements were taken from an analysis using the Bechtel Structural Analysis Program (BSAP) computer code described in FSAR Section 3.8.4.4. The most critical design load combinations (tornado/seismic) were applied to the model. The model was evaluated for the following conditions: (1) a model without precrack, (2) a model precracked without initial stress, and (3) a model precracked with initial stress in the reinforcement.

Although the staff concluded that the ADINA program is capable of evaluating residual strength of a precracked reinforced concrete element, the results could not be extrapolated for multiple through-crack regions. The modeling of the element around the crack tips was not fine enough to show possible stress concentration. Therefore, the analysis cannot predict possible crack propagation under unfavorable loading conditions. Subsequently, the applicant submitted the following reports as an evaluation of cracks in the seismic Category I structures affected by the soil fill material at the Midland site:

(1) "Evaluation of Auxiliary Building Control Tower and Electrical Penetration Areas at Midland Plant," W. G. Corley and A. E. Fiorato, January 1982.

(2) "Evaluation of Feedwater Isolation Valve Pits at Midland Plant," W. G. Corley and A. E. Fiorato, January 1982.

(3) "Evaluation of Cracking in Service Water Pump Structure at Midland Plant," W. G. Corley and A. E. Fiorato, February 1982.

(4) "Evaluation of the Effect on Structural Strength of Cracks in the Walls of the Diesel Generator Building, Midland Plant Units 1 and 2, Midland, Michigan," M. A. Sozen, February 1982.

These reports discuss the significance of cracks in general and as applicable to the specific structure addressed in the report. The evaluation stresses critical location and location with largest through-cracks. The general criteria used in these evaluations for the determination of the significance of developed cracks include the following: (1) geometry of member, (2) amount and distribution of reinforcement in the member, (3) material properties of the member, (4) function of the member, (5) magnitude and distribution of loads on the member, (6) construction technique, (7) sequence of construction, (8) crack location and distribution, (9) crack size, and (10) interaction of multiple cracks. As a measure of significance of observed cracks relative to future integrity of the structure, the tensile stress that uncracked concrete is assumed to carry was compared with available tensile capacity provided by structural reinforcement crossing the cracks. This calculation was made for sections in the vicinity of cracks that had a measured width of 0.10 in. or greater. Available structural reinforcement was determined from Bechtel drawings. In the calculation, concrete is assumed to carry a principal tensile stress of $4\sqrt{f'_c}$ where f'_c is spe ified concrete compressive strength. This assumption is consistent with Section 11.4.2.2 of the ACI Building Code. Resistance of reinforcement was calculated as $A_s f_y$ where A_s equals area of reinforcement and f_y equals specified yield stress of reinforcement. If calculated resistance provided by reinforcement crossing the crack exceeds $4\sqrt{f'_c}$, there is sufficient reinforcement to carry the stress attributed to the concrete. Failure to meet this criterion would require additional detailed analyses in the form of a limit analysis. The results presented in the above references indicate compliance with the combination of all general and specific criteria as stated above, except for four locations within the SWPS where limit analyses were used to resolve failure to meet the $4\sqrt{f'_c}$, criterion.

The basic approach used in the limit analysis of the SWPS was to determine if forces that can be induced in the structure are sufficient to exceed capacity of walls, assuming the existence of cracks. Inplane shear capacity of cracked walls was of primary concern in these analyses. Results indicate that these four walls have sufficient inplane shear capacity to resist the hypothesized limiting forces.

Based on the review of the results of the above evaluations, the staff finds the applicant's evaluation of cracks in the applicable Category I structures acceptable.

Crack Monitoring and Repair Program

The applicant has identified a crack monitoring and evaluation program to be used during the underpinning operation and the life of the plant, as applicable. Also, details of the crack repair program have been established by mutual agreement between the applicant and the staff after a series of discussions. The program that applies to the DGB, SWPS, CT, and EPAs of the auxiliary building and FIVPs will be completed before the first refueling of the plant. It consists of the following:

(1) Repair by epoxy injection any cracks in the structures which are below the permanent groundwater table and which exhibit weeping characteristics. This repair will be performed from the inside of the structures.

(2) Coat the splash zone of the exterior surface of the south wall of the SWPS that is in contact with cooling pond water with waterproofing compounds.

(3) Repair by epoxy injection existing cracks that are 20 mils and larger and apply a sealant to the surfaces of the concrete walls in the accessible areas discussed below (i.e., areas where removal of soil or installed equipment or installed components is not necessary to perform the repair). The extent (length) of the crack that will be injected with epoxy will be limited to a crack width of 10 mils or larger. Before the initiation of repairs, all cracks 20 mils and larger and weeping cracks in the applicable areas will be identified. A verification of this identification to a tolerance of + 5 mils will be performed. This verification and subsequent repair will be in accordance with the applicant's quality assurance program. The material for structural epoxy adhesive will be "Concresive-1380," manufactured by Adhesive Engineering Company, or equivalent.

The areas to be repaired for each applicable building are as follows:

- DGB

  all accessible interior reinforced concrete walls

  all accessible exterior concrete walls

- CT and EPAs

  all accessible exterior concrete walls

- SWPS

  all accessible exterior walls

The staff finds the applicant's commitments on crack monitoring acceptable and the repairs for the applicable Category I structures acceptable.

3.8.3.6 Fox-Howlett Mechanical Tapered Threaded Splices

Steel for tension elements in reinforced concrete requires direct, positive coupling of adjoining elements in conformance with standards defined by the

American Concrete Institute (ACI), the American Society of Mechanical Engineering (ASME), and staff positions. Currently, the splice system used in such applications is the cadweld system.

The applicant has proposed to use mechanical tapered threaded splices manufactured by Fox-Howlett Industries, Inc. to connect the reinforcing bars in the construction of the underpinning walls for the auxiliary building and SWPS. These splices have not previously been used in a seismic Category I structure for a nuclear power plant.

The Fox-Howlett splice system is used to positively connect steel reinforcing bars to develop the minimum ultimate tensile strength of the concrete reinforcing steel on which they are used. The coupler design concept consists of taper threading the ends of a standard rebar and connecting the rebar ends with a threaded coupler. The Fox-Howlett coupler is furnished in three types (the standard coupler, the position coupler, and the structural connector) and comes in several sizes to accommodate nos. 9, 10, 11, 14, and 18 rebar.

The staff, with the assistance of a consultant from Science Applications, Inc., (SAI) has evaluated the data obtained from Fox-Howlett and a number of users of the couplers, as well as applicable codes and regulations. The SAI report includes a description of coupler design features, the process used to manufacture the couplers, installation procedures, quality assurance/inspection procedures, engineering analyses, and performance tests conducted by Fox-Howlett to substantiate the coupler design.

The staff is conducting a generic review of all splice systems. A report discussing the details of the staff's findings will be issued at the conclusion of this review.

Based on the review and evaluation of all available data prepared by SAI for the Fox-Howlett coupler, the staff accepts the use of only the standard coupler for nos. 9, 10, 11, and 14 rebar. This acceptance criterion meets the needs identified by the applicant for the construction of the underpinning structures for the auxiliary building and the SWPS.

The staff has not approved use of the Fox-Howlett position coupler because sufficient test data have not been provided. The applicant will therefore use cadweld splices in place of Fox-Howlett position couplers.

3.8.3.7 Conclusion

The criteria used in the analysis, design, and construction of the auxiliary building, SWPS, BWSTs, and DGB to account for anticipated loadings--including past and future settlement and postulated conditions that may be imposed upon each structure during its service lifetime--are in conformance with criteria, codes, standards, and specifications acceptable to the staff.

The use of these criteria as defined by applicable codes, standards, and specifications; the loads and loading combinations; design and analysis procedures; the structural acceptance criteria; the material, quality control, and special construction techniques; and the testing and inservice surveillance requirements provide reasonable assurance that, in the event of winds, tornadoes, earthquakes, and various postulated accidents occurring within the structures, the structures will withstand the specified design conditions without impairment of structural integrity or the performance of required safety functions. Conformance with these criteria, codes, specifications, and standards, pending resolution of one confirmatory item, will constitute an acceptable basis for satisfying, in part, the requirements of GDC 2 and 4.

Long-term settlement monitoring plans as applicable to the structures are to be provided in the Technical Specifications. As-built conditions, as well as results from short-term monitoring requirements during the underpinning operation, will be incorporated in the FSAR at the appropriate time.

3.8.4 Foundations

The discussion of foundations is in Section 3.8.3 of this SSER.

3.8.5 Masonry Walls

Section 3.8.2 of the SER noted, as a confirmatory issue, that the applicant had been asked to comply with staff criteria on masonry walls in seismic Category I structures. The issue also was identified as item 3 in SER Section 1.8.

The materials, testing, analysis, design, construction, and inspection related to the design and construction of seismic Category I masonry walls must conform to the requirements of Appendix A to the Standard Review Plan (NUREG-0800), Section 3.8.4, NRC Criteria for Safety-Related Masonry Walls.

The loads and load combinations used by the applicant in the analysis and design of seismic Category I masonry walls for the Midland Plant are in conformance with staff criteria given in Appendix A of SRP 3.8.4 and are, therefore, acceptable.

The applicant's specifications for the installation of concrete expansion anchors rely upon installation torque to determine proper installation. The required load capacity of the installed anchors was established by test data supplied by the applicant on July 23, 1982 to qualify the use of expansion anchors in masonry walls. The information transmitted by the applicant's letter of July 23, 1982 included a Bechtel report, 7220-CTR-1-1, "Report on the Testing of Concrete Expansion Anchors and Grouted Anchors Installed in Concrete Block Walls." The test procedure and results used to establish ultimate load capabilities of expansion anchors in seismic Category I masonry walls are acceptable to the staff. The applicant used a factor of safety of 5 to establish allowable load capacities of expansion anchors used in masonry walls. This factor of safety is compatible with the applicant's test data and is consistent with the value in IE Bulletin 79-02. Use of this factor of safety is, therefore, acceptable to the staff.

The criteria used by the applicant in the design analysis of the seismic Category I masonry walls to account for anticipated loadings that may be imposed upon the structures during their service lifetime are in conformance with the staff's criteria for masonry walls, and with codes, standards, and specifications acceptable to the staff. The staff concludes that in the event of earthquakes and various postulated accidents, the seismic Category I masonry walls will withstand the specified design conditions without impairment of structural integrity.

Conformance with these criteria constitutes an acceptable basis for satisfying, in applicable part, the requirements of GDC 2 and 4. Accordingly, confirmatory item 3 in SER Section 1.8 is closed.

3.9 Mechanical Systems and Components

3.9.3 ASME Code Class 1, 2, and 3 Components Component Supports, and Core-Support Structures

3.9.3.1 Loading Combination, Design Transients, and Stress Limits

The staff review of underground seismic Category I piping has been performed with the assistance of a consultant, Energy Technology Engineering Center (ETEC), Canoga Park, California.

Sections 1.12.10 and 3.9.3.1 of the SER identified the underground seismic Category I piping at the Midland Plant and discussed the status of staff concerns pertaining to the effects of past, current, and future differential soils settlement on the structural integrity and functional adequacy of the buried piping throughout the life of the plant. These SER sections noted that proposed remedial actions were under staff review. The staff review is now complete. Section 3.9.3.1.1 of this SSER describes the proposed remedial actions that address these staff concerns. The status of the underground seismic Category I piping, acceptance criteria, and the resolution of staff concerns are also discussed.

The staff finds that the remedial actions for underground piping proposed by the applicant, as described in various submittals and meetings including a design audit on July 27-30, 1982, are acceptable. However, as a confirmatory item, the documentation provided in these submittals and meetings should be combined and submitted as a single document.

3.9.3.1.1 Status of Seismic Category I Underground Piping

The seismic Category I piping in the Midland Plant fill is identified in Figure 2.11 and Table 3.1 of this SSER.

The applicant's letter of March 16, 1982 identified a 48-in.-diameter line (48"-OHBC-2/48"-OJYY-1) between the SWPS and cooling tower which, at that time, was indicated by the applicant to be seismic Category I for approximately 10 ft from the SWPS. SER Section 1.12.10 noted that staff review of this 10-ft length of 48-in.-diameter piping from the SWPS was continuing.

Inside the SWPS there are two seismic Category I butterfly valves isolating the line (48"-OHBC-2/48"-OYJJ-1 on FSAR Figure 9.2-2) from the normal routing to the cooling pond. Functionally, a class change occurs at the butterfly valves; however, the applicant's design practice, as noted by his letter of April 15, 1982, was to extend the pipe class to the first anchor point beyond the safety/nonsafety isolation valves. In this case, the anchor point was the soil to about 10 ft outside the SWPS. At the class change, the line also changed from

Table 3.1 Seismic Category I underground piping

Service water lines		Diesel fuel oil lines	
8"-1HBC-310	26"-0HBC-54	1-1/2"-1HBC-3	2"-1HBC-497
8"-2HBC-81	26"-0HBC-55	1-1/2"-1HBC-4	2"-1HBC-498
8"-1HBC-81	26"-0HBC-56	1-1/2"-2HBC-3	2"-2HBC-497
8"-2HBC-310	26"-0HBC-15	1-1/2"-2HBC-4	2"-2HBC-498
8"-1HBC-311	26"-0HBC-16		
8"-2HBC-82	26"-0HBC-19	Borated water lines	
8"-1HBC-82	26"-0HBC-20		
8"-2HBC-311	36"-0HBC-15	18"-1HCB-1	
10"-0HBC-27	36"-0HBC-16	18"-1HCB-2	
10"-0HBC-28	36"-0HBC-19	18"-2HCB-1	
26"-0HBC-53	36"-0HBC-20	18"-2HCB-2	
Penetration pressurization lines		Control room pressurization lines	
1"-1CCB-45		4"-0DBC-1	
1"-2CCB-46		1"-0CCC-1	

carbon steel to concrete. The applicant has now reclassified the 48-in.-diameter line downstream of the valves to a nonseismic category. Section 9.2.1 of this SSER presents the staff finding that failure of the 48-in.-diameter line would not cause loss of the essential SWS cooling. The staff, therefore, concurs in this reclassification of the 48-in.-diameter line downstream of the butterfly valves.

The seismic Category I SWS piping consists of Code* Class 3 SA-106 and SA-155 carbon steel piping in various diameters (8, 10, 26, and 36 in.).

All the 36- and 26-in.-diameter SWS piping was subjected to extensive profile and pipe ovalization measurement programs in November 1981. Profile data were obtained at 5-ft intervals along the pipe length and at welds and are accurate to 1/16 in. These 1981 data supersede the 1979 data, which were accurate only to 1/4 in. The 1982 data show that the piping is, on the average, approximately 5 in. below its design elevation with deviations of up to 8 to 12 in. These deviations are outside the scope of the 2-in. deviation permitted by the applicant's construction specifications. The 1982 data also show that, in general, pipe ovalizations were between 1 and 1.5%, with some locations of 2% and greater, with a maximum of 3%. Both the initial profile and ovality are unknown. Measurements of rattlespace annuli were also made at all building penetrations.

All the 8- and 10-in. SWS piping is located in the vicinity of the DGB. All these lines were installed before the soils settlement problem was recognized

*ASME Boiler and Pressure Vessel Code, Section III, 1980 Edition, including the Winter 1981 addenda.

and they were in place during the DGB surcharge loading program. These lines were profiled in 1979, and the data indicated that they were, on the average, 6 to 8 in. below their design elevation with a maximum of up to 21 in. The two longest lines that exhibited the greatest deviations are located north of the DGB and were rebedded after removal of the DGB surcharge. Additionally, pipe-diameter verification has been conducted on four 30-ft lines. Verification indicated that these four lines were acceptable in accordance with American Waterworks Association (AWWA) requirements (i.e., less than 5% ovality). These rebedded and diameter-verified lines are presently disconnected at the bolted connections at or near their DGB penetrations and will be recentered in their rattlespace annuli.

The borated water lines are 18 in. in diameter and are comprised of ASME SA-358, Grade 304 stainless steel. The piping was installed and fabricated in accordance with the requirements of the Code, Class 2. Profile data obtained in 1979 and 1981 show that these lines are below their design elevation by up to 2 in., the construction installation limit. The differential settlement effects for these lines have been evaluated, and the staff finds them to be acceptable.

The diesel fuel oil lines are 1-1/2- and 2-in.-diameter Code Class 3 piping. These lines were installed in June 1980 after completion of the DGB surcharge program. They are attached to unistrut frames embedded in concrete piers. These supports are located at approximately 20-ft intervals. Both piping and supports are covered with approximately 2 ft of compacted fill and, as noted in SSER 1, are to be provided with tornado-missile protection. As noted in SER Section 1.12.10, these small-diameter lines are flexible and were connected after completion of the surcharge for the DGB. Thus, the applicant has shown that present and future settlement stresses should not be excessive. The staff agrees.

The control room pressurization lines are 1- and 4-in.-diameter Code Class 3 piping and were installed in 1981, after major fill settlements had occurred. The 1-in.-diameter penetration pressurization lines are Code Class 2 and have not yet been installed. The differential settlement effects in either type of pressurization line are, therefore, expected to be negligible and hence acceptable.

3.9.3.1.2 Staff Concerns Regarding Underground Piping

Initial staff concerns regarding the effects of differential soils settlements on buried piping were expressed in written questions pursuant to 10 CFR 50.54(f).

These questions and the applicant's responses are given in the applicant's "Responses to NRC Requests Regarding Plant Fill," which was incorporated into the FSAR by reference by Amendment 72. The questions requested information on the following:

- loss of continuous support under buried piping and resulting stresses not accounted for in the design (Question 16)
- adverse effect on safety-related structures, foundations, and/or equipment as a result of failure of buried piping (Question 17)

- assurance of compliance with Code criteria throughout the life of the plant and measures necessary for this assurance (Question 17)

- evaluations planned for the assurance of the previous item (Question 18)

- effects of the DGB surcharge program on piping in the vicinity of the DGB (Question 19)

- assurance that stress levels on seismic Category I piping components (e.g., pumps, valves, vessels, and supports) will be within their Code-allowable limits and that deformations on active pumps and valves are within operability limits (Question 20)

- protection of buried piping from heavy vehicular loads during construction and operation (Question 34)

- minimum rattlespace criteria and adequacy of actual rattlespace dimensions at building penetrations (Question 45)

In addressing the above concerns, the staff and its consultant reviewed data transmitted by the applicant in various submittals, meetings, and telephone conversations. During the course of the review, two site visits (one on February 27-28, 1980, and on July 29, 1982) were made and independent or confirmatory analyses and literature searches were conducted. Additionally, several of the analyses performed for the applicant by Bechtel were audited during the July 29, 1982 audit.

3.9.3.1.3 Design Criteria for Underground Piping

Section 3.9.3 of the SRP defines the design criteria and load combinations to be employed in the design of Code Class 1, 2, and 3 items.

Stresses in piping as a result of soil settlement are not addressed in SRP Section 3.9.3 or the ASME Code. The applicant proposed a design criterion of 3 S_c* that would have provided an acceptable conservative limit for their evaluation. Stress analyses based on the assumption that the existing deviations from the design configurations are due solely to differential settlement effects give stresses in excess of this 3 S_c criterion. However, sufficient uncertainties regarding the as-built configuration rendered these results inconclusive and, therefore, a combination of acceptable design criteria and remedial action was required. The criteria that were accepted were based on engineering judgment and consideration of adequate margins of safety. The remedial actions that were proposed by the applicant and accepted by the staff were reviewed to ensure that the underground seismic Category I piping is adequate to withstand adverse load combinations including soil settlement effects without loss of structural integrity or functional adequacy throughout the life of the plant.

To this end, the following design criteria were reviewed:

<u>Strength Criteria</u>: These criteria provide assurance that the overall cross-sections of piping are capable of resisting the forces and moments due to all

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*S_c = basic material allowable stress at minimum (cold) temperature, psi

loads imposed upon the piping over the life of the plant. These loads include pressure, thermal expansion, overburden and traffic, soils settlement, and seismic loads.

Buckling Criteria: These criteria provide assurance that local buckling and gross collapse will not occur.

Minimum Rattlespace Criteria: These criteria provide assurance that both local and gross overstressing of the piping and gross overstressing or distortion of piping components or attached equipment does not occur due to loads to be imposed during the life of the plant.

Nozzle and Other Interface Loads Criteria: These criteria provide assurance that the structural integrity or functional adequacy of attached components (e.g., pumps, valves, vessels, supports, etc.) associated with the seismic Category I piping will not be compromised over the life of the plant.

Criteria for Effects of non-Category I Piping: Because both seismic Category I and non-Category I piping are founded in the plant fill, these criteria provide assurance that failures of non-Category I piping have no detrimental effects on Category I piping ("II under I" problem).

The bases for acceptance of these criteria are discussed in the following paragraphs. As mentioned above, the criteria were based on engineering judgment and considerations of margins of safety. Each underground pipe was considered separately, and applicable criteria selected on the basis of anticipated settlement effects throughout the life of the plant. The following discussions are therefore not pipe specific, but general in nature.

Strength Criteria: In addition to satisfying the various applicable Code criteria, other load combinations that included the effects of soil settlement were considered in the formulation of these criteria. It was considered acceptable not to include settlement stresses in any existing Code load combination because (1) most of the total settlement anticipated over the life of the plant has already occurred, (2) the rate of future settlement is slow; (3) settlement, strain gage, and flow measurements will be monitored for the life of the plant, and (4) settlement-induced stresses are secondary stresses (in accordance with the Code classification of stresses). For settlement only, the acceptance criterion was that the stresses not exceed 3 S_c.

In cases where this criterion could not be satisfied, the effects of load combinations that could lead to catastrophic effects in a time that is short in comparison to the proposed monitoring frequency were considered. In particular, effects during the SSE plus Settlement combination were considered and provisions made for adequate margins of safety. Additionally, overburden and vehicular load effects were assessed relative to the margins of safety for existing Code criteria.

From the above, the following load combinations and criteria were required in addition to those required by the Code:

(a) $S_{ss} \leq 3.0\ S_c$ (1)

where

S_{ss} = stresses due to differential soil settlement only.

(b) If equation (1) cannot be satisfied, the total ovality due to an SSE plus soils settlement must be less than the maximum allowable ovality permitted for the diameter to wall thickness (D/t) ratio of the pipe (see "Buckling Criteria" below).

(c) $S_{SL} + S_{O/B} \leq 1.5\ S_h$ (2)

where

$S_{O/B}$ = stress due to overburden loads.

S_{SL} = stress due to sustained loads, as defined in the Code.

S_h = basic material allowable at operating temperature, psi.

(d) $S_{OL} \leq 1.8\ S_h$ (3)

where

S_{OL} = stress due to occasional loads, as defined in the Code, except that it includes bending or other stresses due to traffic loads.

Buckling Criteria: These criteria provide assurance that the underground piping will perform its intended function throughout the life of the plant. Buckling data were obtained from theoretical and experimental sources available in the current technical literature (see Appendix B, Ades, Bouwkamp, Bushnell, Jirsa, Reedy, Rodabaugh, Sorenson, Wood). These data were reviewed in depth by the staff and adapted for specifiying buckling criteria for underground piping. For this type of piping, these criteria are experessed specifically in terms of ovality and strain criteria. Ovality of a pipe is defined as

$$\text{Ovality} = \frac{D_{max} - D_{min}}{D}$$

where D = outside diameter of unovalized pipe

D_{max} = maximum outside diameter of ovalized pipe

D_{min} = minimum outside diameter of ovalized pipe

Based on these data, the allowable ovality adopted for the underground piping over the life of the plant is 4% based on a pipe with a ratio D/t equals 69, where t is the nominal wall thickness of the pipe, and a factor of safety of 1.5.

Where monitoring of pipe ovality is to be specified, the ovality will be determined by measuring pipe strains. The strains will be converted to ovality by means of certain formulae that the applicant has presented and the staff has accepted. For pipes with D/t less than 69, the permissible maximum ovality is greater than 4%, but the applicant has agreed to this limit.

Minimum Rattlespace Criterion: The conditions in the piping at building penetrations or other structural penetrations are dependent on the proposed remedial action for the associated piping in the plant fill and on the configuration of the piping at the penetrations. These conditions are therefore quite variable and required case-by-case studies for their resolution.

In general, assurance that minimum rattlespace will be adequate over the life of the plant was provided by the following methods:

(a) Analytical Method

This method was applied to the 36-in.-diameter SWPS penetrations. This criterion is given as

$$\delta_{R/S\ MIN} \equiv \delta_{R/S\ AVL} - (\delta_{SSE} + \delta_{SET}) \geq 0.5 \text{ in} \tag{4}$$

where $\delta_{R/S\ AVL}$ = currently available rattlespace

δ_{SET} = calculated pipe displacement due to future settlement

δ_{SSE} = calculated pipe displacement due to SSE

(b) Monitoring Method

This method will be applied at all SWS and BWST piping of a diameter less than 36 in. and requires that the total pipe strains, $\Delta\varepsilon$, near penetrations be limited over the life of the plant as follows:

$$\Delta\varepsilon \leq 0.40\% \tag{5}$$

In cases where piping was not rebedded, the additional condition that the measured minimum rattlespace dimension $\delta_{R/S\ MIN}$ be not less than 0.5 in. throughout the life of the plant will also be required; i.e.,

$$\delta_{R/S\ MIN} \geq 0.5 \text{ in.} \tag{6}$$

Rebedded piping was recentered at penetrations and the criterion given by equation (5) is a conservative safeguard against buckling. At nonrebedded piping penetrations, the criterion given by equation (6) will provide direct verification of sufficient rattlespace.

Other Criteria: No special requirements were developed for the criteria for "Nozzle and other Interface Loads" and "Effects of Non-Category I Piping." However, their application in this review was extended to include differential soils effects.

3.9.3.1.4 Evaluation and Resolution of Concerns

SWS Piping

(1) All the 36-in.-diameter piping will be replaced with thicker walled piping with D/t ratio of 57.6. This new piping is to be of ASME SA-672, Grade B70 Class 20 material. The potential for local buckling of the 36-in.-diameter piping is therefore considerably and acceptably reduced.

(2) The existing fill in which all of the 36-in.-diameter and portions of connected 26-in.-diameter piping is located is to be replaced with 250 psi K-KRETE down to elevation 610 ft. Settlement over the life of the plant for the K-KRETE fill is estimated to be 1.5 in., which is less than the 3.0 in. estimated for the existing fill. This fill replacement therefore reduces the adverse effects of differential soils settlement. Transition piping between the existing fill and the K-KRETE fill will be encased in 6-in.-thick Ethafoam 220 compressible material. The transition piping will further minimize the effects of differential settlement between the existing fill and the K-KRETE fill. Strain gage and settlement monitors will be located near the ends of two of the four transition piping runs to verify their effectivity.

(3) Analyses of the piping described in (1) and (2) above have been performed. The analyses were based on modification of existing designs involving installation of expansion joint assemblies in lines 36"-OHBC-19 and 36"-OHBC-15 located inside the SWPS near the SWPS penetrations and modified pipe support designs. These analyses were comprehensive and extended from 30 ft in the existing plant fill beyond the transition piping described in (2) to the first 30 ft of piping in the cooling pond for unanchored piping extending throughout the SWPS or to nozzles anchored to pump strainers within the SWPS. These analyses answer many of the concerns that were the subject of the July 1982 audit. The results of these analyses demonstrate compliance with (1) all applicable Code criteria, (2) the 3 S_c soils settlement criterion, (3) the analytical rattlespace criterion, and (4) the applicable interface or nozzle loads requirements. Additionally, the adequacy of the transition piping design was demonstrated.

(4) All the 26-in.-diameter piping, except that mentioned in (2) above, will be subject to pipe strain and settlement monitoring programs over the life of the plant. Additionally, settlement at Borros anchors located throughout the yard fill will be monitored over the life of the plant.

Ovality of the 26-in. piping, as determined through strain measurements, will be limited to 4% over the life of the plant. Current ovality is assumed to be the same as measured in 1981 because data from the Borros anchors indicate that negligible settlements have occurred since then. Allowable additional ovality is the difference between the 4% allowable and the 1981 ovality.

The strain gage and settlement monitoring program, together with the results of the "II under I" analyses, and the control of heavy vehicular loads in the yard, will ensure the functional adequacy of the 26-in. SWS yard piping.

The proposed monitoring programs described below will ensure adequate functionality rattlespace and equipment operability for the lines addressed in (5) through (8).

(5) Minimum rattlespace dimensions and pipe strains at the four 26-in.-diameter SWS piping auxiliary building penetrations will be monitored over the life

of the plant. Current minimum rattlespace dimension and maximum ovality at these penetrations are 1 in. and 2%, respectively. The minimum rattlespace dimension and total ovality over the life of the plant will be limited to 0.5 in. and 4%, respectively. In the case of lines 26"-0HBC-15 and 26"-0HBC-16, strain (ovality) monitoring will be implemented past the first elbow on the lines outside the penetrations.

Review of the continuation of the piping within the auxiliary building shows that no anchors or load-sensitive equipment is near the penetrations.

(6) Similar minimum rattlespace and strain monitoring will be implemented at 26-in.-diameter SWS piping valve pit penetrations. These lines were cut, refitted, and recentered in their penetrations; hence, current ovality can be assumed to be zero in satisfying the limits of (5) above.

(7) Strain monitoring programs only will be implemented at the lines 8"-1HBC-81, 8"-1HBC-2, 8"-2HBC-310, and 8"-2HBC-311 DGB penetrations of SWS piping lines. These lines will be or have been cut, refitted, and recentered in their penetrations as part of the rebedding program, hence the ovality assumption of (6) is also applicable to these lines. The current minimum rattlespace dimension for these lines is 1-1/8 in.

(8) Minimum rattlespace and pipe strain monitoring programs similar to those of (5) are also to be implemented at the 8"-2HBC-81, 8"-2HBC-82, 8"-1HBC-310, and 8"-1HBC-311 lines. These lines were pigged for diameter verification. Maximum additional ovality and minimum rattlespace dimension will be limited to 4% and 0.5 in., respectively. The current minimum rattlespace dimension for these lines is 5/8 in.

BWST Piping

(1) The portions of the four 18-in.-diameter BWST piping lines between the valve pits and the dike wall will be rebedded. No remedial actions are planned for the remaining portions of these lines. Stress analyses based on the profile data for these lines indicate that stresses satisfy the 3 S_c criterion. However, monitoring programs are to be implemented at the ends of the piping to address rattlespace concerns. Specifically, these are as described in (2), (3), and (4) below.

(2) Pipe strain will be monitored only at the valve pit penetrations. The lines will have been cut, refitted, and recentered in these penetrations. Maximum future ovality is therefore limited to 4%.

(3) Pipe strain and minimum rattlespace dimension will be monitored at the auxiliary building penetrations. The maximum additional ovality and minimum rattlespace dimension will be limited to 4% and 0.5 in., respectively, throughout the life of the plant. The current minimum rattlespace dimension is 1-7/8 in.

(4) The monitoring programs in (2) and (3) will address rattlespace concerns.

Other Lines

No remedial actions are planned for the remaining lines because they were or are to be installed after the most significant settlements have occurred.

3.9.3.1.5 Staff Conclusions

Based on review of Section 3.9.3.1.2 above, the staff finds that the proposed remedial actions for the safety-related buried piping designed to meet seismic Category I standards are such as to provide assurance that the effects of soils settlement will not impair the ability of the piping to withstand an earthquake affecting the site, or an upset, emergency, or faulted plant transient occurring during normal plant operation. The resulting combined stresses imposed will not exceed allowable stress and strain limits of the material as defined in the Code and modified by the special criteria defined in Section 3.9.3.1.3 of this SSER, for the materials of construction. The limits for the loading combinations considered provide a conservative basis for the piping to withstand the most adverse combination of loading events without loss of structural integrity or functional adequacy. The design load combinations and associated stress and strain limits imposed constitute an acceptable basis for design in satisfying applicable portions of GDC 1, 2, and 4.

3.9.3.1.6 Technical Specifications and Reporting Requirements

The applicant has committed to incorporate various items in Technical Specifications and procedures as a result of this review. Specific items are related to the following:

Demonstration Solution

(1) Pipe Strain and Settlement Monitoring: These requirements are discussed in Section 3.9.3.1.5 above. Additional pipe ovalization and settlement limits are to be incorporated in Technical Specifications.

(2) Soil Settlement Monitoring: Similar to (1).

(3) Yard Exclusion Zones: These zones will be designated on yard piping drawings along with maximum allowable loads and laydown times for heavy laydown loads or other heavy loads for these zones.

Reporting Requirements

(1) The alignment and ovality of the piping to be rebedded or replaced is to be documented and reported to the staff before and after these operations.

(2) Reporting requirements relative to Technical Specification limits are to be documented.

3.12 Corrosion Control on Buried Piping

This staff evaluation has been performed with the technical assistance of a staff consultant from the Brookhaven National Laboratory.

3.12.1 Protection of Carbon Steel Service Water and Diesel Fuel Piping From External Corrosion

All carbon steel piping used in the service water and diesel fuel lines is protected from corrosion by a combination of a primer paint and a wrapping to provide electrical insulation (as well as a physical barrier) between the piping and the corrosive environment. Procedures for both shop coating of piping and field coating of field welds are carefully specified to ensure that this piping will be protected from external corrosion. Furthermore, the completed piping was 100% inspected by Bechtel for defects in the coating and found by Bechtel quality control inspectors to be acceptable. The wrapping material consists of reinforced fiberglass followed by a Tapecoat Co. "CT" Tape Coat for the field-coated material. Both techniques are standard commercial practices for protecting carbon steel piping from groundwater attack. Field installation and backfill techniques were carefully specified to minimize damage to the coatings. These were also monitored by the Bechtel quality assurance department. An independent check of the condition of the pipe wrappings will be possible when the 36-in. pipes are excavated and replaced before startup of the plant.

The entire Midland site is protected by a galvanic protection system design to maintain all buried piping to a potential of 0.85 V negative to the copper/copper sulfate reference electrode. This is standard practice and ensures that should any defects develop in the protective coating of these pipes, localized corrosion will not occur. This galvanic protection system consists of an array of buried electrodes charged from a central rectifier, as well as zinc protective anodes that can be used both for controlling corrosion and for monitoring the effectiveness of the applied galvanic current protection system. The combination of a protective coating on the pipe surface and a sitewide galvanic protection system represent, in the staff's opinion, sound practice for minimizing the corrosion of the outer surfaces of the buried piping. Leaching tests on sand samples from the backfill used at the Midland site have shown only trace amounts of chlorides and a pH greater than neutral (8.6 to 8.9). This combination should minimize the extent of corrosion that might occur should the galvanic protection system or the pipe wrappings not perform their job. Therefore, the staff concludes that external corrosive attack on the buried carbon steel piping is unlikely to be significant at the Midland Plant site.

3.12.2 Effects of Settling on External Corrosion of Buried Carbon Steel Pipe

The pipe-coating materials, such as fiberglass wrap or a coal tar paper wrap, are inherently flexible and should not fail as a result of the amounts of strain that have occurred on the buried piping. The NRC has set maximum acceptable limits of 4% ovalization and approximately 0.5% strain on the piping. The protective wrap can "give" within these limits. Further, should flaws develop in this protective wrap, the galvanic protection system is specifically designed to prevent corrosion at such flaws. Therefore, it is not anticipated that significant localized corrosion of the carbon steel piping will occur as a result of the settling, this system would be protecting them from corrosion at the present time.

3.12.3 Pitting Corrosion of Buried Stainless Steel Lines

Buried stainless steel piping at the Midland site is not coated on the outside but is protected from corrosion through the galvanic protection system. During construction, the applicant reported that pitting had been observed in the stainless steel piping from the condensate storage tank. The utility's consultants examined this piping, and concluded that this corrosion was a highly localized pitting, present on only one side of the piping. In view of the good soil chemistry (cited in Section 3.12.1 of this SSER) at the Midland site, it seems unlikely that this pitting would have been caused by interaction between the piping and the soil before the galvanic protection system was activated. Subsequently, the utility's consultants suggested that these corrosion pits were caused by stray currents resulting from improper grounding during field welding of other components at the Midland site. The staff concurs that this is a likely explanation for the pitting attack. Selected lengths of buried stainless steel piping in the borated water storage tank injection lines are being excavated and examined to determine the condition of the external surface of this piping before the start of operation of the plant. Portions of the condensate storage lines have already been examined during the applicant's investigation. The staff concurs that this proposed inspection followed by replacement of any defective piping will ensure the integrity of these systems. The staff also concurs that the galvanic protection system now in effect will prevent any future external corrosion by the groundwater. The applicant also advised that proper grounding of field welding equipment is now in practice at the Midland site.

The settling stresses should not have an effect on the corrosion behavior of stainless steel piping, both because of the galvanic protection system and because stainless steels are inherently resistant to stress corrosion cracking in water (pH 8.5 to 9) low in chlorides at ambient temperatures.

Should the galvanic protection system become inoperative, corrosion at hypothetical flaws in the coating on the carbon steel pipes would not be serious for periods up to at least 6 months, because pitting depths would not exceed half the 1/16-in. corrosion allowance in this period of time. As noted above, the staff also would not anticipate significant corrosion of Type 304 stainless steel to occur in backfill material used at Midland in the same 6-month period. In addition, during any periods the galvanic protection system is inoperative, some protection would continue to be provided by the buried zinc anodes.

5 REACTOR COOLANT SYSTEM AND CONNECTED SYSTEMS

5.4 Component and Subsystem Analysis

5.4.4 Decay Heat Removal System

5.4.4.2 Cold Shutdown Capability

Manual Action Outside the Control Room

Section 5.4.4.2 of the SER provided the staff's conclusions regarding manual actions outside the control room necessary for achievement of cold shutdown at Midland. The SER noted that the Midland decay heat removal (DHR) system design requires local operator action to align the DHR suction valves from the reactor building sump to the reactor coolant system (RCS) hot leg before the system can be brought into service. The staff's review of the Midland DHR design was performed recognizing that manual action outside the control room in the absence of a postulated single failure is, in general, not consistent with RSB BTP 5-1 for Class 2 plants (i.e., plants with construction permits docketed before January 1, 1978 and operating license issuance scheduled on or after January 1, 1979). Two of the more significant factors of the staff's evaluation are (1) the time available for the action and (2) accessibility of the operator to the valve. The SER noted that review of the latter consideration was continuing.

Time Consideration

The design of the DHR system requires manual operator action outside the control room between 6 to 30 hours during a cooldown so the DHR suction valves can be aligned. Once the DHR system is aligned, it can be operated from the control room without further remote manual action. In view of the ample time available for operator action and the ability of the DHR system to be operated remotely once properly aligned, the staff concluded in the SER that the system meets the requirements of BTP RSB 5-1 and is therefore acceptable, subject to resolution of the accessibility item below.

Accessibility

The manual DHR valves are located in the lower level of the auxiliary building, six levels below the control room. The valves are equipped with reach-rods that pass through a concrete wall between the auxiliary building hallway and the room housing the manual valves to reduce the radiation exposure to the operator from radioactivity which might be contained in the DHR water. The SER noted that the applicant is required to provide an evaluation of the environment that might exist in the vicinity of the valve hand wheels and in the passages that must be traversed between the control room and the manual DHR valves.

This evaluation was provided in the applicant's letter dated June 7, 1982. The applicant evaluated the flooding hazard from failure of all water-containing tanks on the 568-ft level where the handweels are located.

These tanks were determined to produce a water depth of 10 in. in the corridor. The water could be drained using the auxiliary building sump located in the corridor, although the DHR valve handwheels would be accessible even for the flooded condition. The maximum radiation field within the corridor to which an operator might be exposed is 8.5 rems/hr. This condition is postulated for degraded core conditions in one unit 6 hours after the accident. Sufficient time would be available, however, for an operator to open the suction valve in one of the redundant DHR trains of the other unit to permit it to be brought to cold shutdown, if required, without exceeding the dose guidelines of 10 CFR 20.101 (i.e., 3 rems/quarter total whole-body dose). In the event of a fire in the auxiliary building, the flexibility in the required time for DHR actuation (6 to 30 hours) provides ample time to deal with any postulated fire and then to actuate the DHR system, if required.

The staff concludes that the Midland DHR system meets the cold shutdown requirements of BTP RSB 5-1 for Class 2 plants. This confirmatory issue, identified as item 8 in SER Section 1.8, is therefore closed with respect to manual action outside the control room. However, confirmatory item 7 identified in SER Section 1.8 and also discussed in SER Section 5.4.4.2 relative to required tests and analysis of the approach to cold shutdown under natural circulation remains open; it will be addressed in a subsequent SSER.

6 ENGINEERED SAFETY FEATURES

6.2 Containment Systems

6.2.7 Fracture Prevention of Containment Pressure Boundary

The SER noted, as a confirmatory issue, that certain information was needed from the applicant that was expected to confirm that the lowest temperatures to be experienced by the limiting materials of the reactor containment pressure boundary under the conditions cited by GDC 51 will be in compliance with the temperatures identified in the staff's analysis. This issue also was identified as item 13 in SER Section 1.8. The confirmatory information has been provided, in part, by the applicant's letter of March 30, 1982. The remaining information subsequently was provided by the applicant's letter of August 25, 1982. The staff has reviewed this information and finds that it demonstrates compliance with the temperatures identified in the staff's analysis and is acceptable. This confirmatory item is, therefore, closed.

6.3 Emergency Core Cooling System

6.3.1 Confirmation of Safety Margin in Design-Basis Loss-of-Coolant-Accident Calculations

As stated in SER Section 6.3, the NRC staff concluded that the Babcock & Wilcox (B&W) loss-of-coolant-accident (LOCA) evaluation model is in conformance with Appendix K to 10 CFR 50. Appendix K specifies conservative features and assumptions that are required for acceptance of LOCA evaluation models. Since the development of Appendix K, additional experimental data have become available, including semiscale experiments and data from the loss-of-fluid test (LOFT) facility. These data have been used in the development of advanced computer codes such as TRAC, which was developed by the Los Alamos National Laboratory (LANL).

To provide further confirmation that the Appendix K requirements are as conservative as originally intended, the NRC staff contracted with LANL to perform best estimate analyses of the large- and small-break sizes determined by the applicant to produce the most severe conditions of core uncovery and cladding heatup. Best estimate analyses assume expected values of decay heat, heat transfer, and fluid flow based on experimental data. Also, emergency core cooling systems (ECCSs) are assumed to be fully operational. Appendix K requires conservative decay-heat, heat transfer, and fluid-flow assumptions. The B&W LOCA model also assumes degraded ECCS flow to demonstrate redundancy as required by GDC 35. The peak cladding temperature results from the LANL analyses and those predicted by the B&W evaluation model are presented in Figures 6.1 and 6.2 of this SSER.

These results indicate that the applicant's large-break analysis is conservative by 850°F and that the small-break analysis is conservative by 650°F. Although

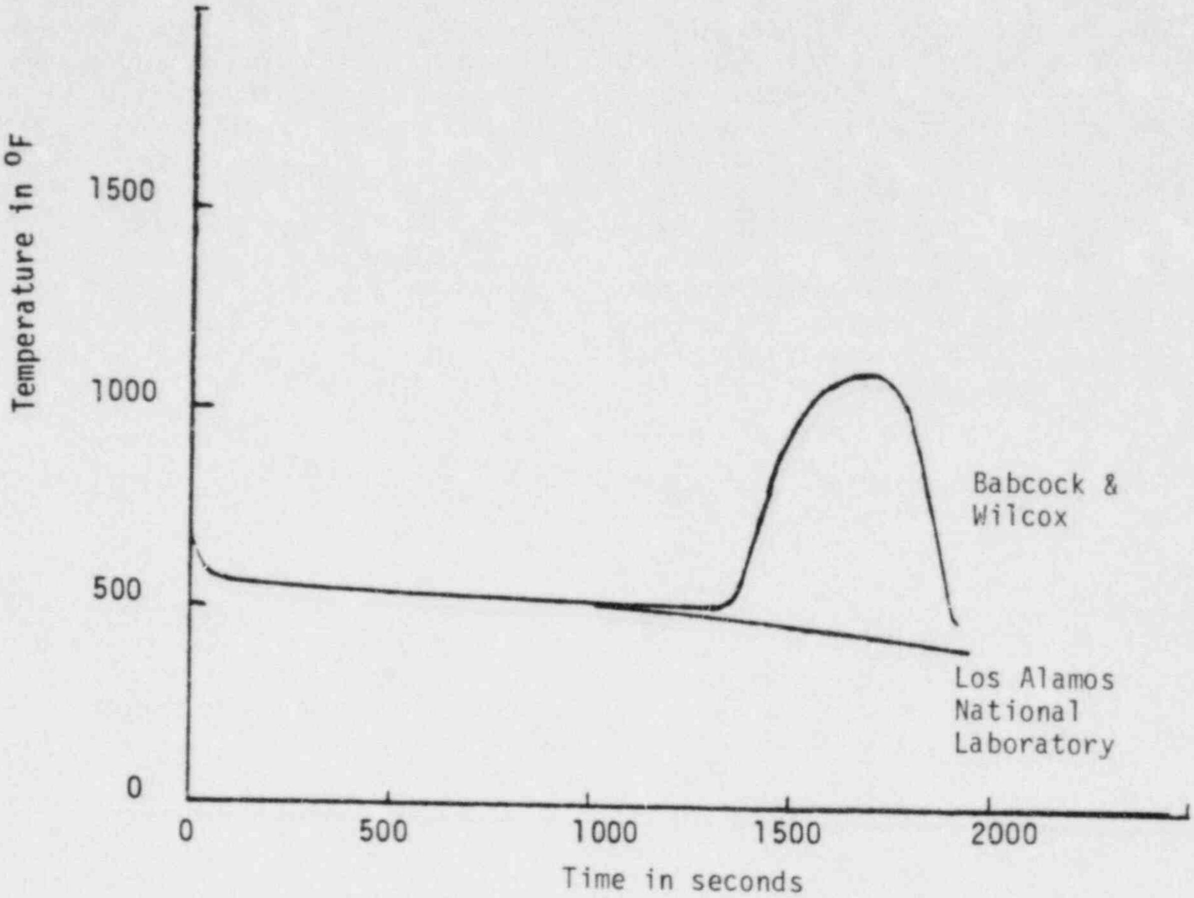

Figure 6.1 Peak cladding temperature for worst case small-break loss-of-coolant accident (0.07-ft^2 cold leg)

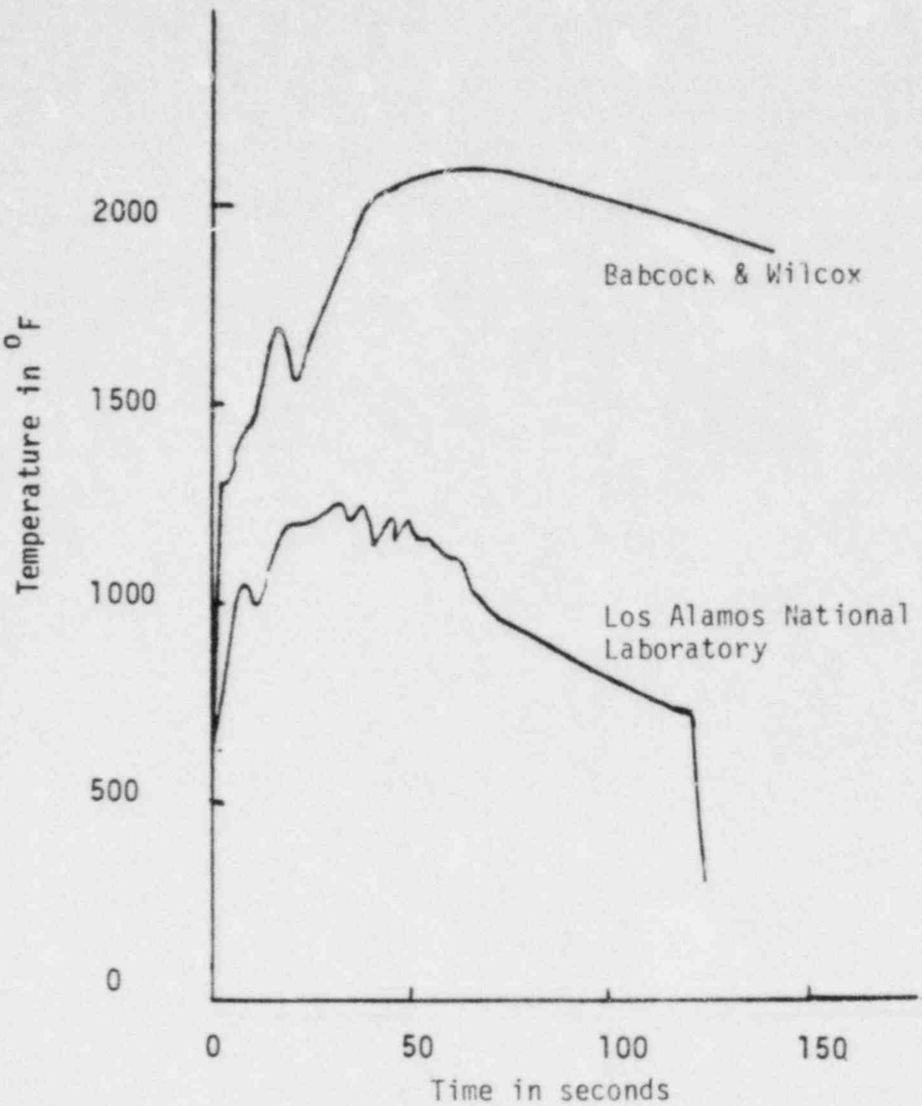

Figure 6.2 Peak cladding temperature for worst case large-break loss-of-coolant accident (double-ended cold leg)

these values are estimates, subject to the uncertainties of the computer models and actual initial and boundary conditions, they are nevertheless useful in demonstrating the relative margin that is provided by Appendix K conservatism. They also help confirm that these Appendix K assumptions provide the conservatism they were originally intended to provide.

Although the above assessment indicates that an acceptable margin is provided for the limiting small-break accident, the staff is still reviewing the B&W small-break model for less limiting small breaks that interrupt natural circulation and that are sufficiently small so that natural circulation decay heat removal is required. Section 6.3.4.1 of SSER 1 addresses the staff's concerns in this area.

9 AUXILIARY SYSTEMS

9.2 Water Systems

9.2.1 Service Water System

SER Section 9.2.1 presents staff acceptance of the Midland service water system (SWS). Details of the SWS design are described in FSAR Section 9.2.1, including FSAR Figure 9.2-2, Sheets 1 and 2. The SWS design includes provisions for supplemental cooling by a cooling tower during periods of high summer temperature. A 48-in.-diameter underground line is used to route the returning service water to the cooling tower in lieu of discharge to the emergency cooling water reservoir.

Inside the service water pump structure (SWPS) there are two seismic Category I butterfly valves that isolate the line (48"-OHBC-2/48"-OYJJ-1 on FSAR Figure 9.2-2) from the normal routing to the cooling pond.

SER Sections 1.12.10 and 3.9.3.1 identified the seismic Category I underground piping that was being reviewed by the staff because the plant fill supporting these pipes was inadequately compacted and is settling. Section 3.9.3.1.1 of this SSER notes that the underground seismic Category I piping at the Midland Plant previously including a segment of the 48-in.-diameter service water system line. The applicant has subsequently reclassified this segment nonseismic because its failure, in the applicant's opinion, would not represent a safety problem.

The staff has reviewed the consequences of failure of either the supply or return line between the cooling tower and the SWPS. The staff finds that a failure of either nonseismic line would not result in loss of essential SWS cooling, even assuming a single active failure. A failure of either line would not cause loss of net positive suction head to the SWS pumps because redundant seismic Category I makeup pumps automatically maintain adequate water level within the pump pit by pumping water from the SWPS forebay to the pump pit. Sluice gates between the pump pit and the forebay are designed to open automatically upon receipt of the signal of a low water level within the pump pit. This low level signal also causes an automatic transfer of the cooling function from the cooling towers to the cooling pond. All mechanical and electrical equipment involved with this transfer is redundant and is designed to seismic Category I requirements. The volume of water within the pump pit is sufficient to supply all four SWS pumps while the sluice gates to the forebay are opening.

On the basis of these findings, the staff concurs in the reclassification of the 48-in.-diameter line downstream of the butterfly valve.

16 TECHNICAL SPECIFICATIONS

This section of the SER identified 21 issues that must be included in the Technical Specifications as a condition of staff acceptance. Section 1.9 of the SER also identified several issues which may become license conditions, either in the form of a statement in the body of the Operating Licenses or as a Technical Specification appended to the licenses. Additional issues to be included in the Technical Specifications have been identified in this SSER. These additional issues are listed below and are discussed further in the sections of this SSER as indicated.

Item	SSER Sections
(22) Long-term monitoring of all seismic Category I structures	2.5.4.7, 3.8.3.5, 3.8.3.7
(23) Long-term settlement and strain monitoring of seismic Category I underground piping and supporting soil	2.5.4.7, 3.9.3.1.6
(24) Restriction on placement of heavy loads over buried piping and conduits	2.5.4.7, 3.9.3.1.6

17 QUALITY ASSURANCE

17.3 Quality Assurance Program

17.3.1 Quality Assurance (QA) Program for Remedial Soils Activities

As identified in Section 1.12 of this SSER, several remedial soils activities have been and are being implemented to correct foundation support problems resulting from inadequate compaction of safety-related ("Q") plant fills at the Midland site.

For the safety-related and nonsafety-related remedial soils activities, the applicant has committed to comply with the quality assurance requirements previously approved by the staff that are described in the applicant's report CPC-1-A, "Quality Assurance Program Manual for the Midland Nuclear Plant," and Bechtel's topical report BQ-TOP-1, Revision 1A, "Bechtel Power Corporation QA Topical Report."

The quality assurance (QA) program for the remedial soils activities is described in two Midland Project Quality Plans (MPQPs). MPQP-2, Revision 0, "Quality Plan for Remedial Soils Activities and Soils Related Work in Q Areas," describes the overall CPCo and Bechtel QA plan for remedial soils activities. MPQP-1, Revision 3, "Quality Plan for Underpinning Activities," describes in more detail the QA plans for the underpinning activities associated with the auxiliary building and SWPS. These plans, both bearing an effective date of July 26, 1982, were forwarded for staff review and approval by the applicant's letter of August 9, 1982.

Some of the more significant areas covered by these programs include:

(1) Placing underpinning supports beneath the SWPS and auxiliary building.

(2) Removing and replacing fill, and loading activities beneath the FIVP areas, auxiliary building EPAs, CT, and the turbine building.

(3) Installing monitoring system and monitoring structural response to underpinning activities.

(4) Installing, operating, and monitoring both permanent and temporary dewatering systems.

(5) Installing and operating the temporary freeze wall.

(6) Repairing the BWST foundations and releveling the tank.

(7) Rebedding or replacing underground service water and BWST piping

(8) Any construction work in soil materials, or placing, compacting, excavating, or drilling soil materials under or around safety-related structures and systems or within "Q" listed zones, as defined by Bechtel drawing C-45(Q).

The Midland Project Quality Assurance Department (MPQAD) is responsible for the review of design documents, procurement orders, and implementing procedures of the applicant, Bechtel, and subcontractors to ensure that the necessary quality requirements are specified, including quality control inspection call outs. Throughout the implementation phase, MPQAD is also responsible for overviewing and auditing the soils activities to verify that they are being carried out correctly in accordance with previously approved requirements.

The staff concludes that the QA program for the remedial soils activity meets the requirements of Appendix B to 10 CFR 50 and is acceptable.

APPENDIX A

CONTINUATION OF CHRONOLOGY

May 12, 1982	Letter to applicant regarding April 7, 1982 telephone conversation and April 16, 1982 meeting on buried piping.
July 6, 1982	Meeting with the applicant to provide staff comments on quality plans on underpinning and soils related activities (summary issued as Attachment 3 of applicant's letter of August 9, 1982).
July 7, 1982	ASLB issues Memorandum and Order (Reopening Record on QA Matters and Establishing Schedule for Prehearing Conference and Discovery).
July 8, 1982	ASLB issues Order concerning issuance of subpoenas of four individuals associated with the Government Accountability Project.
July 9, 1982	Letter to applicant requesting report on design adequacy and construction quality.
July 13, 1982	Letter to applicant forwarding SSER 1.
July 13, 1982	ASLB issues Memorandum (Telephone Conference Call of July 12, 1982).
July 14, 1982	Letter from applicant requesting approval of new seismic damping values for design basis.
July 19, 1982	Letter from applicant concerning security plan and safeguards contingency plan.
July 19, 1982	Letter to applicant forwarding draft SSER 2 on soils-related issues.
July 20, 1982	Letter from applicant concerning proposed organizational modification of the nuclear activities department.
July 21, 1982	Meeting with applicant to discuss the SER supplement regarding soils-related open items.
July 23, 1982	Letter from applicant concerning masonry walls and ultimate containment capacity information.
July 27, 1982	ASLAB issues Memorandum and Order dismissing Barbara Stamiris' appeal from ASLB April 30, 1982 Memorandum and Order.

July 27-30, 1982	Design audit on soils remedial activities.
July 28, 1982	ASLB issues Notice of Prehearing Conference. Conference will commence at 9:00 am, August 12, 1982, at Midland County Courthouse.
August 2, 1982	Letter from applicant concerning concrete wall crack repair program.
August 4, 1982	Meeting with applicant to discuss use of expansion anchors in masonry walls (summary issued August 9, 1982)
August 5, 1982	Letter from applicant providing information for the staff's independent confirmation piping analysis of decay heat removal and core flooding systems.
August 6, 1982	Letter from applicant concerning B&W steam generator auxiliary feedwater header design change.
August 9, 1982	Board notification (BN 82-75) issued regarding accident sequence precursor program report.
August 9, 1982	Letter from applicant forwarding Midland Plant Quality Plan for Underpinning (MPQP-1, Revision 3) and Midland Plant Quality Plan for Remedial Soil Activities and Soils-Related Work in Q Areas (MPQP-2, Revision 0).
August 9, 1982	Applicant issues Stop Work Order on soils work to review excavation done in Q-listed areas without required staff approval.
August 11, 1982	Letter to applicant concerning SER open item on makeup nozzle cracking.
August 11, 1982	Letter from applicant providing probabilistic analysis for tornado-missile damage to the auxiliary building.
August 11, 1982	Letter to applicant concerning steam generator auxiliary feedwater header modifications.
August 11, 1982	Meeting with applicant to discuss excavation beneath duct bank at freezewall intersection and relocation of fire lines.
August 12, 1982	Letter from applicant concerning revised small-break LOCA methods.
August 12, 1982	NRC Region III staff and applicant execute authorization procedure in contractor's work covered by the Licensing Board's Order of April 30, 1982.
August 12-14, 1982	Prehearing conference and ASLB site tour.
August 13, 1982	Letter from applicant concerning emergency planning relationship with Dow Chemical Co.

August 14, 1982	ASLB issues prehearing conference order ruling on new contentions and documenting other determinations reached at the August 12-14, 1982 prehearing conference.
August 16, 1982	Memorandum from W. D. Shafer of NRC Region III transmitting NRC and CPCo Work Authorization Procedure that provides a mechanism for Region III review and authorization of soils-related work.
August 16, 1982	Letter to applicant concerning evaluation of program for control of heavy loads.
August 17, 1982	Meeting with applicant to discuss his request for a construction release on the soils remedial program (summary issued September 7, 1982).
August 18, 1982	Letter to applicant concerning TMI Action Item II.B.5.
August 19, 1982	ASLB supplements Prehearing Conference Order of August 14, 1982.
August 23, 1982	Meeting with applicant to discuss nearby explosive hazards (summary issued September 2, 1982).
August 24, 1982	Letter from applicant submitting Amendment 107 to application for CPs and OLs consisting of revision 5 to the Midland Plant Security Plan and revision 2 to the Safeguards Contingency Plan.
August 25, 1982	Letter from applicant forwarding Certified Material Test Reports.
August 25, 1982	Letter from applicant providing further response to staff request for analysis of ultimate containment capability.
August 26, 1982	Meeting with applicant to discuss the effectiveness of the quality assurance and quality control programs.
August 26, 1982	Letter from applicant providing conceptual design for reactor vessel head vent and requesting exemption until first refueling.
August 26, 1982	Letter to applicant concerning the Independent Safety Engineering Group role described in NUREG-0737.
August 27, 1982	Letter from applicant providing revised pump and valve inservice testing plan.
September 1, 1982	Letter from applicant providing radiological consequences of seizure of reactor coolant pump shaft.
September 2, 1982	ASLAB issues Memorandum and Order regarding telephone conference call of September 1, 1982.

September 2, 1982	Meeting with applicant regarding quality assurance improvement measures.
September 3, 1982	Letter from applicant providing stress results for buried SWS piping and location of pipe monitoring stations.
September 8, 1982	Meeting with applicant on quality assurance program improvements.
September 8, 1982	Letter from applicant transmitting Nuclear Accident Appendices to the Bay, Saginaw, and Midland County Emergency Preparedness Plan.
September 9, 1982	Atomic Safety and Licensing Appeal Board issues Decision (ALAB-691) affirming Board's decision not to impose sanctions, deferring review of the radon issue, and denying or dismissing certain motions and appeals.
September 9, 1982	Letter from applicant announcing reorganization of the Midland Project QA Department for soils remedial work.
September 10, 1982	Meeting with applicant on plant physical security plan (summary issued September 22, 1982).
September 10, 1982	ASLB Memoradum and Order regarding telephone conference call of September 10, 1982, establishing revised hearing dates of October 26-29 and November 1-5, 1982, and tenative date of October 30, 1982.
September 14, 1982	Letter from applicant requesting information from the staff regarding synergistic effects of low radiation dose rates on cables.
September 16, 1982	Board notification (BN 82-94) issued regarding the Zack Co.'s 10 CFR 21 report on welder records discrepancies.
September 17, 1982	Letter from applicant discussing implementation of the QA program for soils remedial work and summarizing meeting discussions of September 2, 1982.
September 17, 1982	Letter from applicant discussing implementation of the total Midland QA Program and summarizing efforts since NRC meeting of September 2, 1982.
September 17, 1982	ASLB Memorandum and Order providing opportunity to respond to positions of the applicant and the staff on certain rewritten contentions.
September 21, 1982	Letter from applicant regarding cooling pond dike inspection.
September 24, 1982	Board notification (BN 82-93) issued regarding semi-scale test results.

September 28, 1982 — Board notification (BN 82-98) issued regarding QC requalification program.

September 28, 1982 — Board notification (BN 82-90) issued regarding welds in main control panels.

APPENDIX B

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APPENDIX D

ABBREVIATIONS

ACRS	Advisory Committee on Reactor Safeguards
ASLB	Atomic Safety and Licensing Board
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
AWWA	American Waterworks Association
B&W	Babcock & Wilcox
BTP	Branch Technical Position
BWST	borated water storage tank
CPCo	Consumers Power Company
CT	control tower
CWIS	circulating water intake structure
DGB	diesel generator building
DHR	decay heat removal
EPA	electrical penetration area
ETEC	Energy Technology Engineering Center
FIVP	feedwater isolation valve pit
FSAR	Final Safety Analysis Report
GDC	General Design Criterion
LANL	Los Alamos National Laboratory
LOCA	loss-of-coolant accident
LVDT	linear variable differential transducer
LOFT	loss-of-fluid test
MSL	mean sea level
OBE	operating basis earthquake
OM+OL	refers to consolidated hearing on Order on Modification and Operating Licenses
RBA	railroad bay area
RCS	reactor coolant system
RG	Regulatory Guide
SAI	Science Applications, Inc.
SER	Safety Evaluation Report
SPT	standard penetration test
SSE	safe shutdown earthquake
SSER	Supplement to the Safety Evaluation Report
SWP	service water pump
SWPS	service water pump structure
SWS	service water system
TB	turbine building

APPENDIX E

NRC STAFF CONTRIBUTORS AND CONSULTANTS

This supplement is a product of the NRC staff and its consultants. The staff members and consultants listed below were principal contributors to this report.

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APPENDIX H

CONTINUATION OF ERRATA TO MIDLAND PLANT SAFETY EVALUATION REPORT

SER Section	Page	Change
2.4.4	2-20	In first sentence of sixth full paragraph, change "erosion on" "overtopping of."
15.5.5	15-25	In Section title, change "Instrument" to "Small." In last sentence of the first paragraph, change "the accident and" to "for this accident the."
Table 15.8	15-26	In title of table, change "instrument" to "small."
15.7.6	15-33	Delete third paragraph (the subject is covered by Section 15.7.4).

APPENDIX I

UNDERPINNING CONSTRUCTION PROGRAM

The purpose of this appendix is to summarize the construction steps planned to achieve the remedial underpinning beneath the auxiliary building and the SWPS. This construction summary is based on the applicant's letter of April 19, 1982, and is supplemented by recent Bechtel and contractor drawings provided to the staff during the course of its review. Although preliminary, these drawings represent the planned approach on which staff review and approval have been based.

1 AUXILIARY BUILDING

The auxiliary building underpinning work is to be accomplished in four construction phases, which are identified in the following paragraphs as Phases I through IV. Construction support measures include control of groundwater and preparation of vertical access shafts. Underpinning construction consists of installing temporary underpinning, followed by permanent underpinning. Upon completion of this work, certain structural strengthening will be performed.

1.1 Groundwater Control

At the start of underpinning work, the groundwater level will be at about el 600 ft. Because this work will extend at least 29 ft below that level, control of groundwater level is necessary.

The underpinning work is in a location with limited access, bounded by the two containment buildings, the main auxiliary building, and the turbine building. In the immediate construction area, groundwater will be removed by pumping from dewatering wells.

To reduce recharge of groundwater into the underpinning excavation area, an underground freeze curtain dam has been constructed. The layout of the dam can be seen in the site photograph at the beginning of this SSER. The dam was formed by drilling a line of boreholes at approximately 4.5-ft spacing and circulating a coolant at low temperatures through pipes in the boreholes. The coolant freezes the soil in a narrow strip along the line from el 610 ft down to the bottom of the borehole that terminates in the undisturbed glacial till. The frozen soil acts as a dam and reduces subsequent seepage of groundwater from the pond side toward the underpinning construction area. The freeze curtain dam is formed in permeable sandy soils and clays and silts that exist above the glacial till and below el 610 ft. The safety-related utilities crossing the freeze curtain dam are isolated by an excavation that extends below their foundation level to eliminate any potential heave of the ground due to freezing operations.

The existing dike with the clay cutoff along the western edge of the power block area forms a part of the underground dam. The effectiveness of the dewatering system will be monitored by measurements of the groundwater levels using piezometers located on either side of the freeze wall and dike.

1.2 Access Shaft

Immediately east and west of the two FIVPs and adjacent to the turbine building, vertical shafts were constructed to provide access for workers and equipment for the underpinning work. Each of the two shafts is about 16 ft by 26 ft in area.

As indicated in Figures I.1 and I.2 of this SSER, the shafts will be excavated in three stages. At present, they have been excavated to el 609 ft to eventually permit installation of the initial underpinning piers beneath the adjacent turbine building basemat. These piers will constitute permanent underpinning for the turbine building along its northern boundary. When the initial turbine building underpinning is completed, the access shafts will be lowered to el 600 ft to provide access for excavation beneath the FIVPs. After all temporary underpinning is completed for the FIVPs and EPAs, the two access shafts will be gradually lowered in the third and final stage from el 600 ft to el 571 ft. At that time, a level working surface extending into the shafts will be constructed for the general excavation and removal of soil down to el 571 ft beneath the FIVPs, EPAs, and CT.

The shafts are constructed using standard excavation methods and utilize soldier piles, wales, and lagging.

NRC concurrence for installation to el 609 ft, which is the foundation level of the FIVP, auxiliary building, and turbine building, was indicated by letter dated November 24, 1981. This activity constituted "Phase I" of the construction logic for auxiliary building underpinning. As part of its technical approval of "Phase II," by letter of May 25, 1982, the staff has approved the applicant's plans for deepening the access shafts.

1.3 Temporary Underpinning

To construct the permanent underpinning, it is necessary first to install a temporary underpinning system to support the FIVPs and portions of the turbine building, EPAs and CT. The temporary underpinning system is shown on Figures I.1 and I.2 of this SSER.

The following is a summary of the present plans for the construction sequence of the temporary underpinning on the east side. The sequence for the west side is similar. Variations in the present construction sequence may be implemented, subject to prior NRC approval, as early construction data are obtained and assessed.

With the access shafts constructed to el 609 ft and the freeze wall activated, the next step is the start of Phase II work; it begins by providing support to the turbine building near the EPA by constructing Piers E-9 and E-12. The completion of these turbine building piers is necessary to construct the tunnel/drift under the turbine building and to access the first support, Pier E-8, for the EPA.

Pier E-8 will be completed as part of Phase II. At the start of Phase III, the first excavation under the EPA will be begun to install the needle beams needed to provide the first support for the EPA. The completion of Pier E-8 and the needle beams is a significant milestone in the temporary underpinning operation because once the weight of the EPA is supported, any loss of soil support under the EPA is less critical. Pier E-8 and the needle beams must be in place before the tunnel under the turbine building is extended easterly to access the first corner Pier E-1 of the control tower. While the access tunnel is being extended, additional piers to support the turbine building columns are constructed.

The corner Pier E-1 of the control tower will be completed and jacked next. After completion of the control tower corner piers, the remaining control tower and EPA temporary underpinning piers are simultaneously constructed.

With completion of the temporary underpinning piers, the weight of the EPA and control tower is completely supported and mass excavation under the EPA and control tower will begin. For performing the mass excavation, the access shaft will be extended to el 571 ft.

1.4 Permanent Underpinning

Upon completion of mass excavation, the permanent wall under the EPAs and the permanent section of the wall in the control tower area will be constructed. At this stage, compacted backfill will be placed below the FIVP area and a new slab will be poured to el 600 ft. Construction of the permanent walls and replacement of the FIVP foundation constitute Phase IV work.

After completion, jacks will be placed on the permanent wall. Jacking forces will be transferred from the temporary to permanent walls in stages. Adjustments will be made until all the load is transferred from the temporary to the permanent underpinning and the wall has reached the final design jacking load. The slab under the FIVP foundation also will be jacked against the FIVP to transfer the load from the temporary steel support to the new slab.

The present plans for the permanent wall load transfer jacking sequence are shown on Figures I.3 of this SSER. Variations in the present plan may be implemented, subject to prior NRC approval, as early construction data are obtained and assessed.

Jacking loads will be held on the permanent underpinning and the settlements monitored. When the settlement rate has reached a predetermined value, the jacking load will be locked off. The permanent underpinning walls will be connected to the existing structure by grouting and the gaps filled with grout. For the FIVP, the area between the new slab and the FIVP existing foundation slab will be filled with lean concrete. At this stage, the excavation will be backfilled with fill or lean concrete and the permanent underpinning will be complete.

1.5 Building Modification

Upon completion of underpinning, and before fuel load, the applicant will strengthen an existing slab at el 659 ft between the control tower and spent fuel pool at the operating floor level. This strengthening is needed to resist loads during a seismic event.

2 SERVICE WATER PUMP STRUCTURE

The remedial action for the SWPS generally consists of installing a permanent, continuous underpinning wall under the foundation of the overhang portion of the structure (see Figure 2.9 of this SSER). The wall transfers the loads of this part of the structure through the fill to the competent undisturbed natural foundation soils. The wall is connected to the existing structure. The sequential numbering of the wall's underpinning piers as shown on Figure 2.9 coincide with the actual construction sequence that will be followed to complete the SWPS underpinning work.

2.1 Temporary Post-Tensioning

As the initial construction step, a temporary post-tensioning system, whose design was based on the American Concrete Institute (ACI) Code 318 and American Institute of Steel Construction (AISC) codes, was installed to apply a compressive force to the upper part of the building along the east and west exterior walls. This post-tensioning compensates for the loss of buoyancy, which results in additional forces on the overhang, when the construction site dewatering is installed.

2.2 Groundwater Control

At the start of the underpinning work, groundwater level will be about el 600 ft. Because this underpinning will extend at least 15 ft below this level, the control of groundwater is necessary.

The groundwater level will be lowered below el 585 ft by using approximately 65 temporary dewatering wells located both inside the SWPS and the adjacent CWIS and outside the SWPS along its exterior perimeter. As part of the temporary dewatering procedure, piezometers will be installed to monitor the groundwater level. These wells will be sealed after the underpinning wall is completed.

Staff concurrence for installation and operation of construction dewatering and piezometers was indicated by letter dated April 2, 1982.

2.3 Access

To construct the underpinning piers, which are below the existing foundations, access is needed from the grade elevation. This access is provided from the outside of the building by open excavation for the piers on the north and the east walls. The access for the piers on the west wall, which is adjacent to the CWIS, is provided by an access shaft from the grade and a tunnel under the base slab for the overhang portion of the SWPS.

2.4 Underpinning

The underpinning wall is constructed in small sections (piers) which are structurally tied together to form a continuous wall. The piers are constructed by hand digging pits and filling them with concrete. After a pier is completed, the load from the structure is transferred by jacking to a predetermined value known as initial jacking load.

During the underpinning operation, to protect the existing structure, only small portions of supporting soil will be removed, and these portions will be replaced with piers of greater load-bearing capacity. In addition, the structure will be monitored frequently for strains and differential settlement to ensure that these remain below predetermined limits.

The first piers to be constructed are approximately 30-ft deep and 5 ft by 4 ft in cross-sectional area. These piers are constructed in hand-dug sheeted pits located at each corner of the overhang. After the subgrade for these pits is inspected and approved by the resident geotechnical engineer, reinforcement, subgrade settlement and stress monitoring instrumentation, and anchor bolt assemblies to tie the pier to the underside of the slab are installed. The piers are then encased with concrete. An initial jacking load is applied to the overhang from jacks placed on the pier tops. The completion of these piers is significant to the underpinning operation, because at the end of this stage, the weight of the overhang can safely be supported on these piers without depending on the fill. Therefore, the loss of fill support would be less critical after this stage.

After the corner piers are completed, the remaining piers, except four sections on the east and west walls, are completed based on a predetermined sequence. At this stage the building is supported by initial jacking loads.

Stress monitoring instrumentation will be installed in designated piers. The piers are tied together with threaded reinforcing bar couplers and shear keys to form a continuous underpinning wall.

The final jacking loads are applied after No. 10 piers (see Figure 2.9) are constructed and the underpinning wall has progressed to within 6 ft of the vertical interface with the existing structure. Settlements caused by this load are monitored. When the resident geotechnical engineer determines that the settlement has decreased to a predetermined rate, the load is transferred from the jacks to wedges positioned between the top of the piers and the underside of the overhang, and the jacks are removed. The No. 11 piers are poured, encasing dowel bars that were previously drilled and grouted into the vertical face of the existing structure, thereby connecting the underpinning wall to the existing structure. The space between the top of the underpinning wall and the underside of the base slab is filled with nonshrink grout, and previously placed anchor bolt assemblies are tightened. The underpinning wall is connected to the structure at both the vertical and horizontal interfaces. The No. 12 piers are then constructed, completing the underpinning wall.

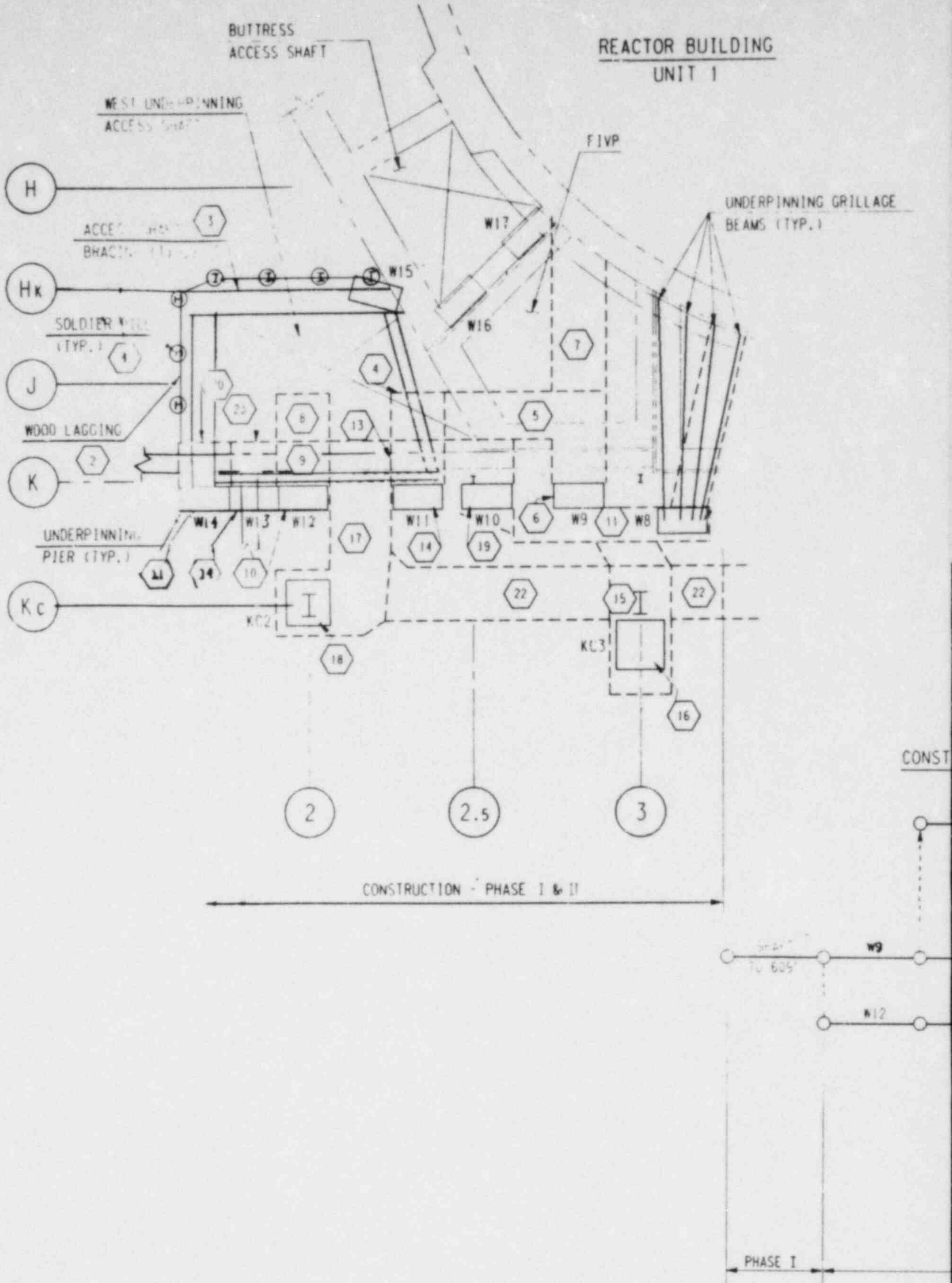
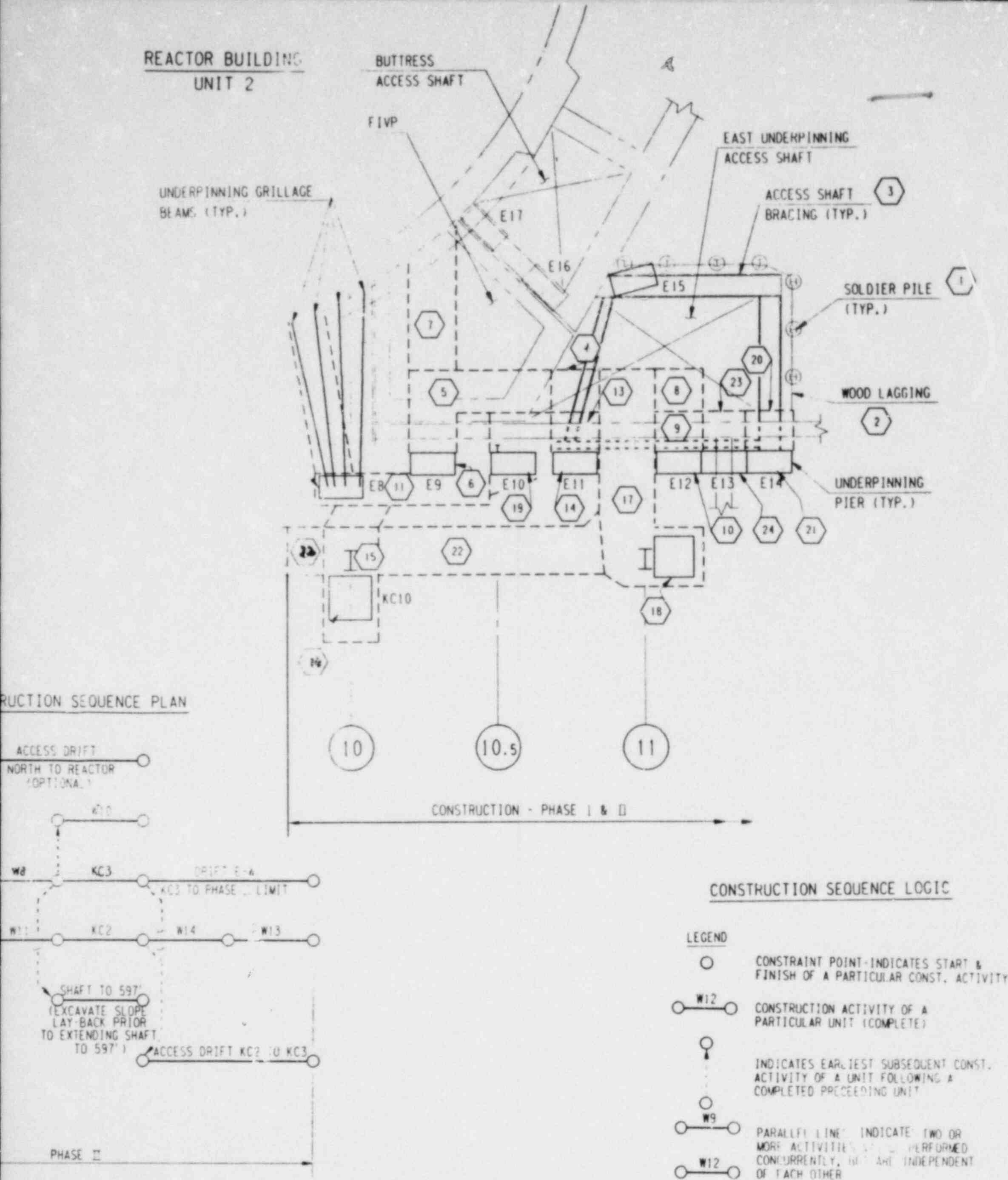


Figure 1.1 Auxiliary building underpin
(Source: Bechtel drawing 7



ning, construction sequence plan and logic, Phases I and II
720-C-1418(Q), Rev. A)

NOTES FOR FIGURE I.1

PHASE I - CONSTRUCTION EAST AND WEST ACCESS SHAFTS TO ELEVATION 609.0

1.1 ACCCESS SHAFTS TO ELEVATION 626'0" (NON Q)
DRAWING NO'S C-1420 and C-1421

⟨1⟩ 1.1.1 Drill and install soldier piles to elev. 561'-0"

⟨2⟩ 1.1.2 Excavate incrementally and install lagging to elev. 626'-0"

⟨3⟩ 1.1.3 Install access shaft bracing level "A" at elev. 629'-0"

1.2 EXTEND ACCESS SHAFTS TO ELEVATION 609'-0" (NON Q)
DRAWING NO'S C-1420 and C-1421

⟨2⟩ 1.2.1 Excavate incrementally and install lagging to elev. 609'-0"

⟨3⟩ 1.2.2 Install access shaft bracing level "B" at elev. 613'-0"

PHASE II - CONSTRUCT PHASE II PIERS AND EXTEND ACCESS SHAFTS TO ELEV. 597'-0"

2.1 PIERS E/W 9

⟨4⟩ 2.1.1 Excavate incrementally and lag approach pits adjacent to piers E/W 11

⟨5⟩ 2.1.2 Excavate, brace and lag approach drifts to piers E/W 9

⟨6⟩ 2.1.3 Excavate incrementally and lag shafts for piers E/W 9

2.1.4 Place rebars and concrete piers E/W 9

2.1.5 Jack design loads into piers E/w 9

⟨7⟩ NOTE: OPTIONAL ACCESS DRIFT NORTH TO REACTOR MAY PROCEED AT THIS TIME.

2.2 PIERS E/W 12

⟨8⟩ 2.2.1 Excavate incrementally and lag approach pits adjacent to piers E/W 12

⟨9⟩ 2.2.2 Excavate, brace and lag approach drifts to piers E/W 12

⟨10⟩ 2.2.3 Excavate incrementally and lag shafts for piers E/W 12

2.2.4 Place rebars and concrete piers E/W 12

2.2.5 Jack design loads into piers E/W 12

NOTES FOR FIGURE 1.1 Continued

2.3 PIERS E/W 8

11 2.3.1 Excavate, brace and lag approach drifts to piers E/W 8

12 2.3.2 Excavate incrementally and lag shafts for piers E/W 8

2.3.3 Place rebars, instrumentation and concrete piers E/W 8

2.3.4 Jack temporary loads into piers E/W 8

2.4 PIERS E/W 11

13 2.4.1 Excavate, brace and lag approach drifts to piers E/W 11

14 2.4.2 Excavate incrementally and lag shafts for piers E/W 11

2.4.3 Place rebars and concrete piers E/W 11

2.4.4 Jack design loads into piers E/W 11

2.5 EXCAVATE SLOPE LAY-BACK AT TOP OF SHAFT

2.6 EXTEND ACCESS SHAFTS TO ELEV. 597' - 0"

2 2.6.1 Excavate incrementally and install lagging to elev. 597'-0"

2.6.2 Install access shaft bracing level "C" to elev. 599'-6"

2.7 PIERS (W)KC3 AND (E)KC10

15 2.7.1 Extend approach drifts from piers E/W 8 south to piers KC3 and KC10

16 2.7.2 Excavate incrementally and lag shafts for piers KC3 and KC10

2.7.3 Place rebars and concrete piers KC3 and KC10

2.7.4 Jack design loads into piers KC3 and KC10

2.7.5 After KC3 and KC10 are loaded, the temporary supports at piers E/W 8 may be removed.

2.8 PIERS (W)KC2 and (E)KC11

17 2.8.1 Excavate, brace and lag access drifts from between piers E/W 11 and E/W 12 to piers KC2 and KC11

18 2.8.2 Excavate incrementally and lag shafts for KC2 and KC11

2.8.3 Place rebars and concrete piers KC2 and KC11

2.8.4 Jack design loads into piers KC2 and KC11

NOTES FOR FIGURE I.3 Continued

2.9 PIERS E/W 10

19 2.9.1 Excavate incrementally and lag shafts for piers E/W 10

2.9.2 Place rebars and concrete piers E/W 10

2.9.3 Jack design loads into piers E/W 10

2.10 PIERS E/W 14

20 2.10.1 Excavate incrementally and lag approach pits to piers E/W 14

21 2.10.2 Excavate incrementally and lag shafts for piers E/W 14

2.10.3 Place rebars and concrete piers E/W 14

2.11 ACCESS DRIFTS EAST - WEST

22 2.11.1 Excavate, brace and lag access drifts between piers (W)KC2 and KC3 and piers (E)KC11 and KC10

Extend E/W access drifts to limits of phase II

2.12 PIERS E/W 13

23 2.12.1 Excavate incrementalmly and lag approach pits to piers E/W 13

24 2.12.2 Excavate incrementally and lag shafts for piers E/W 13

2.12.3 Place rebars and concrete piers E/W 13

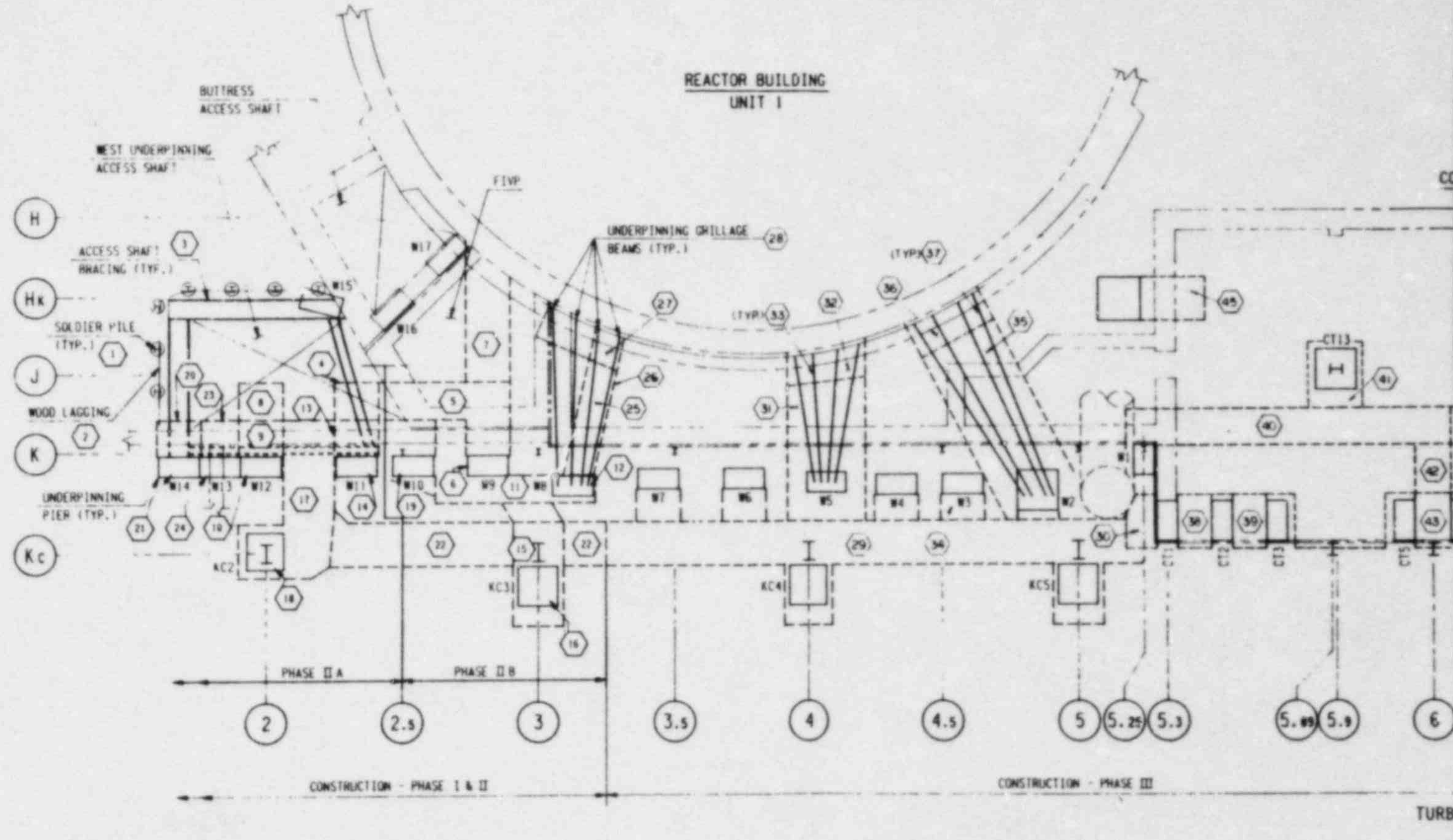
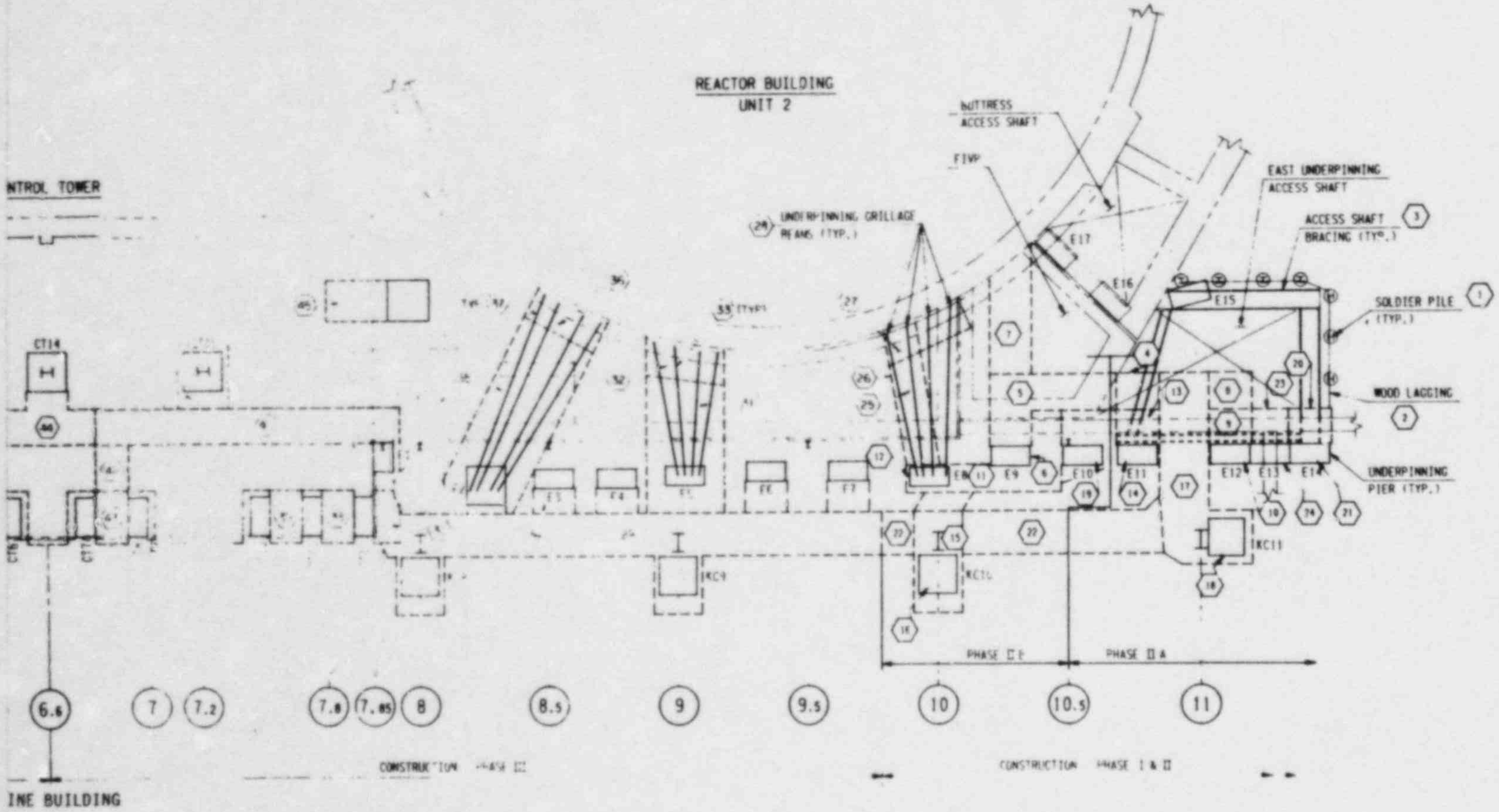


Figure I.2 Auxiliary building underpinning, con
(Source: Bechtel drawing 7720-SK C-



struction plan and logic, Phases III and IV
-0101, Revision 0, preliminary)

NOTES FOR FIGURE I.2

PHASE III

3.1 Grillage at E/W 8

3.1.1 Excavate and lag drift 25 and transform to bulkhead 26

3.1.2 Mass excavate shaft FIVP and edge EPA to bulkhead from el 609 to 601

3.1.3 Excavate and lag pit and install grillage columns from el 601 to 591 27

3.1.4 Install grillage beams 28 and jack design load

3.1.5 At EPA, excavate and lag shaft from el 601 to 597 and install Level C wall

3.2 Drift from Column 3.25/9.75 to 5.27/7.82 with sets and lagging 29 at el 601

3.3 Pier CT-1/12

3.3.1 Excavate and lag approach pit 30 from el 601 to 594

3.3.2 Excavate and lag shaft CT-1/12 and excavate bell

3.3.3 Place rebar and concrete CT-1/12 to el 594

3.3.4 Install load transfer frame from el 594 to 602 and jack design load

3.4 Pier and grillage at E/W 5

3.4.1 Excavate drift 31 with steel sets and lagging

3.4.2 Excavate and lag shaft and excavate bell for E/W 5

3.4.3 Rebar and concrete E/W 5

3.4.4 Excavate and lag pit and install grillage columns from el 601 to 591 32

3.4.5 Install grillage beams 33 and jack design load at EPA and turbine building

NOTES FOR FIGURE I.2 Continued

3.5 Pier at E/W-2

3.5.1 Excavate drift with steel sets and lagging

3.5.2 Excavate and lag shaft and excavate bell E/W-2

3.5.3 Place rebar and concrete E/W-2

3.5.4 Jack design load to turbine building

3.6 Pier at E/W-3

3.6.1 Excavate drift ⟨34⟩

3.6.2 Excavate and lag shaft and excavate bell E/W-3

3.6.3 Place rebar and concrete E/W-3

3.6.4 Jack design load to turbine building

3.7 Grillage at E/W-2

3.7.1 Excavate drift ⟨35⟩

3.7.2 Excavate pit and install grillage columns from el 601 to 591 ⟨36⟩

3.7.3 Remove jack load at E/W-2 after Item 3.6.4 is complete

3.7.4 Install grillage at E/W-2 ⟨37⟩

3.7.5 Jack design load at EPA and turbine building

3.8 Pier CT-2/11

3.8.1 Excavate drift ⟨38⟩ with steel sets and lagging

3.8.2 Excavate and lag shaft and excavate bell CT-2/11

3.8.3 Install rebar and concrete CT-2/11 to el 594

3.8.4 Install load transfer frame from el 594 to 602 and jack design load

3.9 Pier CT-3/10

3.9.1 Excavate drift ⟨39⟩ with steel sets and lagging

3.9.2 Excavate and lag shaft and excavate bell CT-3/10

3.9.3 Place rebar and concrete CT-3/10

3.9.4 Jack design load

NOTES FOR FIGURE I.2 Continued

3.10 Rebar and concrete piers CT-1/12 and CT-2/11 between el 594 and 600

3.11 Excavate drift 40 with steel sets and lagging

3.12 Pier CT-13/15

3.12.1 Excavate drift 41 with steel sets and lagging

3.12.2 Excavate and lag shaft and excavate bell CT-13/15

3.12.3 Rebar and concrete CT-13/15

3.12.4 Jack design load

3.13 Piers CT-5/8 and 6/7

3.13.1 Excavate and lag approach pit 42 and excavate drift 43 in steel sets and lagging

3.13.2 Excavate and lag shaft and excavate bell CT-5/8

3.13.3 Rebar and concrete CT-5/8

3.13.4 Excavate and lag shaft and excavate bell CT-6/7

3.13.5 Rebar and concrete CT-6/7

3.13.6 Jack design loads CT-5/8 then CT-6/7

3.14 Pier CT-14

3.14.1 Excavate drift 44

3.14.2 Excavate and lag shaft and excavate bell CT-14

3.14.3 Rebar and concrete CT-14

3.14.4 Jack design load

3.15 Excavate from el 609 to 602 between Column Lines 5.3/7.8 and 6.6, and between H and K.5

3.16 Excavate from el 602 to 591 between Column Lines 5.3/7.8 and 6.6, and between H and K.5, and lag between CT-Piers at K_c

3.17 Support utility banks in control tower area as encountered in excavation

3.18 Install top level struts in control tower at el 598

NOTES FOR FIGURE I.2 Continued

3.19 Piers E/W 1, 4, 6, 7, 15, 16, 17

3.19 General

Excavate and lag shafts and excavate bells

Rebar and concrete

Jack design load

3.19.1 Install E/W 1

3.19.4 Install E/W 4

3.19.6 Install E/W 6

3.19.7 Install E/W 7

3.19.15 Install E/W 15

3.19.16 Install E/W 16

3.19.17 Install E/W 17

3.20 Piers K_c4, K_c5, K_c8, K_c9

3.20 General

Excavate and lag shafts and excavate bell

Rebar, as required, and concrete

Jack design load

3.20.4 Install K_c4

3.20.5 Install K_c5

3.20.8 Install K_c8

3.20.9 Install K_c9

3.21 Pier H_k x 5.25/7.85

3.21.1 Excavate and lag approach pit 45

3.21.2 Excavate and lag shaft and excavate bell HX5.25/7.85

3.21.3 Rebar and concrete

3.21.4 Jack design load

NOTES FOR FIGURE I.2 Continued

3.22 Excavate

3.22.1 Excavate and lag shaft from el 597 to 591

3.22.2 Excavate and lag under FIVP from el 600 to 591

3.22.3 Excavate and lag under EPA from el 609 to 591 between Column Lines 3/10 and 4/9

3.22.4 Excavate and lag under EPA from el 609 to 591 between Column Lines 4/9 and 5.3/7.8

3.23 Struts

3.23.1 Install ring system struts at el 599 from Column Lines 2.5/10.5 to 5/8.25

3.23.2 Tension ring system and preload struts

3.24 Excavation

3.24.1 Excavate and lag shaft from el 591 to 581 and install level D wale

3.24.2 Excavate and lag under FIVP and EPA from el 591 to 581

3.25 Struts

3.25.1 Install struts and ring system at el 589-6 from Column Lines 2.5/10.5 to 5/8.25

3.25.2 Tension ring system and preload struts

3.26 Excavation

3.26.1 Excavate and lag shaft from el 581 to 571

3.26.2 Excavate and lag under FIVP and EPA from el 581 to 572

3.26.3 Excavate and lag under control tower from el 591 to 582 from Column Lines 5.3/7.8 to 6.6 between H and K_c

3.26.4 Install CT-Piers according to Item 3.30, except 3.30.3 and 3.30.4

3.26.5 Excavate and lag under control tower from el 582 to 571 from Column Lines 5.3/7.8 to 5.9/7.2 between H and K_c

NOTES FOR FIGURE I.2 Continued

3.27 Install struts in control tower at el 583-6 between Column Lines H and K_c from 5.3/7.8 to 5.9/7.2

3.28 Install tie rods between CT-13, 13A, 13B, and CT-15, 15A, and 15B at el 581

3.29 Excavate and lag under control tower from el 571 to 563 from Column Lines 5.3/7.8 to 5.9/7.2

3.30 Excavate subgrade from el 563 to 562 and place mud mat from Column Lines 5.3/7.8 to 5.9/7.2

3.31 Piers CT-13A, 13B, 14A, 14B, 15A, 15B

3.31 General

Excavate and lag shaft

Rebar and concrete to el 582

3.31.1 Install 13A

3.31.2 Install 13B

3.31.3 Install 14A

3.31.4 Install 14B

3.31.5 Install 15A

3.31.6 Install 15B

PHASE IV

4.1 Concrete between CT piers

4.1.1 Grout CT pier horizontal rebar tubes

4.1.2 Rebar and concrete between Piers CT-5 and CT-8

4.2 Concrete wall between Pier CT-13 and Column Line K_c from el 582 and 601-607 (varies)

4.3 Concrete wall between Pier CT-14 and Column line K_c from el 582 to 601-607 (varies)

4.4 Concrete wall between Pier CT-15 and Column Line K_c from el 582 to 601-607 (varies)

NOTES FOR FIGURE I.2 Continued

4.5 Concrete permanent wall

4.5.1 Rebar and concrete from el 562 to 571 in control tower between Column Lines H.2 and K.2 from 5.3/7.85 to 5.6/7.6

4.5.2 Rebar and concrete from el 571 to 586 between Column Lines H.2 and K.2 from 5.3/7.85 to 5.6/7.6

4.5.3 Rebar and concrete from el 571 to 586 between column lines 3.2/10.1 and 5.3/7.85

4.5.4 Rebar and concrete from el 571 to 586 between Column Lines H.2 and K.2 from 5.3/7.85 to 5.6/7.6

4.5.5 Rebar and concrete from el 586 to 598 between Column Lines 3.2/10.1 and 5.3/7.85

4.6 Backfill

4.6.1 Backfill control tower from el 571 to 582 between Column Lines 5.6/7.6 and 5.9/7.2

4.6.2 Backfill EPA between permanent wall and containment from el 571 to 598

4.6.3 Backfill EPA between permanent wall and temporary wall from el 571 to 598

4.6.4 Backfill access shaft and FIVP to el 600

4.6.5 Backfill access shaft from el 600 to 634

4.6.6 Restrut at el 585 and remove struts at el 583-6 in EPA and control tower

4.6.7 Restrut at el 597 and remove struts at el 598 in EPA and control tower

4.7 Load transfer to permanent wall

4.7.1 Transfer load to east/west wall

4.7.2 Remove grillages at Piers E/W 2, 5, 8

4.8 Load test permanent wall

4.8.1 Load test east/west wall

4.8.2 Load test control tower south wall

4.8.3 Verify and redistribute control tower load

NOTES FOR FIGURE I.2 Continued

4.9 Final connections

4.9.1 Install horizontal dowels at Column Lines H x 5.6/7.6

4.9.2 Grout horizontal dowels at Column Lines H x 5.6/7.6

4.9.3 Install vertical dowels, east/west permanent wall

4.9.4 Grout vertical dowels, east/west permanent wall

4.9.5 Install vertical dowels, control tower permanent wall, Column Line K_c

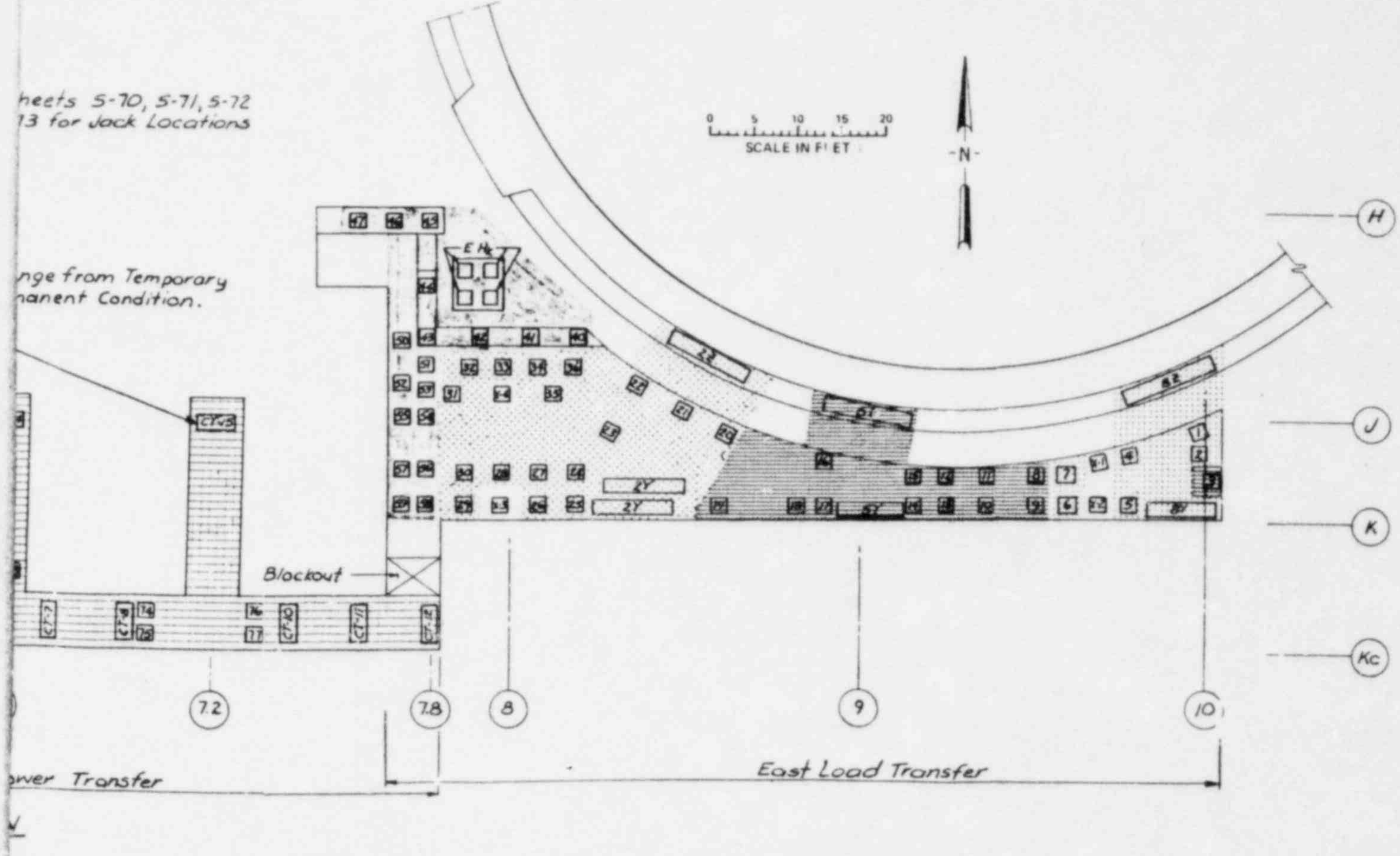
4.9.6 Grout vertical dowels, control tower permanent wall, Column Line K_c

4.10 Complete turbine building

4.10.1 Encase jackstands at all piers supporting turbine building except Pier E/W 2, 5, 8

4.10.2 Encase jackstands at Piers E/W 2, 5, 8

4.10.3 Backfill drifts and grout



, auxiliary building underpinning

ime/Hanson Engineers drawing S-74, Revision 2, preliminary)

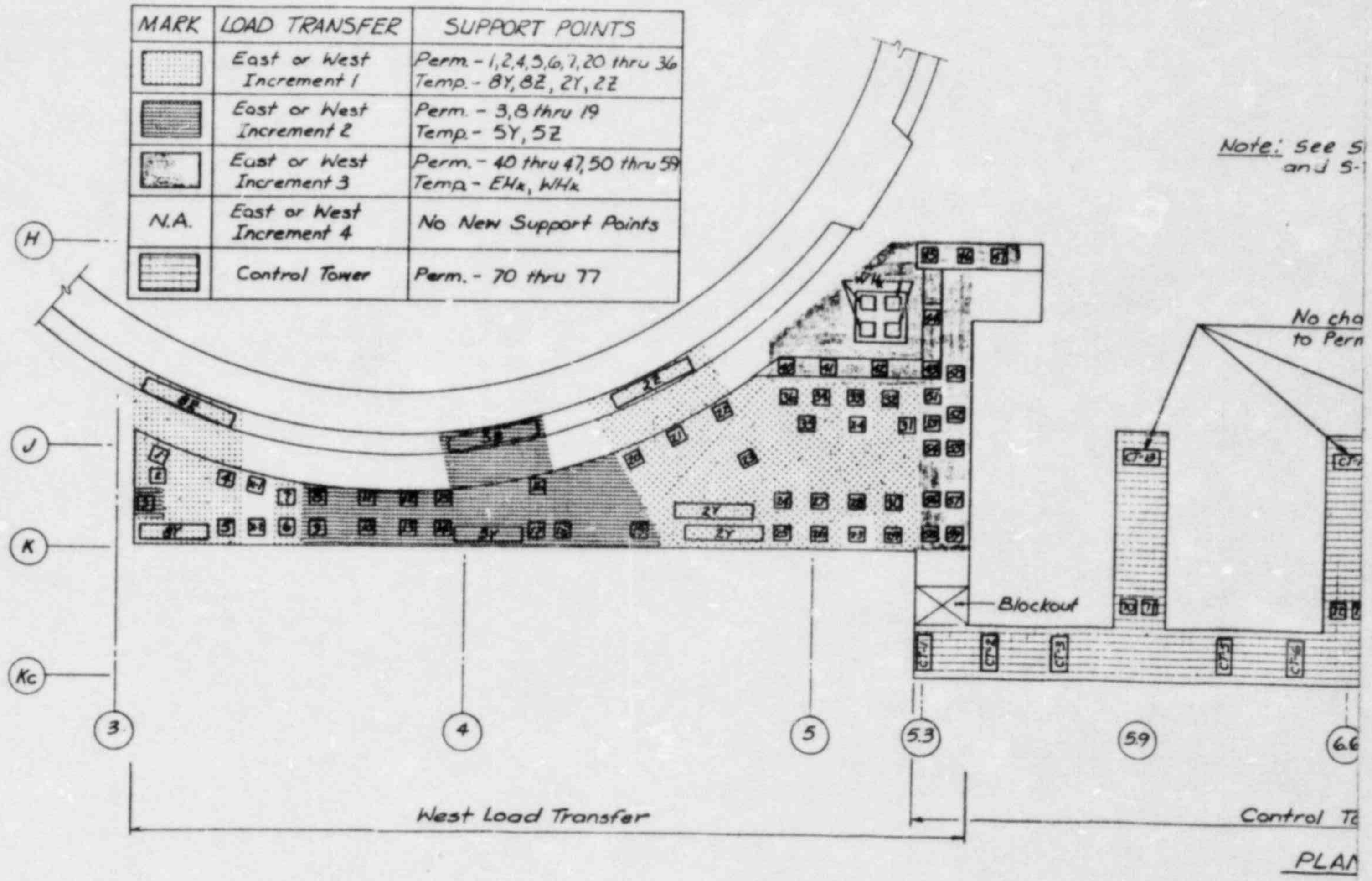


Figure 1.3 Jacking sequence,
(Source: Mergent

Notes for Figure I.3

1. SEQUENCE OF LOAD TRANSFER

   A: CONTROL TOWER - USE CONTROL TOWER LOAD VALIDATION CRITERIA AS SHOWN THIS SHEET.
   B: EAST AND WEST TRANSFER - USE INCREMENTS 1 THRU 4 AS SHOWN THIS SHEET. THESE TRANSFERS CAN BE DONE INDEPENDENTLY.
   C: CONTROL TOWER REDISTRIBUTION - USE PROCEDURE AS SHOWN THIS SHEET. THIS CAN OCCUR AFTER EAST OR WEST TRANSFER IS COMPLETE.

2. ADDITIONAL REQUIREMENTS - LOAD TRANSFER

   A: SEE TURBINE BUILDING TRANSFER CRITERIA FOR EAST AND WEST TRANSFERS.
   B: SEE PROCEDURES IN AUX. WING STRUCTURE MOVEMENT CRITERIA IF MOVEMENTS IN EXCESS OF SPECIFICATIONS OCCUR.
   C: SEE TABLE ON SHEET S-74A FOR DETAILED JACKING LOADS AND LOCATIONS.

3. PRIOR TO EAST AND WEST LOAD TRANSFER, PUT GRILLAGE 5Y AND 5Z JACKS UNDER PRESSURE, WITH WEDGES HAND TIGHT.

4. TEMPORARY LOADS SHOWN FOR THE AUXILIARY WING BASED ON 500 KIPS AT GRILLAGE 8Z, 1100 KIPS AT GRILLAGE 8Y, 900 KIPS AT GRILLAGE 5Z, 2200 KIPS AT GRILLAGE 5Y, 800 KIPS AT GRILLAGE 2Z AND 3400 KIPS AT GRILLAGE 2Y. THE TEMPORARY LOAD ON THE CONTROL TOWER IS BASED ON 1300 KIPS AT CT1, 2, 11 & 12 AND 1100 KIPS AT CT3 THRU 10. IF THE ACTUAL JACKED LOADS PLACED ON THESE TEMPORARY SUPPORTS DIFFER, THIS LOAD TRANSFER SCHEME SHOULD BE ADJUSTED ACCORDINGLY.

5. JACKING PRESSURE BASED ON 230 TON JACK WITH 8 IN. DIAMETER RAM AND 562 TON JACK WITH 12 IN. DIAMETER RAM.

6. THE DESIGN LOADS ARE 220 KIPS FOR THE 230 TON JACKS AND 495 KIPS FOR THE 562 TON JACKS. THE COLUMN TOWERS ARE DESIGNED FOR 1.5 TIMES THE DESIGN LOAD. THE JACKS HAVE A CAPACITY OF 2.25 TIMES THE DESIGN LOAD. SYSTEM DESIGN PRESSURE FOR JACKS IS 4,400 PSI.

CONTROL TOWER LOAD VALIDATION CRITERIA

THIS TRANSFER OCCURS BEFORE EAST AND WEST LOAD TRANSFER ARE STARTED.

STAGE A:

CTV.1 AFTER CT1 THRU CT15 AND INTERMEDIATE CONCRETE IS FINISHED, LOAD CT1 THRU CT15 WITH LOADS AS SHOWN IN TABLE S-74A.

CTV.2 LOAD TRANSFER COMPLETE. MAINTAIN PRESSURES IN JACKS, START LOAD TEST OF SOUTH WALL - CONTROL TOWER. SEE SPECIFICATIONS FOR ACCEPTANCE CRITERIA.

INCREMENT 1:

STAGE A:

1.A.1 ADD 25% LOAD TO INCREMENT 1 PERMANENT JACKS.

1.A.2 REMOVE 25% LOAD FROM TEMPORARY GRILLAGE 2Y, 2Z, 8Y & 8Z.

1.A.3 IF GRILLAGE 5Y & 5Z SHOWS INCREASE IN PRESSURE, CONVERT PRESSURE TO LOAD AND JACK ADDITIONAL LOAD INTO PERMANENT JACKS TO DECREASE PRESSURE AT GRILLAGE 5Y & 5Z TO ORIGINAL PRESSURE.

1.A.4 IF PERMANENT WALL JACKS SHOWS INCREASE IN PRESSURE DUE TO REMOVAL OF LOAD AT GRILLAGE 2Y, 2Z, 8Y & 8Z, CONVERT THIS PRESSURE TO LOAD AND SUBTRACT FROM NEXT STAGE ADDED LOAD.

Notes for Figure I,3 Continued

STAGE B:

1.B.1 ADD 25% LOAD TO INCREMENT 1 PERMANENT JACKS, ADJUSTED IF REQUIRED.
1.B.2 REMOVE 25% LOAD FROM TEMPORARY GRILLAGE 2Y, 2Z, 8Y & 8Z.
1.B.3 CHECK PRESSURE AT GRILLAGE 5Y & 5Z - SIMILAR TO 1.A.3.
1.B.4 CHECK CHANGE IN PRESSURE IN PERMANENT JACKS, (SIMILAR TO 1.A.4.

STAGE C:

1.C.1 ADD 25% LOAD TO INCREMENT 1 PERMANENT JACKS, ADJUSTED IF REQUIRED.
1.C.2 REMOVE 25% LOAD FROM TEMPORARY GRILLAGE 2Y, 2Z, 8Y & 8Z.
1.C.3 CHECK PRESSURE AT GRILLAGE 5Y & 5Z (SIMILAR TO 1.A.3).
1.C.4 CHECK CHANGE IN PRESSURE IN PERMANENT JACKS, (SIMILAR TO 1.A.4).

STAGE D:

1.D.1 ADD 12% LOAD TO INCREMENT 1 PERMANENT JACKS, ADJUSTED IF REQUIRED.
1.D.2 REMOVE FINAL 25% FROM TEMPORARY GRILLAGE 2Y, 2Z, 8Y & 8Z.
1.D.3 CHECK PRESSURE AT GRILLAGE 5Y & 5Z, (SIMILAR TO 1.A.3).
1.D.4 AT THIS POINT GO INTO LOAD MAINTENANCE FOR 72 HOURS, MONITOR PRESSURE AT GRILLAGE 5Y & 5Z, IF THE PRESSURE AT GRILLAGE 5Y & 5Z CHANGES BY MORE THAN 33%, CONVERT THIS PRESSURE TO LOAD AND JACK ADDITIONAL LOAD INTO PERMANENT JACKS TO DECREASE PRESSURE AT GRILLAGE 5Y & 5Z TO ORIGINAL PRESSURE.
1.D.5 THE INCREMENT 1 JACKING SYSTEM SHALL BE MAINTAINED AT ONE UNIFORM SYSTEM PRESSURE.
1.D.6 AT THIS POINT 65% OF WING LOAD SUPPORTED ON PERMANENT SUPPORT JACKS AND 35% ON GRILLAGE 5Y & 5Z.
1.D.7 REMOVE BEAM 1 IN GRILLAGE 8 SUPPORT SYSTEM TO ALLOW PLACEMENT OF COLUMN 3 PERMANENT JACKING TOWER.

INCREMENT 2:

STAGE A:

2.A.1 ACTIVATE INCREMENT 2 PERMANENT JACKS (0 LOAD).
2.A.2 REMOVE 25% LOAD FROM TEMPORARY GRILLAGE 5Y & 5Z.
2.A.3 INCREMENT 1 GROUP SYSTEM PRESSURE WILL INCREASE
2.A.4 ADD PRESSURE IN INCREMENT 2 JACKS TO REDUCE SYSTEM PRESSURE TO PRESSURE AT END OF INCREMENT 1, BUT DO NOT ADD PRESSURE GREATER THAN 25% OF DESIGN PRESSURE.

STAGE B:

2.B.1 REMOVE 25% LOAD FROM TEMPORARY GRILLAGE 5Y & 5Z.
2.B.2 INCREMENT 1 GROUP SYSTEM PRESSURE WILL INCREASE
2.B.3 ADD PRESSURE IN INCREMENT 2 JACKS TO REDUCE SYSTEM PRESSURE TO PRESSURE AT END OF INCREMENT 1, BUT DO NOT ADD PRESSURE GREATER THAN 25% OF DESIGN PRESSURE.
2.B.4 AT THIS POINT 83% OF WING LOAD SUPPORTED ON PERMANENT SUPPORT JACKS AND 17% ON GRILLAGE 5Y & 5Z.

INCREMENT 3:

STAGE A :

3.A.1 ACTIVATE INCREMENT 3 PERMANENT JACKS (0 LOAD).
3.A.2 REMOVE 25% LOAD FROM HK PIT.
3.A.3 INCREMENT 1 & 2 GROUP PRESSURE WILL INCREASE.
3.A.4 ADD PRESSURE TO INCREMENT 3 JACKS TO REDUCE INCREMENT 1 & 2 PRESSURES WITH MINIMUM ADDED LOAD 25% DESIGN LOAD.

Notes for Figure I,3 Continued

STAGE B:

3.B.1 REMOVE 25% LOAD FROM HK PIT
3.B.2 INCREMENT 1 & 2 GROUP PRESSURE WILL INCREASE.
3.B.3 ADD PRESSURE TO INCREMENT 3 JACKS WITH MINIMUM ADDED LOAD 25% DESIGN LOAD.

STAGE C:

3.C.1 REMOVE 25% LOAD FROM HK PIT.
3.C.2 INCREMENT 1 & 2 GROUP PRESSURE WILL INCREASE.
3.C.3 ADD PRESSURE TO INCREMENT 3 JACKS WITH MINIMUM ADDED LOAD 25% DESIGN LOAD.

STAGE D:

3.D.1 REMOVE FINAL 25% LOAD FROM HK PIT.
3.D.2 INCREMENT 1 & 2 GROUP PRESSURE WILL INCREASE.
3.D.3 ADD PRESSURE TO INCREMENT 3 JACKS WITH MINIMUM ADDED LOAD 12% DESIGN LOAD.
3.D.4 AT THIS TIME, INCREMENT 1 & 3 JACKS SHOULD BE AT THE SAME PRESSURE AND THESE SYSTEMS COMBINED. IF INCREMENT 2 JACKS ARE AT SAME PRESSURE AS 1 & 3, COMBINE IT ALSO.

INCREMENT 4:

STAGE A:

4.A.1 REMOVE 25% LOAD FROM TEMPORARY GRILLAGE 5Y & 5Z.
4.A.2 INCREMENT 1, 2 & 3 JACKS WILL INCREASE.
4.A.3 IF INCREMENT 2 JACKS NOT COMBINED WITH INCREMENTS 1 & 3, ADD PRESSURE TO INCREMENT 2 JACKS WITH MINIMUM ADDED LOAD 25% DESIGN LOAD, COMBINE WITH INCREMENTS 1 & 3 IF AT SAME PRESSURE.
4.A.4 IF INCREMENT 2 JACKS COMBINED WITH INCREMENT 1 & 3, MAINTAIN UNIFORM PRESSURE IN TOTAL SYSTEM.

STAGE B:

4.B.1 REMOVE FINAL 25% LOAD FROM TEMPORARY GRILLAGE 5Y & 5Z.
4.B.2 INCREMENT 1, 2 & 3 JACKS WILL INCREASE.
4.B.3 IF INCREMENT 2 JACKS NOT COMBINED WITH INCREMENTS 1 & 3, ADJUST PRESSURE IN INCREMENT 2 JACKS TO MATCH INCREMENT 1 & 3 AND COMBINE.
4.B.4 IF INCREMENT 2 JACKS COMBINED WITH INCREMENT 1 & 3, MAINTAIN UNIFORM PRESSURE IN TOTAL SYSTEM.
4.B.5 AT THIS POINT 100% OF WING SUPPORTED ON PERMANENT SUPPORT JACKS AND 0% ON TEMPORARY GRILLAGE.

STAGE C:

4.C.1 INCREMENT 1, 2 & 3 PERMANENT JACKS COMBINED AS ONE SYSTEM.
4.C.2 IF SYSTEM PRESSURE IS BELOW 95% OF SYSTEM DESIGN PRESSURE, INCREMENT SYSTEM PRESSURE WITH 2% INCREMENTS. STOP INCREMENTING AT 95% SYSTEM DESIGN PRESSURE, OR STOP INCREMENTING AT FIRST UPWARD MOVEMENT.
4.C.3 IF FIRST UPWARD MOVEMENT AT GRILLAGE 8 END OF WING OCCURS AT PRESSURE BELOW 95% SYSTEM DESIGN PRESSURE, ADD LOAD TO GRILLAGE 2 END OF WING BY LOADING X3 & X4 JACKS. LOAD AT 100^K INCREMENTS UNTIL 95% DESIGN LOAD IS ACHIEVED OR 495^K PER JACK OR AT UPWARD MOVEMENT.

IF FIRST UPWARD MOVEMENT AT GRILLAGE 2 END OF WING OCCURS AT PRESSURE BELOW 95% SYSTEM DESIGN PRESSURE, ADD LOAD TO GRILLAGE 8 END BY LOADING X1 & X2 JACKS. LOAD AT 100^K INCREMENTS UNTIL 95% DESIGN LOAD IS ACHIEVED OR 495^K PER JACK OR AT UPWARD MOVEMENT.

Notes for Figure I.3 Continued

4.C.4 LOAD TRANFER COMPLETE. MAINTAIN SYSTEM PRESSURE. START LOAD TEST OF EAST OR WEST TRANSFERS. SEE SPECIFICATONS FOR ACCEPTANCE CRITERIA.

CONTROL TOWER REDISTRIBUTION

FINAL ADJUSTMENT OF LOADS ON SOUTH WALL OF CONTROL TOWER CAN OCCUR AFTER EAST OR WEST LOAD TRANSFERS COMPLETE.

STAGE A:

CTR.1 INSTALL WEDGE PLATES AT LOCATIONS 70 THRU 72 AS SHOWN. DRIVE WEDGES TIGHT.
CTR.2 ACTIVATE CT1 THRU CT12 JACKS AND LOAD VALUE AS SHOWN IN TABLE S-74A.

TURBINE BUILDING - LOAD TRANSFER CRITERIA

WHEN GRILLAGE Y & Z JACKS ARE ACTIVATED, THE GRILLAGE X JACK SHALL BE ACTIVATED TO SPECIFIED LOADS AND MAINTAINED BY BLEEDING JACKS TO COMPENSATE FOR SOIL REBOUND.

AUX. WING STRUCTURE MOVEMENT CRITERIA

1. DURING LOAD TRANSFER

   A. IF DOWNWARD MOVEMENT OF GRILLAGE 8 END OCCURS, ENGAGE X1 & X2 JACKS, IF EXCESSIVE MOVEMENT CONTINUES REINGAGE GRILLAGE JACKS. IF DOWNWARD MOVEMENT OF GRILLAGE 2 END OCCURS, ENGAGE X3 & X4 JACKS, IF EXCESSIVE MOVEMENT CONTINUES REINGAGE GRILLAGE JACKS.

   B. IF DOWNWARD MOVEMENT OF THE STRUCTURE IN THE VICINITY OF KC LINE AT THE CONTROL TOWER OCCURS, INCREASE ACTIVE SYSTEM PRESSURE, IF EXCESSIVE MOVEMENT CONTINUES REINGAGE GRILLAGE JACKS.

2. DURING LOAD TEST

   A. IF DOWNWARD MOVEMENT OF GRILLAGE 8 END OCCURS, ADD LOAD TO X1 & X2 JACKS. IF DOWNWARD MOVEMENT OF GRILLAGE 2 END OCCURS, ADD LOAD TO X3 & X4 JACKS.

   B. IF DOWNWARD MOVEMENT OF THE STRUCTURE IN THE VICINITY OF KC LINE AT THE CONTROL TOWER OCCURS, INCREASE SYSTEM PRESSURE.

TABLE I.1 Permanent wall-load transfer jacking sequence

Group	Reference notes Fig. I.3	Loading
INCREMENT 1		
Increment 1 - 230-T jacks	1.A.1	Add 25% = 55^k = 1100 psi
Increment 1 - 562-T jacks	1.A.1	Add 25% = 124^k = 1100 psi
Temp. Support - 2Y	1.A.2	Remove 25% = 850^k
2Z	1.A.2	Remove 25% = 200^k
8Y	1.A.2	Remove 25% = 275^k
8Z	1.A.2	Remove 25% = 125^k
Increment 1 - 230-T jacks	1.B.1	Add 25% = 55^k = 1100 psi
Increment 1 - 562-T jacks	1.B.1	Add 25% = 124^k = 1100 psi
Temp. Support - 2Y	1.B.2	Remove 25% = 850^k
2Z	1.B.2	Remove 25% = 200^k
8Y	1.B.2	Remove 25% = 275^k
8Z	1.B.2	Remove 25% = 125^k
Increment 1 - 230-T jacks	1.C.1	Add 25% = 55^k = 1100 psi
Increment 1 - 562-T jacks	1.C.1	Add 25% = 124^k = 1100 psi
Temp. Support - 2Y	1.C.2	Remove 25% = 850^k
2Z	1.C.2	Remove 25% = 200^k
8Y	1.C.2	Remove 25% = 275^k
8Z	1.C.2	Remove 25% = 125^k
Increment 1 - 230-T jacks	1.D.1	Add 12% = 26^k = 530 psi
Increment 1 - 562-T jacks	1.D.1	Add 12% = 59^k = 530 psi
Temp. Support - 2Y	1.D.2	Remove final 25% = 850^k
2Z	1.D.2	Remove final 25% = 200^k
8Y	1.D.2	Remove final 25% = 275^k
8Z	1.D.2	Remove final 25% = 125^k
INCREMENT 2		
Temp. Support - 5Y	2.A.2	Remove 25% = 550^k
5Z	2.A.2	Remove 25% = 225^k
Increment 2 - 230-T jacks	2.A.4	Add 25% = 55^k = 1100 psi
Increment 2 - 562-T jacks	2.A.4	Add 25% = 124^k = 1100 psi
Temp. Support - 5Y	2.B.1	Remove 25% = 550^k
5Z	2.B.1	Remove 25% = 225^k
Increment 2 - 230-T jacks	2.B.3	Add 25% = 55^k = 1100 psi
Increment 2 - 562-T jacks	2.B.3	Add 25% = 124k = 1100 psi

Source: Mergentime/Hanson Engineers drawing S-74A, Revision 2, preliminary

TABLE I.1 (Continued)

Group	Reference notes Fig. I.3	Loading
INCREMENT 3		
Temp. Support - HK	3.A.2	Remove 25% = 575^k
Increment 3 - 230-T jacks	3.A.4	Add 25% = 55^k = 1100 psi
Increment 3 - 562-T jacks	3.A.4	Add 25% = 124^k = 1100 psi
Temp. Support - HK	3.B.1	Remove 25% = 575^k
Increment 3 - 230-T jacks	3.B.3	Add 25% = 1100 psi
Increment 3 - 562-T jacks	3.B.3	Add 25% = 124^k = 1100 psi
Temp. Support - HK	3.C.1	Remove 25% = 575^k
Increment 3 - 230-T jacks	3.C.3	Add 25% = 55^k = 1100 psi
Increment 3 - 562-T jacks	3.C.3	Add 25% = 124^k = 1100 psi
Temp. Support - HK	3.D.1	Remove final 25% = 575^k
Increment 3 - 230-T jacks	3.D.3	Add 12% = 26^k = 530 psi
Increment 3 - 562-T jacks	3.D.3	Add 12% = 59^k = 530 psi
INCREMENT 4		
Temp. Support - 5Y	4.A.1	Remove 25% = 550^k
5Z	4.A.1	Remove 25% = 225^k
Increment 2 - 230-T jacks	4.A.3	Add 25% = 55^k = 1100 psi
(Note 2) 562-T jacks	4.A.3	Add 25% = 124^k = 1100 psi
Temp. Support - 5Y	4.B.1	Remove final 25% = 550^k
5Z	4.B.1	Remove final 25% = 225^k
Increment 2 - 230-T jacks	4.B.3	Adjust as required
(Note 2) 562-T jacks	4.B.3	Adjust as required
Increments 1,2,3		
- 230-T jacks	4.C.2	Add 2% = 4.4^k = 90 psi
- 562-T jacks	4.C.2	Add 2% = 9.9^k = 90 psi
(Repeat as necessary)		(Note 3)
CONTROL TOWER LOAD VALIDATION		
CT-1 - 2-560-T jacks	CTV.1	650k = 5750 psi, each jack
CT-2 - 2-560-T jacks	CTV.1	650^k = 5750 psi, each jack
CT-3 - 2-560-T jacks	CTV.1	550^k = 4860 psi, each jack
CT-5 - 2-560-T jacks	CTV.1	550^k = 4860 psi, each jack
CT-6 - 2-560-T jacks	CTV.1	550^k = 4860 psi, each jack
CT-7 - 2-560-T jacks	CTV.1	550^k = 4860 psi, each jack
CT-8 - 2-560-T jacks	CTV.1	550^k = 4860 psi, each jack
CT-10 - 2-560-T jacks	CTV.1	550^k = 4860 psi, each jack

TABLE I.1 (Continued)

Group	Reference notes Fig. I.3	Loading
CONTROL TOWER LOAD VALIDATION (Cont.)		
CT-11 - 2-560-T jacks	CTV.1	650^k = 5750 psi, each jack
CT-12 - 2-560-T jacks	CTV.1	650^k = 5750 psi, each jack
CT-13 - 2-560-T jacks	CTV.1	700^k = 6190 psi, each jack
CT-14 - 2-560-T jacks	CTV.1	700^k = 6190 psi, each jack
CT-15 - 2-560-T jacks	CTV.1	700^k - 6190 psi, each jack
CONTROL TOWER REDISTRIBUTION		
CT-1 - 2-560-T jacks	CTR.2	463^k = 4090 psi, each jack
CT-2 - 2-560-T jacks	CTR.2	325^k = 2870 psi, each jack
CT-3 - 2-560-T jacks	CTR.2	550^k = 4860 psi, each jack
CT-5 - 2-560-T jacks	CTR.2	575^k = 5080 psi, each jack
CT-6 - 2-560-T jacks	CTR.2	400^k = 3540 psi, each jack
CT-7 - 2-560-T jacks	CTR.2	400^k = 3540 psi, each jack
CT-8 - 2-560-T jacks	CTR.2	575^k = 5080 psi, each jack
CT-10 - 2-560-T jacks	CTR.2	550^k = 4860 psi, each jack
CT-11 - 2-560-T jacks	CTR.2	325^k = 2870 psi, each jack
CT-12 - 2-560-T jacks	CTR.2	463^k = 4090 psi, each jack

INCREMENT 1

Permanent Jacks - 230-T - 1,2,5,6,20-33, and 35 (19 total each side)
562-T - 4,7,34, and 36 (4 total each side)

INCREMENT 2

Permanent Jacks - 230-T - 3,9-19 (12 total each side)
562-T - 8 (1 total each side)

INCREMENT 3

Permanent Jacks - 230-T - 40,41,42,46,50-59 (14 total each side)
562-T - 43,44,45, and 47 (4 total each side)

CONTROL TOWER REDISTRIBUTION - 70-77 (8 wedge plate assemblies)

NOTES

1. Activities within each box can be done in any order.
2. This step not required if Increment 2 jacks are combined with Increment 1 and 3 jacks.
3. Stop incrementing at jack pressure 4200 psi (approx 95% design load). System design pressure is 4400 psi.

Table I.1a Jack pressure conversion chart

Jack pressure (psi)	Load (kips) 230-T jack	Load (kips) 562-T jack
200	10	23
400	20	45
600	30	68
800	40	90
1000	50	113
1200	60	136
1400	70	158
1600	80	181
1800	90	204
2000	100	226
2200	111	249
2400	121	271
2600	131	294
2800	141	317
3000	151	339
3200	161	362
3400	171	385
3600	181	407
3800	191	430
4000	201	452
4200	211	475
4400	221	498
4600	231	520
4800	241	543
5000	251	565
5200	261	588
5400	271	611
5600	281	633
5800	292	656
6000	302	679

Note: Jack loads based on 8-in.-dia. ram for 230-T jack and 12-in.-dia. ram for 562-T jack.

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16. ABSTRACT *(200 words or less)*

This report supplements the Safety Evaluation Report, NUREG-0793, issued May 1982 by the Office of Nuclear Reactor Regulation of the U.S. Nuclear Regulatory Commission with respect to the application filed by Consumers Power Company, as applicant and owner, for licenses to operate the Midland Plant, Units 1 and 2 (Docket Nos. 50-329 and 50-330). The facility is located in the city of Midland in Midland County, Michigan. This supplement provides recent information regarding resolution of some of the open items identified in the Safety Evaluation Report. Most of the open items are associated with soils-related problems at the Midland site.

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